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SAFETY ISSUES OF RED-LIGHT RUNNING AND UNPROTECTED LEFT-TURN AT SIGNALIZED INTERSECTIONS

by

XUEDONG YAN

B.Sc. Xi’an University of Architecture & Technology
M.Sc. University of Central Florida

A dissertation submitted in partial fulfillment of requirements for the Ph.D. of Civil Engineering in the Department of Civil and Environmental Engineering in the College of Engineering and Computer Science at the University of Central Florida Orlando, Florida

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Major Professor: Essam Radwan
ABSTRACT

Crashes categorized as running red light or left turning are most likely to occur at signalized intersections and resulted in substantial severe injuries and property damages. This dissertation mainly focused on these two types of vehicle crashes and the research methodology involved several perspectives.

To examine the overall characteristics of red-light running and left-turning crashes, firstly, this study applied 1999-2001 Florida traffic crash data to investigate the accident propensity of three aspects of risk factors related to traffic environments, driver characteristics, and vehicle types. A quasi-induced exposure concept and statistical techniques including classification tree model and multiple logistic regression were used to perform this analysis.

Secondly, the UCF driving simulator was applied to test the effect of a proposed new pavement marking countermeasure which purpose is to reduce the red-light running rate at signalized intersections. The simulation experiment results showed that the total red-light running rate with marking is significantly lower than that without marking. Moreover, deceleration rate of stopping drivers with marking for the higher speed limit are significantly less than those without marking. These findings are encouraging and suggesting that the pavement marking may result in safety enhancement as far as right-angle and rear-end traffic crashes at signalized intersections.
Thirdly, geometric models to compute sight distances of unprotected left-turns were developed for different signalized intersection configurations including a straight approach leading to a straight one, a straight approach leading to a curved one, and a curved approach leading to a curved one. The models and related analyses can be used to layout intersection design or evaluate the sight distance problem of an existing intersection configuration to ensure safe left-turn maneuvers by drivers.
To my parents
Yan Xinmin and Zhang Junrong
Who encourage me to go after my dreams forever

To my wife, Fan
And my son, Yingnan (Michael)
Who awake up my spirit and responsibilities
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CHAPTER 1. INTRODUCTION

Crashes categorized as running red light or turning left to cross oncoming traffic are most likely to occur at signalized intersections. On a national basis, red-light running contributes to substantial numbers of motor vehicle crashes and injuries. At signalized intersections, especially in urban areas, a substantial proportion of motor vehicle angle collisions resulted from red-light running violation. On the other hand, left-turning traffic continues to be a major source of conflicts at intersections. Left-turning traffic constitutes between 10 to 15 percent of all approach traffic at an intersection yet they contribute approximately 45 percent of all traffic crashes. At signalized intersections, the typical left-turn accidents happen during the unprotected left-turn phase because of the left-turn vehicles that fail to yield right-of-way.

Disabling damage rate and functional damage rate for both red-light running and left-turn crashes are extremely high, because most points of impact is in front or lateral side of the vehicles and the speed of coming through vehicles is relatively higher. Therefore, the main objectives of this dissertation are to thoroughly investigate the characteristics of these two types of vehicle accidents and to explore related improvement countermeasures. The research efforts involved several perspectives including accident analyses of red-light running and left turning based on Florida crash database, driving
simulator experiment study to test the effect of pavement marking to reduce red-light running rates, and geometric models to calculate left-turn sight distance.

1.1 Accident analysis based on Florida crash database

To examine the overall characteristics of red-light running and left-turning crashes, this research applied 1999-2001 Florida traffic crash database to investigate the accident propensity of three aspects of risk factors related to traffic environments, driver characteristics, and vehicle types. A quasi-induced exposure concept and statistical techniques including classification tree and multiple logistic regression were used to perform this analysis. The crash data for the 1999-2001 periods were obtained from the Florida Department of Highway Safety and Motor Vehicles (DHSMV). The DHSMV data constitute a relational database that includes seven files. Each file deals with a specific aspect of traffic crashes. Files may be linked as needed to combine the information contained in each file. These files are:

1. EVENTS: this file contains general information about the crash characteristic and circumstance and includes 35 related variables, such as Time of crash, First harmful event, Day of week, Weather, Road surface condition, Number of lanes, Rural/urban, Traffic-way characters, Total number of vehicles, and other factors.
2. VEHICLE: This file includes 25 variables that contain information about the vehicles and vehicles actions in the traffic crash, such as Type of vehicle, Estimated speed, Posted speed, Disabling/functional damage, Vehicle movement, and other factors.

3. DRIVER: This file includes 23 variables that contain information about the drivers and condition or action of the driver that contribute to the crash, such as Driver age, Gender, Alcohol/drug use, Physical defect, Residence code, Injury severity, First contributing cause, Second contributing cause, Third contributing cause, and other factors.

4. PEDESTRIAN: This file includes 16 variables that deals with information on any pedestrians involved in the crash, such as Age, Alcohol/drug use, Physical defect, Race, Residence code, Pedestrian action, Injury severity, and other factors.

5. VIOLATIONS: This file lists the citations (if any) issued in connection with the traffic crash, by statute number.

6. PASSENGER: This file includes 10 variables that provide information about any passenger involved in the traffic crash, such as state of the residence, passenger age, location in vehicle, injury severity, and other factors.

7. D.O.T. SITE LOCATION: This file includes 9 variables that provide additional information about crash locations occurring on state roads only, such as DOT number of
lanes, DOT site location, DOT average daily traffic, and other factors. This data is supplied by the Department of Transportation.

The seven files can be linked each other, using the uniquely case report number. Especially, the driver and corresponding vehicle in a crash can be link together by a section number, which help to identify each specific case. Using this database, traffic engineers and safety researchers can examine the frequency of any type of crashes on the Florida basis and to identify the characteristics of such crashes and the drivers, vehicles, crash types, and environmental factors involved.

1.2 Driving Simulator Experiment to Test a Pavement Marking Countermeasure to Reduce Red-light Running Rates

The UCF driving simulator housed in the Center for Advanced Transportation Systems Simulation (CATSS) is an I-Sim Mark-II system with a high driving fidelity and immense virtual environments. The simulator is mounted on a motion base capable of operation with 6 degrees of freedom. It includes 5 channels (1 forward, 2 side views and 2 rear view mirrors) of image generation, an audio and vibration system, steering wheel feedback, operator/instructor console with graphical user interface, sophisticated vehicle dynamics models for different vehicle classes, a 3-dimensional road surface model, visual database with rural, suburban and freeway roads plus an assortment of buildings and operational traffic control devices, and a scenario development tool for creating real world driving conditions. The output data include detailed events pertaining to every
car’s steering wheel, accelerator, brake, every car’s speed and coordinates, and a time stamp. The sampling frequency is 60Hz.

In this research, the objective is to apply the UCF driving simulator to study an important traffic concept, pavement marking countermeasure, which is proposed to help drivers make a clear stop/go decision at the onset of yellow phase at signalized intersections. It is hoped that this marking would minimize signalized intersection accident rates. To test the effectiveness of the pavement marking countermeasure on red-light running, this study documented a simulation research based on a driving simulator experiment design. The pavement marking with word message ‘SIGNAL AHEAD’ is placed on the pavement of the upstream approach of a signalized intersection and is sufficient to permit vehicles cruising at speed limit to stop safely before reaching the intersection stop bar. The proposed policy is that, when drivers are located upstream of the marking at the yellow onset, they are encouraged to stop at the intersection if they are cruising around speed limit. On the other hand, when drivers are located downstream the marking at the yellow onset, they are encouraged to cross the intersection if they are cruising around speed limit. However, if their speeds are fairly lower than the speed limit, they should be able to brake and stop before the stop bar. Moreover, the proposed design may discourage unsure drivers with relatively higher approaching speed from crossing the intersection and hence reduce the danger of potential red-light running.
1.3 Geometric Models to Calculate Left-turning Sight Distance

During the unprotected left turn green phase at four-leg intersections on divided highways, left turn drivers from the major road need to accept proper gaps or lags to cross the opposing through traffic into the minor road. However, vehicles in the opposing left turn lane often block the left turner’s view. For that situation, available sight distance for left turners is a very important geometric design factor. Inadequate visibility of opposing through traffic can cause not only serious safety problem due to driver’s misjudging gap, but also increase intersection delay for the left turn traffic. Furthermore, if the intersection is located on a horizontal curve of the major road, the calculation method of available left-turn sight distance will be very different from that for intersections with linear approaches.

The 2001 AASHTO manual pointed out that the typical poor visibility of opposing through traffic usually occurs at signal intersections with medians wider than 18 ft and recommended two improvement methods, parallel offset left-turn lanes and tapered offset left-turn lanes. However, it did not provide the specific design guideline; nor did it present the related geometric design model. Based on literature research, currently there is a lack of literatures related to sight distance models when the signalized intersections have curve approaches. This study developed a series of sight-distance geometric models to compute left-turn sight distance for different signalized intersection configurations, presented visibility improvement effects of the two offset methods, and analyzed the relationship between available sight distance and related intersection geometric
parameters. The models and related analyses can be used to layout intersection design or evaluate the sight distance problem of an existing intersection configuration to ensure safe left-turn maneuvers by drivers.
CHAPTER 2. LITERATURE REVIEW

2.1 Safety Issues Related to Red-light Running Accidents

Red-light running contributes to substantial numbers of motor vehicle crashes and injuries on a national basis. Retting et al reported that drivers who run red-lights were involved in an estimated 260,000 crashes each year, of which approximately 750 are fatal, and the number of fatal motor vehicle crashes at traffic signals increased 18% between 1992 and 1998, far outpacing the 5% rise in all other fatal crashes (Retting et al., 2002). Motorists are more likely to be injured in crashes involving red-light running than in other types of crashes, according to analyses of police-reported crashes from four urban communities; occupant injuries occurred in 45% of the red-light running crashes studied, compared with 30% for all other crashes in the same communities.

In Texas, a report showed that the number of people killed or injured in red-light running crashes had increased substantially over the years. The increase (79 percent from 1975 to 1999) is similar to the increase in the number of people killed or injured in motor vehicle crashes in general, and is also similar to the increase in vehicle miles traveled in the state. About 16 percent of people killed in intersection crashes and 19–22 percent of people injured in intersection crashes are involved in red-light running (Quiroga et al., 2003).
According to the Federal Highway Administration (FHWA), the following traffic facts about red-light running were posted in its main website:

- Each year, more than 1.8 million intersection crashes occur.
- In 2000, there were 106,000 red-light running crashes that resulted in 89,000 injuries and 1,036 deaths.
- Preliminary estimates for 2001 indicate 200,000 crashes, 150,000 injuries, and about 1,100 deaths were attributed to red-light running.
- Overall, 55.8 percent of Americans admit to running red lights. Yet ninety-six percent of drivers fear they will get hit by a red-light runner when they enter an intersection.

Red-light running is a highly dangerous driving act and also it is the most frequent type of police-reported urban crash. A study provided 5,112 observations of drivers entering six traffic-controlled intersections in three cities. Overall, 35.2% of observed light cycles had at least one red-light runner prior to the onset of opposing traffic. This rate represented approximately 10 violators per observation hour (Porter and England, 2000). Another study conducted over several months at a busy intersection (30,000 vehicles per day) in Arlington, VA revealed violation rates of one red-light runner every 12 min. and during the morning peak hour, a higher rate of one violation every 5 min. A lower volume intersection (14,000 vehicles per day), also in Arlington, had an average of 1.3 violations per hour and 3.4 in the evening peak hour (Retting et al., 1998).
Thus, based on both previous research and accident data, red-light running crashes represent a significant safety problem that warrants attention.

### 2.1.1 Characteristics of red-light running

Retting, Ulmer, and Williams (1999) analyzed drivers’ characteristics involving fatal red-light running accidents using 1992–1996 data from the FARS and GES databases. For the analysis, they only considered fatal crashes for which one driver had committed a red-light running violation and both drivers were going straight prior to the crash. The following were the main findings of the study:

- Some 57 percent of fatal red-light running crashes occurred during the day. By comparison, 48 percent of other fatal crashes occurred during the day. However, fatal red-light running crashes that involved drivers less than 70 years old peaked around midnight, whereas fatal red-light running crashes that involved drivers 70 years old or older occurred primarily during the day.
- On average, 74 percent of red-light runners and 70 percent of non-runners were male. Of all nighttime red-light runners, 83 percent were male. Of all daytime red-light runners, 67 percent were male. It may be worth noting that male drivers accounted for roughly 61 percent of the vehicle miles traveled on U.S. roads, according to results from the 1995 Nationwide Personal Transportation Survey.
- Some 43 percent of red-light runners were younger than age 30. By comparison, 32 percent of non-runners were younger than age 30.
• Red-light runners were much more likely to drive with suspended, revoked, or otherwise invalid driver licenses. Younger drivers were more likely to be unlicensed.

From the perspective of crash types of red-light running, while most red-light running crashes involve at least two vehicles, crashes involving a single vehicle and an alternative transportation mode (pedestrian or bicyclist) can occur. A single vehicle, hit fixed object crash could occur when either the running-the-red violator or the opposing legal driver takes evasive action to avoid the other and crashes into an object, e.g. a signal pole. Also, a running-the-red violator can hit a pedestrian or bicyclist who is legally in the intersection.

A comprehensive report (FHWA, 2003) on red-light running issue concluded that the following crash types could be possible target crashes for a red-light study: Right-angle (side impact) crashes, Left turn (two vehicles turning), Left turn (one vehicle oncoming), Rear end (straight ahead), Rear end (while turning), and other crashes specifically identified as red-light running.

2.1.2 Reasons of red-light running

The FHWA report also pointed out that researchers reviewed the police reports of 306 crashes that occurred at 31 signalized intersections located in three states. Traffic-signal violation was established as a contributing factor and the reason for the violation was
provided in 139 of the crashes. The distribution of the reported predominant causes is as follows:

- 40 percent did not see the signal or its indication;
- 25 percent tried to beat the yellow-signal indication;
- 12 percent mistook the signal indication and reported they had a green-signal indication;
- 8 percent intentionally violated the signal;
- 6 percent were unable to bring their vehicle to a stop in time due to vehicle defects or environmental conditions;
- 4 percent followed another vehicle into the intersection and did not look at the signal indication;
- 3 percent were confused by another signal at the intersection or at a closely spaced intersection; and
- 2 percent were varied in their cause.

The above research results show that red-light running is a complex problem. There is no simple or single reason to explain why drivers run red lights. However, they can be classified into two types, intersection factors and human factors.

A study’s objective was to examine selected intersection factors and their impact on RLR crash rates and to establish a relationship between them. The results obtained from the model show that the traffic volume on both the entering and crossing streets, the type of
signal in operation at the intersection, and the width of the cross-street at the intersection are the major variables affecting red-light running crashes (Mohamedshah, 2000). The FHWA report summed that, among intersection factors are intersection flow rates, frequency of signal cycles, vehicle speed, travel time to the stop line, type of signal control, duration of the yellow interval, approach grade, and signal visibility (FHWA, 2003).

Bonneson et al. (2002) concluded that the following factors influence the frequency of red-light-running and related crash frequency:

- flow rate on the subject approach (exposure factor),
- number of signal cycles (exposure factor), phase termination by max-out (exposure factor)
- probability of stopping (contributory factor),
- yellow interval duration (contributory factor),
- all-red interval duration (contributory factor),
- entry time of the conflicting driver (contributory factor), and
- flow rate on the conflicting approach (exposure factor).

Human factors that can contribute to the occurrence of crashes include physical or physiological factors (e.g., strength, vision), psychological or behavioral factors (e.g., reaction time, emotion), and cognitive factors (e.g., attention, decision making) (Quiroga et al., 2003).
How intersection factors and human factors interact to increase or decrease the risk of red-light running varies considerably from intersection to intersection. Those factors point to the need to implement engineering countermeasures to improve traffic flow, improve visibility, help drivers make driving maneuvers and reduce conflicts. Other factors, especially related to deliberate illegal driving behaviors, point to the need to also implement strategies such as improved enforcement and public awareness.

Bonneson (2001) also discussed the factors that affect the driver’s decision to stop or proceed through the intersection upon seeing the onset of the yellow. There are three main components of the decision process: driver behavior (expectancy and knowledge of operation of the intersection), estimated consequences of not stopping and estimated consequences of stopping. What if the driver makes his decision to proceed through the intersection based on the factors above, but ends up running the red light? Bonneson divides red-light runners into two categories. The first is the intentional violator who, based on his/her judgment, knows they will violate the signal, yet he/she proceeds through the intersection. This type of driver is often frustrated due to long signal delays and perceives little risk by proceeding through the intersection. The second type of driver is the unintentional driver who is incapable of stopping or who has been inattentive while approaching the intersection. This may occur as a result of poor judgment by the driver or a deficiency in the design of the intersection. Bonneson further indicates that intentional red-light runners are most affected by enforcement countermeasures while unintentional red-light runners are most affected by engineering countermeasures.
2.2 Current Engineering Countermeasures for Red-Light Running

2.2.1 Overview of current engineering countermeasures

According to characteristics and reasons of red-light running, traffic engineers are trying to develop a number of methods to reduce the red-light running rate. Currently, engineering countermeasures include signal operation countermeasures (e.g., increasing the yellow interval duration, providing green extension, improving signal coordination, and improving signal phasing), motorist information countermeasures (e.g., improving sight distance, improving signal visibility and conspicuity, and adding advance warning signs), and physical improvement countermeasures (e.g., removing unneeded signals, adding capacity with additional traffic lanes, and flattening sharp curves). Signal operation countermeasures can effectively reduce the incidence of red-light running by improving traffic flow characteristics and by reducing the exposure of individual vehicles to situations that might result in red-light running. Motorist information countermeasures that focus on attracting the attention of drivers to the signal can effectively reduce the incidence of red-light running.

In recent years, a lot of researches are related to evaluation on effects of red-light camera implementation. In one side, the review of the effectiveness of those systems reveals that red-light cameras are effective deterrence tools and have a positive safety impact; even where the implementation of engineering countermeasures had not preceded the installation and operation of cameras. On the other side, the review also shows that red-
light cameras can contribute to an increase in the number of rear-end crashes; however, this effect is relatively small and temporary and camera presence (or the presence of warning signs) had no significant effect on red-running behavior (Quiroga et al., 2003). Furthermore, some report (The Red-light Running Crisis: Is it Intentional, 2001) questions whether motorists identified in Institute studies as red-light violators are, in fact, innocent drivers who were unable to stop in time to comply with the signals. The fact is that red-light cameras are designed to identify only deliberate violators, those who enter intersections well after the end of a yellow signal phase.

In this research, the purpose of pavement marking method is to help drivers make a clear decision at the onset of yellow phase to reduce red-light running and intersection accident rates, which also belong to motorist information countermeasures. Therefore, in the following section of this literature review, other motorist information countermeasures are paid more attentions to.

To help drivers make their decision at the onset of yellow, some motorist information countermeasures are implemented by enhancing the signal display or by providing advance information to the driver about the signal ahead. With the additional information, the probability that a driver will stop for a red signal may increase. Among them, the two most prevailing and controversial countermeasures are pre-yellow signal indication and advance warning signs.
2.2.2 Advance warning sign and Advance warning flashers

Advance warning signs forewarn drivers that they are approaching a signalized intersection. Figure 2-1 shows two types of warning signs. Figure 2-1a shows a sign that uses a “signal ahead” symbolic message. Flashing beacons sometimes accompany this sign to ensure drivers detect and interpret the sign’s meaning. Figure 2-1b shows a “Be Prepared to Stop When Flashing” sign. This sign has the beacons flashing only during the last few seconds of green. It is sometimes referred to as an “advance warning sign with active flashers.” In this mode, the flashing indicates when the signal indication is about to change from green to yellow. When flashing beacons accompany these advance warning signs, they are also named advance warning flashers (AWF). The purpose of AWF is to forewarn the driver when a traffic signal on his/her approach is about to change to the yellow and then the red phase. An effective AWF implementation is intended to minimize the number of vehicles in the dilemma zone during the change interval. In North America, there are three general types of advanced warning devices and the decision of which to use is based on engineering judgment. These AWFs include:

- Prepare to stop when flashing (PTSWF)—A warning sign, BE PREPARED TO STOP with two yellow flashers that begins to flash a few seconds before the onset of the yellow and continue to flash throughout the red phase. A WHEN FLASHING plaque is recommended in addition to the sign.
- Flashing symbolic signal ahead (FSSA)—Similar to previous type except the wording on the sign is replaced by a schematic of a traffic signal. The flashers operate as above.

- Continuous flashing symbolic signal ahead (CFSSA)—The sign displays a schematic of a traffic-signal symbol but in this case, the flashers operate continuously (i.e. they are not connected to the signal controller).

Figure 2-1: Advance warning sign and advance warning flashers

The location and timing of AWF are key considerations for the sign installation. The distance from AWF location to a signalized intersection must be equal to or greater than that required to perceive and react to the flasher and stop the vehicle safely. The timing refers to the length of time before the yellow interval of the downstream-signalized intersection at which the AWF starts flashing. Sayed et al. (1999) indicated that engineering judgment is often the principal guide for AWF installation according their literature findings. However, they also introduced practical guidelines for AWF
implementation used in British Columbia, which are recommended at provincial intersection s where one of the following conditions is satisfied:

- The posted speed limit on the roadway is 70 km/h or greater,
- The view of the traffic signals is obstructed because of vertical or horizontal alignment (regardless of the speed limit) so that a safe stopping distance not available,
- There is a grade in the approach to the intersection that requires more than the normal braking effort, or
- Drivers are exposed to many kilometers of high-speed driving (regardless of posted speed limit) and encounter the first traffic signal in a developed community.

Location of AWFs is calculated by the following equation:

\[ D = VT + \frac{V^2}{2g(f \pm G)} \]

Where

- \( V = 85^{th} \) percentile operating speed or posted speed limit (m/s)
- \( T = \) reaction time (1.0 s)
- \( g = \) gravitational acceleration (9.81 m/s\(^2\))
- \( f = \) friction factor for wet surfaces, and
- \( G = \) grade (m/100m)
The length of the advanced warning time before the yellow interval of the downstream-
signalized intersection at which the AWF starts flashing is calculated by the following
equation:

\[ AW = \frac{D + D_p}{V} \]

Where

- \( AW \) = advanced warning time
- \( D \) = Distance between the AWF and the signal’s stop line
- \( D_p \) = Minimum distance at which the flashers can be perceived (21.3m)

Studying drivers’ reactions to advance warning flashers in the field is highly problematic
because these devices are relatively uncommon and because it is difficult or impossible to
establish a controlled experimental environment in which variable parameters can be
tested individually. Smith (2001) employed the Human Factors Research Lab’s driving
simulator to investigate effects of Advance Warning Flashers at signalized intersections
on simulated driving performance. After analysis of the large volume of experimental
data, the researchers concluded that AWFs often improve stopping behavior at suitable
intersections. But as is often seen in human factors research, human response to a
complex situation is not as simple as a linear relationship. In this case, variability in
human response resulted in some drivers making a more aggressive—and risky—
decision to proceed through the intersection. This finding has obvious implications for
field implementation of advance warning flashers at dangerous intersections (Smith,
2001).
Sayed et al. (1999) utilized and analyzed data from British Columbia using two different methods. Models were used to develop expected accident rates at 106 signalized intersections for total, severe and rear-end accidents. Twenty-five of these intersections had AWFs. Although the results indicate that intersections with AWFs have a lower frequency of accidents, the difference between those with AWFs and those without is not statistically significant. An additional before-and-after study was performed for the 25 intersections equipped with AWFs to estimate the accident reduction specific to each location and its approach volumes. A correlation was found between the magnitude of the minor approach traffic volumes and the accident reduction capacity of AWFs, showing that AWF benefits exist at locations with moderate to high minor approach traffic volumes (minor street AADT of 13,000 or greater).

2.2.3 Traffic light change anticipation system

The Traffic Light Change Anticipation System (TLCAS) utilizes flashing amber during the last few seconds of the green phase. The flashing amber is considered to be a legal green signal, and is used to warn drivers of the impending termination of the green phase. Some findings indicated that this pre-yellow signal indication could help drivers react more safely to the impending onset of yellow; however, other evaluations showed that the flashing amber phase was associated with an increase in rear-end accidents and negligible changes in right-angle collisions (Quiroga et al., 2003).
A research study used a driving simulator to study the efficiency of TLCAS. Eighteen males and twenty-three females were drawn from the student and staff population at Arizona State University (Newton, 1997). The simulator uses an IBM 486 platform, and incorporates a rear projection system that projects the roadway, intersections, and buildings. The results of the experiment showed an increased variability in first response five times larger than the regular program. This finding, in conjunction with traditional measures, indicates that the new system performs comparably to an increased amber duration by increasing the potential for conflicting decisions between successive drivers approaching an intersection. Altogether, the results suggested that this alternative signal phasing program would not improve intersection safety.

Another study evaluating the effect of TLCAS using collected data in three different countries, Austria, Switzerland and Germany (Koll et al., 2002). The researchers discussed the results of extensive measurements of the stopping behavior of drivers during signal programs with and without flashing green before amber. The analysis showed that the flashing green increases the number of early stops, as drivers tend to underestimate the duration of the time to the end of yellow. However they also indicated that it produces a large option zone, where drivers can both safely stop and cross. This large option zone generates a period of uncertainty, where a following driver cannot easily predict, if the car in front will stop or cross, so that it could lead to an increased number of rear end collisions.

2.2.4 Rumble strips
Another warning device that has been used to alert drivers to the presence of a signal is transverse rumble strips (FHWA, 2003). Rumble strips are a series of intermittent, narrow, transverse areas of rough-textured, slightly raised, or depressed road surface. The rumble strips provide an audible and a vibro-tactile warning to the driver. When coupled with the SIGNAL AHEAD warning sign and also the pavement marking word message— SIGNAL AHEAD—the rumble strips can be effective in alerting drivers of a signal with limited sight distance. There are no known studies reporting on how this treatment can reduce red-light violations or the resulting crashes; hence their use should be restricted to special situations. If used, they should be limited to lower-speed facilities (less than 40 mph) and be reserved for locations where other treatments have not been effective.

However, according to literature findings, there is no related pavement marking countermeasure to provide drivers yellow phase information and diminish the likelihood of red-light running rate. This research introduced a pavement marking design to help drivers make a clear decision at the onset of yellow phase to reduce red-light running and intersection accident rates.

### 2.3 Safety Issues Related to Left-turning Accidents

Some of the most dangerous accidents are those involving left-turning vehicles, because the relative impact forces are high so the propensity for injury and damage is great.
According to the 1991 statistics (Wang and Knipling, 1994), there were 413,000 left-turn crashes occurred at intersections, which constituted 6.8 percent of all police-reported crashes. These crashes resulted in 295,000 injuries, including 41,000 fatal or incapacitating injuries. At signalized intersections, the typical left-turn accidents happen during the unprotected left-turn phase due to the left-turn vehicles that fail to yield right-of-way. The factors that affect the unprotected left-turn accidents are often interactive and cannot usually be isolated completely from one another. Based on the literature review, the main factors that adversely affect safety of the left-turn traffic include incorrectly accepting a gap between the coming-through traffic, sight distance obstruction caused by opposing left-turning vehicles, left-turn driver’s distraction, and misunderstanding the intersection signal phase.

Incorrect gap acceptance may cause around 30 percent left-turn accidents (Chovan et al., 1994). There are several factors that have significant effects on driver’s gap acceptance, including driver’s age and gender, vehicle speed, major street volume, intersection geometry, minor road approach grade, and movement turn angle (Yan et al., 2003; Kyte et al. 1996). The perceptual task of unprotected turn left during at a signalized intersection requires a driver to integrate speed and distance information of a longitudinally coming though vehicle moving in depth without change in visual direction. Through both simulator and field measures, Staplin (1995) indicated that older drivers show relative insensitivity to vehicle approach speed in left-turn maneuvers across the major road traffic when compared with younger drivers. She pointed out that this
produces a reliance on perceived distance alone increasing the risk of accidents if there is a lone speeder in the traffic scheme.

Aside from gap judgment, an additional concern is whether left-turning drivers understand the traffic meaning conveyed by the signal and any ancillary (regulatory) signs. Curtis et al. (1988) found that the circular green indication under permitted control was correctly interpreted by approximately 60 percent of the subjects. For protected-only operations, the green arrow (with circular red for through movement) was correctly answered by approximately 75 percent of drivers. For protected/permitted operation, the circular green alone was correctly answered by only 50 percent of the respondents, while the green arrow in combination with the circular green had approximately 70 percent correct responses.

Available sight distance for left-turners is a very important geometric design factor. Inadequate visibility of opposing through traffic can cause a serious safety problem due to driver’s misjudging the gap. The alignment of opposite left-turn lanes and the horizontal and vertical curvature on the approaches are the principal geometric design elements that possibly restrict available sight distance to a left-turning driver (FHWA, 2001). From the perspective of traffic operation, several studies pointed out that vehicles in the opposite left-turn lane waiting to turn left may obstruct the (left-turning) driver's view of oncoming traffic in the through lanes, especially for intersections with relatively wider medians (McCoy et al., 1992; Joshua and Saka, 1992). Totally, there may be 24.4 percent left-turn accident resulted from driver’s view obstruction (Chovan et al., 1994).
In addition, driver’s distraction may also play a significant role in left-turn accidents. Cooper and Zheng (2002) conducted an experiment that testified the negative effect of driver’s distraction on left-turning gap acceptance decision-making. Moreover, cell phone use could be a potential source for the driver’s distraction and car crashes. Laberge-Nadeau et al. (2003) reported that experimental studies conducted on driving simulators or on specially equipped vehicles have shown that the use of wireless phones while driving affects the performance of the driver, particularly the reaction time, mental load and lateral control of the vehicle.

2.4 Left-turn Sight Distance

During the unprotected left-turn green phase at four-leg intersections on divided highways, left-turning drivers from the major road need to accept proper gaps or lags to cross the opposing through traffic into the minor road. However, vehicles in the opposing left-turn lane often block the left-turner’s view. For that situation, available sight distance for left-turners is a very important geometric design factor. Inadequate visibility of opposing through traffic can cause not only a serious safety problem due to driver’s misjudging the gap, but also increase intersection delay for the left-turn traffic. A number of related studies had shown that sight distance problems at intersections usually result in a higher accident rate (Mitchell, 1972; Hanna, et al., 1976; David and Norman, 1979). McCoy et al. (1992) reported that in California, signalized intersections with opposing
left-turn lanes were found to have significantly more accidents than intersections without opposing left-turn lanes, which is attributed primarily to sight distance obstructions caused by opposing left-turn vehicles.

AASHTO (2001) reported that the typical poor visibility of opposing through traffic usually occurs at intersections with medians wider than 18 ft. A related study suggested the use of protected-only left-turn phases when medians are wider than 18 ft (Reilly, et al, 1980). To avoid the sight distance problem for left-turners, the AASHTO design guide (2001) recommended two methods to highway designers. One is a parallel offset left-turn lane; the other is a tapered offset left-turn lane. Both of these designs can reduce the width of the medial separators, maximize the offset between the opposing left-turn lanes, and place vehicles waiting to make a left-turn as far to the left as practical. The advantages of offsetting left lanes are improving visibility, decreasing the probability of left-turn accidents, and maintaining the design capacity of left-turn traffic. Especially, the tapered offset is helpful to the left-turn maneuver of longer vehicles, such as logging trucks. However, AASHTO did not provide the specific design guideline; nor present the related geometric design model. In addition, it is noticeable that the sight distance problem can also occur with medians narrower than 18 ft (1992), although AASHTO did not give any suggestions for such cases.

In previous studies, Joshua (1992) and McCoy et al. (1992) developed geometric models for parallel left-turn lanes to calculate available left-turn sight distance and evaluate the improvement effect related to offset value between opposing parallel left-turn lanes.
Joshua’s model was based on the sight-distance model of the 1994 AASHTO manual, but the section on Intersection Sight Distance has been completely revised in the 2001 AASHTO manual, which is based on a time gap acceptance methodology. McCoy’s work paid main attention to departure positions of left-turn vehicles, in which the left-turn vehicles are permitted to enter the intersection before they execute the turn maneuver, but for signalized intersections, moving the vehicle out beyond the stop bar of the left-turn lane is considered as an illegal driving behavior. From the perspective of intersection design, traffic engineers are supposed to ensure that left-turn drivers can have sufficient sight distance to correctly judge gap sizes in the opposing through traffic within left-turn lanes. In addition, related literature about geometric models specifically developed to analyze and evaluate the effect of visibility improvement at tapered offset left-turn lanes has not been found.

Furthermore, the sight distance problem concerned by AASHTO criteria is provided only for linear-approach intersections. However, the presence of a horizontal curve on the intersection approaches represents an additional risk for left-turners beyond that of a typical intersection with linear approaches. Especially, traffic environments combined with a horizontal curve, a signalized intersection and high traffic volumes contribute to a relatively complex situation for the driver. The high accident rate at intersections indicates that the existing sight distance around a horizontal curve may be inadequate.

In previous studies, more attentions was paid to sight obstruction on the inside of curves, which can be objects such as cut slopes, walls, buildings, bridge piers, and longitudinal
barriers. Very few studies were found to be related to opposing left-turn vehicle as sight
obstruction to the left-turn vehicle at an intersection located on a horizontal curve,
although the several researchers developed geometric models to calculate available left-
turn sight distance for such a case at linear-approach intersections (Joshua, 1992; McCoy,
1992). The curve scenario may have an increased probability of sight blockage,
especially for the driver making left-turn toward outside of the curve into the minor road
approach. The likelihood and severity of this problem will increase with the sharpness of
the curve. On the other hand, for left-turners toward the inside of the curve, the sharpness
of the curve may mitigate the vision problem and even contribute to unrestricted sight
distance, since the left-turners benefit from a left-turn lane offset toward the coming
traffic. However, at a linear-approach intersection, the available sight distances for both
opposite left-turners are same, which are related the median width of major approaches.
Therefore, if an intersection is located on a curve major road, the left-turn sight distance
problem maybe becomes more complex, which is needed to be evaluated by developing
special geometric models.

2.5 Driving Simulator Issues

2.5.1 Benefits and limitations of simulator research

With the progress of computer science and electronic engineering in recent years, driving
simulators used for training and research are being rapidly developed. A modern driving
simulator can give a driver on board impression that he/she drives an actual vehicle by predicting vehicle motion caused by driver input and feeding back corresponding visual, motion, audio and proprioceptive cues to the driver. A driving simulator is a virtual reality tool which enables researchers to conduct multi-disciplinary investigations and analyses on a wide range of issues associated with traffic safety, highway engineering, Intelligent Transportation System (ITS), human factors, and motor vehicle product development. The use of a modern advanced driving simulator for human factors research has many advantages over similar real world or on-road driving research. These advantages include experimental control, efficiency, expense, safety, and ease of data collection. Especially, a simulation experiment has the ability to reproduce dangerous driving conditions and situations in a safe and controlled environment to test driver behaviors. In addition, many researches (Alicandri, 1986 and Stuart, 2002) indicated that simulator measures are valid for sign detection and recognition distances, speed, accelerator position changes and steering wheel reversals, because of a high correspondence between real world and simulator data sets.

However, there are also some limitations of simulation research. An important limitation of simulator research is simulator sickness (also euphemistically known as simulator discomfort). In a driving simulator research (Yan, 2003), it is reported that due to driving simulator sickness, about 10% of the younger male subjects and 20% of the younger female subjects were unable to complete the experiment and about 10% of the older male subjects and 40% of the older female subjects could not complete the experiments. Simulator sickness is not identical to motion sickness, although it is sometimes described
as such (e.g. Nilsson, 1993). Motion is essential for motion sickness, but simulator sickness can occur without motion (Kolasinski, et al., 1995). It is related to driving task such as sharp turn or stop, experiment time, and complexity of visual elements. In the proposed simulator experiment, only what can be done to weaken Simulator sickness is to reduce the experiment time.

2.5.2 UCF driving simulator

The UCF driving simulator housed in the Center for Advanced Transportation Systems Simulation (CATSS) is an I-Sim Mark-II system with a high driving fidelity and immense virtual environments. The simulator cab it is a Saturn model that has an automatic transmission, an air condition, a left back view mirror and a center back view mirror inside the cab, as shown as Figure 2-2. The simulator is mounted on a motion base capable of operation with 6 degrees of freedom. It includes 5 channels (1 forward, 2 side views and 2 rear view mirrors) of image generation, an audio and vibration system, steering wheel feedback, operator/instructor console with graphical user interface, sophisticated vehicle dynamics models for different vehicle classes, a 3-dimensional road surface model, visual database with rural, suburban and freeway roads plus an assortment of buildings and operational traffic control devices, and a scenario development tool for creating real world driving conditions. The output data include detailed events pertaining to every car’s steering wheel, accelerator, brake, every car’s speed and coordinates, and a time stamp. The sampling frequency is 60Hz.
The simulator session is controlled from an operator's console in an adjacent control room. Scenarios are created with the scenario editing software on a screen showing the locations of roads, buildings, traffic control devices, pedestrians, etc. The five video channels are monitored on computer screens in the control room. A road map of the database is viewable on the operator's console showing movement of the simulator vehicle and other vehicles which are present (Harold, 2003).

The new simulator is capable of supporting research in driving simulation, driver training, human factors and traffic engineering.
CHAPTER 3. ANALYSES OF RED-LIGHT RUNNING CRASH
USING FLORIDA TRAFFIC CRASH DATABASE

To examine the overall characteristics of red-light running crashes, this study applied 1999-2001 Florida traffic crash data to investigate the accident propensity of three aspects of risk factors related to traffic environments, driver characteristics, and vehicle types. A quasi-induced exposure concept and the multiple logistic regression technique are used to perform this analysis. The results showed that seven environmental factors (Number of lanes, Divided/undivided highway, Accident time, Weather, Highway character, Day of week, and Urban/rural), four factors related to driver characteristics (Driver age, Alcohol/drug use, Physical defect, and Driver residence), and Type of vehicle are significantly associated with the risk of red-light running accidents. Furthermore, the logistic regression technique confirmed significant interaction effects between risk factors including: Number of lanes & Urban/rural, Day of week & Driver age, Driver age & Gender, Alcohol/drug use & Gender, and Type of vehicle & Gender.

3.1 Methodology

3.1.1 Quasi-induced exposure technique
To test crash propensity and explore the traffic crash database, quasi-induced exposure technique (Carr, 1970; Haight, 1973) is becoming widely used in traffic safety research. Stamatiadis and Deacon (1995) developed the term, relative crash involvement ratio (RAIR), as the measure of crash causing propensity used in the quasi-induced exposure analysis. It is equal to the ratio of the percentage of a specific subgroup in at-fault drivers to the percentage of the same subgroup in not-at-fault drivers. The at-fault drivers are those who were mostly responsible for the crash occurrence and the not-at-fault drivers are those victims in the crashes. The key assumption is that the distribution of not-at-fault drivers closely represents the distribution of all drivers exposed to crash hazards.

Previous studies had successfully applied the quasi-induced exposure method to analyze traffic crash risks of drivers and vehicles under a given set of environmental conditions (Stamatiadis, 1995; Aldridge et al., 1999; Hing, et al., 2003). However, few of them focused on the investigation of non-driver-related (environmental) factors as exclusive main effects on the traffic safety. To introduce the environmental factors into statistical model and test their exclusive main effects on crashes, this research extended the application of the quasi-induced exposure. In this study, firstly, two-vehicle crashes occurring at signalized intersections are identified, which are composed by red-light running crashes and non-red-light running crashes. Then, drivers/vehicles who disregarded red signal and result in red-light running crashes and their corresponding environment information are categorized into the at-fault group; drivers/vehicles who had no improper driving action but were involved in non-red-light running crashes and their corresponding environmental conditions are categorized into the not-at-fault group.
To increase the comparability between at-fault group and exposure group, driving behaviors are similar in both groups: going-through the signalized intersections when the crashes were happening. The purpose of eliminating other red-light running behaviors such as left-turn, right-turn or U-turn and those crashes involving more than two vehicles is intended to simplify the assignment of driver culpability. For better understanding the principle of the data classification, Figure 3-1 illustrated the relationship between at-fault and not-at-fault groups.

![Diagram of at-fault and not-at-fault groups](image)

Figure 3-1: At-fault and not-at-fault groups used for quasi-induced exposure analysis red-light running crashes

Based on the above categorization, three types of relative crash involvement ratios to test the main effects of driver, vehicle, and environment factors related to red-light running crashes are calculated. The extended assumption here is that the distributions of environmental factors in non-red-light running crashes may represent the distributions of
environmental factors confronted by all vehicles/drivers. Using the RAIR formula developed by Stamatiadis and Deacon (1995), they are defined as Equation 3-1.

$$RAIR_{i,j} = \frac{\sum D1_i}{\sum D2_i} \text{ or } \frac{\sum V1_i}{\sum V2_i} \text{ or } \frac{\sum E1_i}{\sum E2_i}$$

(3-1)

where, $RAIR_i$ is the relative crash involvement ratio for type $i$ drivers/vehicles/environments. $D1_i$ is the number of at-fault drivers of driver type $i$, $D2_i$ is the number of not-at-fault drivers of driver type $i$, $V1_i$ is the number of at-fault vehicles type $i$, $V2_i$ is the number of not-at-fault vehicles type $i$, $E1_i$ is the number of red-light running crashes involving environment type $i$, and $E2_i$ is the number of non-red-light running crashes involving environment type $i$.

Furthermore, to test the interaction between type $i$ drivers/vehicles/environments and type $j$ drivers/vehicles/environments, the RIAR can be defined as Equation 3-2.

$$RAIR_{i,j} = \frac{\sum \sum N1_{i,j}}{\sum \sum N2_{i,j}}$$

(3-2)

where, $RAIR_{i,j}$ is the relative crash involvement ratio type $i$ and type $j$ drivers/vehicles/environments. $N1_{i,j}$ is the number of at-fault drivers, vehicles, or the
related environments of type i and j, \( N_{2_{i,j}} \) is the number of not-at-fault drivers, vehicles, or the related environments of type i and j.

3.1.2 Statistical modeling

Previous studies had appropriately applied logistic regression analysis to test significance of traffic crash risk factors based on techniques of induced exposure (Stamatiadis and Deacon, 1995; Hing et al., 2003). Logistic regression belongs to the group of regression methods for describing the relationship between explanatory variables and a discrete response variable. A binary Logistic regression is proper to be used when the dependent is a dichotomous (an event happened or not) and can be applied to test association between a dependent variable and the related potential factors, to rank the relative importance of independents, and to assess interaction effects. Binary logistic regression is used in this study since the dependent variable \( Y \) (crash classification) can only take on two values: \( Y=1 \) for red-light running crashes, and \( Y=0 \) for non-red-light running crashes. The probability that a red-light running crash will occur or not is modeled as logistic distribution in Equation 3-3:

\[
\pi(x) = \frac{e^{g(x)}}{1 + e^{g(x)}} \tag{3-3}
\]

The Logit of the multiple logistic regression model (Link Function) is given by Equation 3-4:
\[ g(x) = \ln \left( \frac{\pi(x)}{1 - \pi(x)} \right) = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_3 + \ldots + \beta_n x_n \]  

(3-4)

where, \( \pi(x) \) is conditional probability of a red-light running crash, which is equal to the number of red-light running crashes divided by the total number of crashes. \( x_n \) is independent variables (driver/vehicle/environment factors). The independent variables can be either categorical or continuous, or a mixture of both. Both main effects and interactions can generally be accommodated. \( \beta_n \) is model coefficient, which directly determines odds ratio involved in the red-light running crash. The odds of an event are defined as the probability of the outcome event occurring divided by the probability of the event not occurring. The odds ratio that is equal to \( \exp(\beta_n) \) tells the relative amount by which the odds of the outcome increase (O.R. greater than 1.0) or decrease (O.R. less than 1.0) when the value of the predictor value is increased by 1.0 units (David and Lemeshow, 1989). Especially for dummy independent variables, the odds ratios represent the crash risk comparison among different levels of drivers/vehicles/environments.

The previous studies (Stamatiadis and Deacon, 1995; Hing et al., 2003) had clearly expressed the relationship between logistic regression and RAIR in the quasi-induced exposure analysis. In fact, for a specific type of drivers/vehicles/environments, the odds generated from the logistic regression model are equivalent to the corresponding RAIRs, and the odds ratios are equivalent to the comparisons among those RAIRs. Furthermore, the P-values generated from the logistic regression qualitatively indicate the statistical importance of those RAIR comparisons between different types of
drivers/vehicles/environments. In this study, the SAS program procedure, LOGISTIC, was used for model development and hypothesis testing based on 0.05 significance level.

3.1.3 Database

The crash data for the 1999-2001 periods were obtained from the Florida Department of Highway Safety and Motor Vehicles (DHSMV). The DHSMV data constitute a relational database that includes seven files. Each file deals with a specific aspect of traffic crashes. Files may be linked as needed to combine the information contained in each file. The files used in the analysis presented here were the event (containing the characteristics and environment of the crash), drivers (containing the drivers’ characteristics), and vehicles (information about the vehicles’ characteristics and vehicles actions in the traffic crash) files. From those three files, the independent variables used to examine the association with the red-light running crashes include Number of lanes, Divided/undivided highway, Location type, Crash time, Weather, Highway character, Day of week, Urban/rural, Posted speed, Driver age, Alcohol/drug use, Physical defect, Driver residence, Gender, and Type of vehicle. For simplicity and ease of interpretation of the results, all those variables are classified categorical variables.

According to the crash classification, the modeling dataset identified from the 3-year database includes total 16,310 red-light running crashes and 41,109 no-red-light running crashes, the amounts of which both are very similar in each year (see Table 3-1). Furthermore, calibrating the logistic regression models for 1999, 2000, 2001 data
separately, the results were not significantly different and there is no strong evidence that
the data did not maintain consistent trend over the three years.

<table>
<thead>
<tr>
<th>Period</th>
<th>Caused by red-light running</th>
<th>Involved by Not-at-fault drivers</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1999</td>
<td>5491</td>
<td>13356</td>
<td>18847</td>
</tr>
<tr>
<td>2000</td>
<td>5493</td>
<td>13691</td>
<td>19184</td>
</tr>
<tr>
<td>2001</td>
<td>5326</td>
<td>14062</td>
<td>19388</td>
</tr>
<tr>
<td>Total</td>
<td>16310</td>
<td>41109</td>
<td>57419</td>
</tr>
</tbody>
</table>

3.2 Results and Analyses

Using the multivariate logistic regression analysis of the main effect model, except for
Divided/undivided highway, Location type, and Driver gender, all the other factors show
significant association with red-light running accidents based on the hypothesis test of
0.05 significance level. Table 3-2 lists the RAIRs and odds ratios properly adjusting other
factors for significant independent variables, where the lowest levels of independent
variables are considered as the default levels. Although driver gender does not show
significance (P=0.6636), it is also included in the current model since it is always
important to the safety research and possibly is a confounded variable that has an
interaction effect with other ones. The following sections document the interpretation of
the regression results for those variables classified by driver characteristics, accidents
environment, and vehicle characteristics.
Table 3-2: Relative Crash Involvement Ratios and Odds Ratios for Significant Independent Variables

<table>
<thead>
<tr>
<th>Factors</th>
<th>Crashes caused by At-fault drivers (%)</th>
<th>Crashes involved by Not-at-fault drivers (%)</th>
<th>RAIR</th>
<th>Adjusted odds ratio</th>
<th>P-value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Number of lanes</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>2-lane</td>
<td>14.32</td>
<td>14.84</td>
<td>0.96</td>
<td>Default</td>
<td></td>
</tr>
<tr>
<td>4-lane</td>
<td>42.19</td>
<td>44.21</td>
<td>0.95</td>
<td>1.01</td>
<td>0.7386</td>
</tr>
<tr>
<td>6-lane</td>
<td>26.71</td>
<td>25.26</td>
<td>1.06</td>
<td>1.17</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Others</td>
<td>16.78</td>
<td>15.68</td>
<td>1.07</td>
<td>1.14</td>
<td>0.0007</td>
</tr>
<tr>
<td><strong>Crash time</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Daytime</td>
<td>76.21</td>
<td>66.32</td>
<td>1.15</td>
<td>Default</td>
<td></td>
</tr>
<tr>
<td>Night</td>
<td>23.79</td>
<td>33.68</td>
<td>0.71</td>
<td>0.57</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td><strong>Weather</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Clear</td>
<td>76.77</td>
<td>76.55</td>
<td>1.00</td>
<td>Default</td>
<td></td>
</tr>
<tr>
<td>Cloudy</td>
<td>17.05</td>
<td>15.73</td>
<td>1.08</td>
<td>1.06</td>
<td>0.0243</td>
</tr>
<tr>
<td>Rain</td>
<td>6.19</td>
<td>7.73</td>
<td>0.80</td>
<td>0.83</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td><strong>Highway character</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.0378</td>
</tr>
<tr>
<td>Straight-level</td>
<td>93.4</td>
<td>94.11</td>
<td>0.99</td>
<td>Default</td>
<td></td>
</tr>
<tr>
<td>Straight-Up/downgrade</td>
<td>5.27</td>
<td>4.61</td>
<td>1.14</td>
<td>1.14</td>
<td>0.0044</td>
</tr>
<tr>
<td>Curve-level</td>
<td>0.98</td>
<td>0.94</td>
<td>1.04</td>
<td>1.06</td>
<td>0.606</td>
</tr>
<tr>
<td>Curve-Up/downgrade</td>
<td>0.35</td>
<td>0.34</td>
<td>1.03</td>
<td>0.94</td>
<td>0.7428</td>
</tr>
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<td><strong>Day of week</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.0002</td>
</tr>
<tr>
<td>Weekday</td>
<td>71.1</td>
<td>72.39</td>
<td>0.98</td>
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<td></td>
</tr>
<tr>
<td>Weekend</td>
<td>28.9</td>
<td>27.61</td>
<td>1.05</td>
<td>1.09</td>
<td>0.0002</td>
</tr>
<tr>
<td><strong>Urban/rural</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Rural</td>
<td>31.11</td>
<td>36.49</td>
<td>0.85</td>
<td>Default</td>
<td></td>
</tr>
<tr>
<td>Urban</td>
<td>68.89</td>
<td>63.51</td>
<td>1.08</td>
<td>1.25</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td><strong>Posted speed</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>25 mph</td>
<td>2.26</td>
<td>2.41</td>
<td>0.94</td>
<td>Default</td>
<td></td>
</tr>
<tr>
<td>30 mph</td>
<td>18.46</td>
<td>15.17</td>
<td>1.22</td>
<td>1.33</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>35 mph</td>
<td>24.1</td>
<td>22.86</td>
<td>1.05</td>
<td>1.16</td>
<td>0.0415</td>
</tr>
<tr>
<td>40 mph</td>
<td>18.84</td>
<td>21.52</td>
<td>0.88</td>
<td>0.98</td>
<td>0.821</td>
</tr>
<tr>
<td>45 mph</td>
<td>31.22</td>
<td>32.85</td>
<td>0.95</td>
<td>1.03</td>
<td>0.6658</td>
</tr>
<tr>
<td>50 mph</td>
<td>2.92</td>
<td>2.89</td>
<td>1.01</td>
<td>1.10</td>
<td>0.308</td>
</tr>
<tr>
<td>55 mph</td>
<td>2.2</td>
<td>2.31</td>
<td>0.95</td>
<td>1.06</td>
<td>0.5557</td>
</tr>
<tr>
<td><strong>Driver age</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>&lt;26</td>
<td>31.65</td>
<td>27</td>
<td>1.17</td>
<td>Default</td>
<td></td>
</tr>
<tr>
<td>26-35</td>
<td>20.45</td>
<td>23.57</td>
<td>0.87</td>
<td>0.69</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>36-45</td>
<td>16.96</td>
<td>21.05</td>
<td>0.81</td>
<td>0.62</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>46-55</td>
<td>10.96</td>
<td>13.41</td>
<td>0.82</td>
<td>0.64</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>56-65</td>
<td>7.55</td>
<td>7.86</td>
<td>0.96</td>
<td>0.75</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>66-75</td>
<td>6.46</td>
<td>4.62</td>
<td>1.40</td>
<td>1.08</td>
<td>0.1144</td>
</tr>
<tr>
<td>Alcohol/drug use</td>
<td>&gt;75</td>
<td>5.95</td>
<td>2.5</td>
<td>2.38</td>
<td>1.81</td>
</tr>
<tr>
<td>--------------------------------------</td>
<td>-----</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
</tr>
<tr>
<td>1-No</td>
<td></td>
<td>95.95</td>
<td>98.9</td>
<td>0.97</td>
<td>Default</td>
</tr>
<tr>
<td>2-Alc-under influence</td>
<td></td>
<td>1.26</td>
<td>0.25</td>
<td>5.04</td>
<td>7.32</td>
</tr>
<tr>
<td>3-Drug-under influence</td>
<td></td>
<td>0.06</td>
<td>0.01</td>
<td>6.00</td>
<td>4.78</td>
</tr>
<tr>
<td>4-Alc&amp;Drug-under influence</td>
<td></td>
<td>0.15</td>
<td>0.05</td>
<td>3.00</td>
<td>3.52</td>
</tr>
<tr>
<td>5-Had been drinking</td>
<td></td>
<td>1.81</td>
<td>0.68</td>
<td>2.66</td>
<td>3.56</td>
</tr>
<tr>
<td>6-Pending BAC test results</td>
<td></td>
<td>0.76</td>
<td>0.11</td>
<td>6.91</td>
<td>9.67</td>
</tr>
<tr>
<td>Physical defect</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-No defect</td>
<td></td>
<td>98.43</td>
<td>99.33</td>
<td>0.99</td>
<td>Default</td>
</tr>
<tr>
<td>2-Eyesight defect</td>
<td></td>
<td>0.78</td>
<td>0.45</td>
<td>1.73</td>
<td>1.41</td>
</tr>
<tr>
<td>3-Fatigue/asleep</td>
<td></td>
<td>0.14</td>
<td>0.01</td>
<td>14.00</td>
<td>27.15</td>
</tr>
<tr>
<td>4-Hearing Defect</td>
<td></td>
<td>0.1</td>
<td>0.04</td>
<td>2.50</td>
<td>1.37</td>
</tr>
<tr>
<td>5-Illness</td>
<td></td>
<td>0.08</td>
<td>0.01</td>
<td>8.00</td>
<td>9.48</td>
</tr>
<tr>
<td>6-Seizure/Epilepsy/Blackout</td>
<td></td>
<td>0.15</td>
<td>0.01</td>
<td>15.00</td>
<td>16.42</td>
</tr>
<tr>
<td>7-Other Physical defect</td>
<td></td>
<td>0.33</td>
<td>0.15</td>
<td>2.20</td>
<td>1.84</td>
</tr>
<tr>
<td>Residence code</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-County of crash</td>
<td></td>
<td>83.06</td>
<td>87.02</td>
<td>0.95</td>
<td>Default</td>
</tr>
<tr>
<td>2-Elsewhere in state</td>
<td></td>
<td>12.49</td>
<td>9.96</td>
<td>1.25</td>
<td>1.36</td>
</tr>
<tr>
<td>3-Non-resident of state</td>
<td></td>
<td>3.65</td>
<td>2.55</td>
<td>1.43</td>
<td>1.54</td>
</tr>
<tr>
<td>4-Foreign</td>
<td></td>
<td>0.8</td>
<td>0.47</td>
<td>1.70</td>
<td>1.82</td>
</tr>
<tr>
<td>Gender</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Male</td>
<td></td>
<td>59.03</td>
<td>59.43</td>
<td>0.99</td>
<td>Default</td>
</tr>
<tr>
<td>Female</td>
<td></td>
<td>40.97</td>
<td>40.57</td>
<td>1.01</td>
<td>1.04</td>
</tr>
<tr>
<td>Type of vehicle</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1-Passenger car</td>
<td></td>
<td>74.41</td>
<td>74.11</td>
<td>1.00</td>
<td>Default</td>
</tr>
<tr>
<td>2-Van</td>
<td></td>
<td>8.02</td>
<td>7.6</td>
<td>1.06</td>
<td>1.14</td>
</tr>
<tr>
<td>3-Light truck</td>
<td></td>
<td>15.02</td>
<td>14.93</td>
<td>1.01</td>
<td>1.08</td>
</tr>
<tr>
<td>4-Large vehicle</td>
<td></td>
<td>2.55</td>
<td>3.36</td>
<td>0.76</td>
<td>0.81</td>
</tr>
</tbody>
</table>

### 3.2.1 Environment factors

Seven environmental factors including Number of lanes, Crash time, Weather, Highway character, Day of week, Urban/rural, and speed limit show significant association with the risk of red-light running crashes. Figure 3-2 illustrates comparisons of relative crash involvement ratios between different levels of environmental factors.
Figure 3-2: Relative crash involvement ratios by environmental factors
The number of lanes at the crash site as originally recorded by the reporting officer includes both sides of the median where applicable. Since most of the red-light running crashes happened inside intersections, it is very possible that some police officers did not accurately report the number of lanes on the appropriate crash form. To test the crash trend and minimize the data error, only 2, 4, and 6-lane highways are considered in this study. The 4-lane highway constitutes the most common type of red-light running crashes (42.19%), followed by 6 lane highway (26.71%). For RAIRs, the 6-lane highway has the largest crash propensity (1.06) and those for 2-lane and 4-lane highways are lower and similar (0.96 and 0.95). The risk of crash involvement for 6-lane highways could be around 17 percent higher than 2-lane and 4-lane highways at a 0.0001 level of significance. The crash analysis is consistent with the results of Porter and England (2000): red-light running rates were higher for intersections with a larger number of lanes and larger volumes.

The results show that most red-light running crashes occurred at daytime (76.21%), versus 23.79% at night. An interesting finding is that the relative crash involvement ratio (1.15) for daytime is also higher than that (0.57) for night, as shown in Figure 3-2. Based on the model, the crash risk for night could be 43% lower than that for daytime at a 0.0001 level of significance. The presumable reasons are that the traffic volume at daytime is higher than that at night; the morning peak and afternoon peak may affect driving attitude and contribute to the red-light running behavior; and actually, the visibility of signal lights at night may be better than during the day, due to the greater contrast between traffic light and dark background.
In Florida, drivers frequently drive in cloudy and rainy weather conditions and the corresponding red-light running crash rates (17.05% and 6.19%) are very high. The analysis shows that the RAIR for cloudy weather is slightly higher than for clear weather, but for rainy weather it is lower than for clear weather. The risk of red-light running crashes for cloudy weather is 6 percent higher than clear weather (P=0.0243), presumably because of worse visibility of signal light and driver’s more aggressive attitude in anticipation of rain. However, the risk for rainy weather is 17 percent lower than for clear weather (P<0.0001) presumably because drivers slow down their travel speed and act more cautiously when it is raining. Moreover, drivers with relatively weak driving ability, for example older drivers or new drivers, are very possible to avoid adverse driving environments.

For the highway characteristics, the most crashes happened at intersections located on straight-level (S-L) highways (93.4%), followed by straight-upgrade/downgrade (S-U/D) highways (5.27%). Based on the model, straight-upgrade/downgrade can contribute to the red-light running crashes and its risk involving crashes could be 14% higher than Straight-level highways at a 0.0044 level of significance. The presumable reason is that the grade of an intersection approach may significantly influence the time and distance needed for a motorist to stop a vehicle at an intersection. If approaching the intersection on a downhill grade, motorists may not account for vehicle mass and momentum which will require longer stopping time (FHWA & NHTSA, 2003). For curve-level (C-L) and
curve-upgrade/downgrade (C-U/D) highways, their RAIRs are also slightly higher than straight-level highways, but the difference is not statistically significant.

Although 71.1% red-light running crashes occurred on a weekday, the crash risk during the weekend is 9 percent higher than weekday at a 0.0002 level of significance. The analysis also shows that red-light running is particularly relevant to urban crashes (69.89%) and the crash risk in urban area could be 25 percent higher than rural area, presumably because urban areas are more dominated by signalized intersections and have higher traffic volume than rural areas. Retting et al. (1995) reported that 56% of urban crashes occurred at intersections; and running traffic controls accounted for 22% of urban crashes studied. Additionally, the most crashes happened at intersections with the 45 mph speed limit (31.22%), followed by the 35 mph (24.1%), and the least crashes happened for the 55 mph (2.2%). For the RAIRs, the crashes tend to be over-involved in the 30 and 35 mph speed limits, as shown in Figure 3-2. The logistic regression model indicated that the odds ratios of involving red-light running crashes for 30 mph and 35 mph speed limits could be around 22 and 5 percent higher than that for the 25 mph speed limit.

3.2.2 Driver characteristics

Four factors related to driver characteristics including Driver age, Alcohol/drug use, Physical defect, and Driver residence shows significant association with the risk of red-light running crashes. Figure 3-3 illustrates comparisons of relative crash involvement ratios between different levels of those factors.
The ten-year interval was chosen to group driver age. The seven driver age groups included younger than 26 years, 26-35 years, 36-45 years, 46-55 years, 56-65 years, 66-75 years, and older than 75 years. Based on the driving population distribution, both crash frequencies of red-light running drivers and not-at-fault drivers decrease greatly as the driver age increases, as shown in Table 3-2. Comparing relative crash involvement ratios, the crash ratios as a function of driver age is illustrated in Figure 3-3. The graph for driver age shows a typical U-shape pattern, which indicates that middle age groups have lower risk and the younger and older groups have relatively higher risk. There is a significant trend of increasing crash involvement as the driver ages increase over 55. The oldest group, older than 75 years of age, presented the highest risk involving red-light
running crashes, which is equal to 1.81 times of the youngest groups (P<0.0001), presumably because of age-related deterioration of their physical and cognitive abilities. The youngest groups, younger than 26 years of age, presented relatively higher risk, which RAIR (1.17) is lower than the second older groups (1.40), but the difference is not statistically significant. The middle age groups, 36-55 years of age, presented the lowest risk, whose odds ratios involving red-light running crashes are between 60-70 percent of that for the youngest groups (P<0.0001). A nationwide telephone survey revealed that there is a descending trend of red-light running violation as the age increases and older drivers were only 0.30 times as likely as the youngest age group to report recent red-light running, but drivers 26–35 years of age were not significantly different from the youngest group (Porter and Berry, 2001). Although older drivers are less likely to have intentional violation behaviors of red-light running or speeding, generally, older drivers react more slowly to events that are not expected and take significantly longer time to make decisions than younger drivers. Therefore, whenever older drivers are running red-light, they are more likely to be involved in a crash.

Another important crash factor is driving under the influence of alcohol and/or illegal drugs. Alcohol reduces alertness, interferes with judgment and impairs vision. Most drugs that affect the central nervous system may have the potential to impair driving ability. There are six levels of potential alcohol/drug use recorded in the Florida crash database. As shown in Table 3-2, total at-fault drivers related to alcohol/drug use constitute 4.05 percent of red-light running crashes. According to the RAIRs, drivers under influence of alcohol, drug, or both are substantially over-involved in the crashes (see Figure 3-3) and
their odds ratios of crash risk could be 7.32, 4.78, and 3.52 times higher than normal drivers, with P-values of 0.0001, 0.0127, and 0.0001. Even drivers who had been drinking also show significantly higher risk than non-drinking drivers (OR=3.56). Drivers with Pending BAC test results show the highest relative crashes involvement rate (6.91) and odds ratio (9.67). It is speculated that those who need BAC test results are the most serious drunk drivers.

The crash database particularly recorded possible physical defects of drivers involved in crashes, which are coded as 1-No defect, 2-eyesight defective, 3-fatigue/sleep, 4-hearing defect, 5-illness, 6- Seizure/Epilepsy/Blackout, and 7-Other Physical defect. It is not surprising that drivers with physical defects of fatigue/sleep and Seizure/Epilepsy/Blackout present the highest risk because drivers with such defects will lose basic ability to control vehicle so as to result in red-light running at intersections. Especially for fatigue/sleep drivers, their odds ratio is estimated to be 27.15 times more likely involved in a red-light running crashes than the normal drivers, as shown in Table 3-2. On the whole, except for hearing defect (P=0.4339), all the recorded physical defects are significant risk factors contributing to red-light running crashes. Of those, drivers with eyesight defective are 41 percent more likely to be involved in crashes than those without, and compared to other drivers with physical defects, eyesight defects is the most common type and constitutes 0.78 percents of red-light running crashes. However, as not-at-fault drivers, those with physical defects (as well as those with alcohol/drug use) performed worse to avoid crashes than normal ones. There is a higher chance to be underestimating their risk involving crashes based on the induced exposure technique.
Moreover, the data sample sizes in some levels of variables physical defect and alcohol/drug use are very small, so that the point estimates of their odds ratios are less significant due to the large standard errors.

The crash database also provided driver classification by residence. The level 1, 2 3, or 4 stand for the residence of drivers living in the local county, elsewhere in the state of Florida, other state, or other country. Normally, local drivers can benefit from their driving experiences to the familiar traffic environments so as to avoid the adverse traffic conditions. Due to the large population exposure, the local driver classification had the highest involvement frequency and captured 83.06 percent of the crashes, as shown in Table 3-2. However, their relative crash rates are apparently lower than those non-local drivers as shown in Figure 3. It appears there is a clear trend in the figure which indicates that as the degree of drivers’ familiarity with the driving environment decreases, they are more likely to be involved in red-light running crashes. This is especially true for foreign drivers, since their risk rate is 82 percent higher than the county residents.

3.2.3 Vehicle characteristics

Vehicle type was also found to significantly affect red-light running crash at a 0.0001 level of significance. There is a total of 13 types of vehicles classified by crash vehicles in the database. Four types of vehicles are focused on in the study including automobile, passenger van, pickup/light truck, and large size vehicle. Large size vehicle is combined with the medium truck, heavy truck, truck-tractor, motor home, and bus, since the sample
size for each one is very small. The other vehicles such as motorcycle, moped, terrain vehicle, and train are excluded from the study. As shown in Table 3-2, there are 74.41% automobiles, 8.02% passenger vans, 15.02% light trucks, and 2.55% large size vehicles resulting in red-light running crashes. As shown in Figure 3-4, the RAIRs for automobile, passenger van, and light truck are very similar, but those large size vehicle drivers present the lowest crash risk. Based on the logistic regression, the odds ratio of large size vehicle could be 19 percent lower than automobiles, while the odds ratios of passenger vans and light trucks are slightly higher than automobile. The result is not consistent with previously published red-light running violation study. Al-Omari and Al-Masaeid (2003) indicated that truck drivers had the highest violation rate, followed by small vehicles and then buses. Another previous study also reported that vehicles that carry heavy loads require additional time to slow and stop when a traffic signal changes to yellow, and drivers of vehicles with heavy loads may forget or disregard the effect of the loads on stopping distances, so as to result in red-light running (FHWA & NHTSA, 2003). However, large size vehicles may not significantly contribute to red-light running crashes. Normally, large size vehicles tend to void traffic peak, their operation speeds are relatively lower than small cars, and their drivers, who mostly are professional, are rarely involved in alcohol/drug use and extreme aggressive driving attitude. Moreover, since large size vehicles are more noticeable on the road, even if running red-light, other conflicting vehicle drivers may more easily detect them so that crashes are possibly avoided.
3.2.4 Interaction effect

After developing the main effect model and confirming the above risk factors, the next multivariate logistic analysis was aimed at exploring significant interactions between those risk factors. It was found that there are five interaction factors associated with red-light running crashes including: Crash time & Highway character (P=0.0003), Number of lanes & Urban/rural (P=0.0017), Weather condition & Driver age (P=0.0422), Driver age & Gender (P<0.0001), Alcohol/drug use & Gender (P=0.0107), and Type of vehicle & Gender (P=0.0194). Figure 3-5 illustrates the effects of interaction factors by relative crash involvement ratios.

Figure 3-4: Relative crash involvement ratios by vehicle characteristics
Figure 3-5: Effects of interaction factors by relative accident involvement ratios.

The results show that during daytime, red-light running crashes are more likely to occur at intersections with straight-Up/downgrade (S-U/D), curve-level (C-L), and curve-upgrade/downgrade (C-U/D) approaches than those with typical straight-level (S-L) approaches. On the contrary, during nighttime, the crash involvement for straight-level (S-L) intersections is relatively higher than the others. Generally, when an intersection
has up/downgrade or is located on a horizontal curve, the stopping sight distance could be restricted by potential sight obstructions around the intersection. If drivers cannot detect signal changing and potential conflicting vehicles, they have higher chance to run red-light and result in a crash. However, as mentioned before, signal heads would be more visible to drivers at night, and drivers may be more cautious at intersections with complex geometric configurations during night time. Based on the interaction analysis between environment factors, it is also found that for 2-lane highways, the crash risk in rural area is lower than that in urban area, but for 6-lane highways, the crash risk in rural area is higher than that in urban area.

The driver age effect is influenced by weather conditions and driver gender respectively. Younger drivers have a larger crash propensity for rainy weather, but on the contrary, older drivers (> 55 years) have a larger crash propensity in clear weather. For middle age groups, there is no difference found in weather conditions. Although the overall gender difference may be insignificant, the gender effect is complex and intimately related to driver age. For middle age groups (26-45 years), gender has little influence on crash propensity. Nevertheless, younger male drivers have a larger crash propensity than younger females while older male drivers have a smaller crash propensity than older females. Moreover, the gender difference in the older groups is obviously increasing with increment of driver ages, which suggests that there is more reduction of driving abilities for the old female drivers than the older male drivers. The analysis is very close to the conclusion drawn by Stamatiadis and Deacon (1995). They explained that, younger female drivers perform better, presumably because of risk-taking and attitudinal factors
and perhaps as well because of when and where they drive; older males make better drivers than older females, presumably because they have more lifetime driving experience and began accumulating it at an earlier age.

Moreover, driver gender shows interaction effects with alcohol/drug use and type of vehicles respectively. For the alcohol use drivers (1-under influence of alcohol or 5-had been drinking), the crash risks of males are similar to females. However, when the drivers were involved in drug use, the crash risks for males were far larger than females, as illustrated in Figure 3-5. For vehicle types, male drivers have larger crash propensities for light trucks and large size vehicles; for passenger vans and automobiles, there is no significant gender difference.

3.3 Conclusions and Discussions

Using 1999-2001 Florida traffic crash databases, this study examined the overall characteristics of red-light running crashes based on Quasi-induced exposure analysis. Through identifying the red-light running crashes by at-fault drivers and the non-red-light running crashes by not-at-fault drivers at signalized intersections, the main effect factors related to traffic environments, driver characteristics, and vehicle types were directly introduced into multiple logistic regression models. The models examined the crash propensities of those factors, as well as their interaction effects.
The analysis showed that the risk of red-light running crashes for 6-lane highways is higher than 2-lane and 4-lane highways. The relative crash involvement ratio for night is apparently lower than daytime and the crash ratio for weekend is higher than weekdays. Compared to clear weather, crashes more likely to occur under cloudy weather and are less involved in rainy weather. Geometric configuration of the intersection can also influence the crash occurrence. Especially during daytime, complex intersection geometric conditions such as up/downgrade and horizontal curve may contribute to the higher crash involvement rate. Moreover, red-light running crashes are more relevant to urban areas and most likely happen with 30 and 35 mph speed limits.

The results indicated that the younger and older drivers (55 years and over) are over-involved in red-light running crashes. There is general consensus among researchers that older drivers tend to process information and take a corresponding action more slowly than younger driver. Slower reaction times for older versus younger drivers contribute to a disproportionately heightened degree of risk especially when older drivers are faced with two or more choices of action (Staplin et al., 2003). However, younger drivers are more likely related to aggressive driving attitude, speeding, and careless driving. Those behaviors greatly contribute to the red-light running violation and crash occurrence. Generally, while younger drivers tend to drive in situations conditions that increase their risk, older drivers tend to avoid adverse conditions in an attempt to compensate for the decline in their driving capability. This concept explained why younger drivers have a larger crash propensity for cloudy or rainy weather, but older drivers are less involved in worse weather. Although the driver gender is not a main effect factor associated with
crash risk, it has interaction effects with driver age, vehicle type, and alcohol/drug use. Young male and old female groups are over-presented in the crashes, and for middle age groups (26-45 years), gender has no apparent crash propensity. Based on vehicle type, vans and light trucks have relatively higher crash risk and large size vehicles have the smallest crash propensity. Considering the interaction effect with gender, male drivers have larger crash propensities for light trucks and large size vehicles.

The analysis confirmed the substantial effect of alcohol/drug use on driver’s safety. Even drivers who had been drinking could be 3.56 times more likely involved in red-light running crashes than non-drinking drivers. Drivers with physical defects, especially those with fatigue/asleep or Seizure/Epilepsy/Blackout problems, were identified as highest risk group at signalized intersections. Medical countermeasures for those drivers are very necessary, especially when they drive in urban area with higher density of traffic-controlled intersections and larger traffic volumes. In addition, non-local driver tend to be over-presented in red-light running crashes.

Corresponding to the adverse environmental conditions and the higher risk driver populations, appropriate engineering countermeasures need to be considered to reduce the red-light running crash rate. From the perspective of the intersection design and operation, improvement of configuration conditions (geometrics) may lead to reduced reaction and stopping times, motorist confusion, or limited visibility of traffic control devices. Especially when a horizontal or vertical curve present at signalized intersections, sufficient sight distance not only to the signal head but also to the other approaches
should be satisfied, in order that the drivers going-through the intersection can detect potential conflicting vehicles in time. At the same time, motorist information countermeasures are necessary to provide advance information to the driver about the signal ahead, such as advanced warning signs. Furthermore, enhancing the signal display may contribute to reducing crash occurrence rate during severe weather conditions (Bonneson et al., 2002). From the perspective of drivers, it is possible that a portion of red-light running violations are unintentional behaviors. Facing the signal change, those with poor driving capability such as older drivers may be incapable of stopping for a red signal because of slower reaction times and incorrect judgment. Those drivers may benefit more from appropriately increasing signal change intervals under consideration of the drivers needing longer reaction time. On the other hand, some red-light runners (such as younger drivers) more likely are intentional violators who tend to drive at higher travel speed and beat the red-light to avoid stop delay. For such drivers, appropriate education program and enhancing enforcement countermeasures, such as red-light camera implementation, may have more apparent effects on reduction of the red-light running rate.

Lastly, it is worth mentioning that the use of red-light running rate as a surrogate measure for predicting crash rates can be a misleading and probably a wrong hypothesis. Traffic crashes are rare events. They do not occur as often as traffic violations and they may involve more complicated situations that traditional measures can not capture. For example, young drivers are identified as the group with the highest violation rate, but their relative crash risk is lower than older group. The crash propensity analyses in this
paper provide a better understanding of the red-light running problem and provide more information to seek effective crash countermeasures.
CHAPTER 4. CHARACTERISTICS OF UNPROTECTED LEFT-
TURN ACCIDENTS AT SIGNALIZED INTERSECTIONS

Left-turning traffic continues to be a major source of conflicts at intersections. At
signalized intersections, although the unprotected green phase can enhance the operation
efficiency, the frequency of serious crashes occurring between left-turn and going-
straight vehicles may increase because the left-turn vehicles failed to yield right-of-way.
This study investigated the unprotected left-turn crash propensity related to traffic
environments, driver characteristics, and vehicle types. The related research was based on
the 1999-2001 Florida Crash Database, which contains specific traffic information for all
police-reported motor vehicle crashes in the state of Florida in the US. Logistic
regression technique, decision tree model, and Quasi-induced exposure are used to
perform the statistical analysis. Statistical tests showed that significant independent
variables associated with the unprotected left-turn crash include the number of lanes,
divided/undivided highway, lighting condition, weather, posted speed limit, driver age,
Alcohol use, physical defect, residence code, gender, and type of vehicle. The tree model
segmented drivers into four homogeneous age groups: younger than 18 years, 18-55
years, 56-65 years, and older than 65 years. Further, the logistic regression confirmed
significant interaction effects between Driver age & Number of lanes, Driver age &
Accident time, Type of vehicle & Gender, and Speed limit & Driver residence. Compared
to the other crashes, left-turn crashes more likely caused serious injury and even fatal
injury. The crash risk for divided highways is higher than undivided ones probably because of sight distance problem. Improving street lighting conditions may help to reduce left-turn crashes at night. Drivers older than 65 years are the most vulnerable group in the crashes. 15-17 years drivers are also over-presented in the crash and more likely to be learner drivers. The particular education program is strongly suggested for the younger group. The crash propensity analyses in this paper may provide a better understanding of the unprotected left-turn risks and more information to seek effective crash countermeasures.

4.1 Methodology

4.1.1 Florida traffic crash database

Based on these variables in Florida traffic crash database, an unprotected left-turn crash at a signalized intersection was defined as a crash that took place at an intersection controlled by a traffic signal and involved two vehicles: one of them going-straight without improper action and the other turning left that failed to yield right-of-way. The purpose of eliminating from those involving more than two vehicles is intended to simplify the assignment of driver culpability. Additionally, the number of lanes at the crash site as originally recorded by the reporting officer includes both sides of the median where applicable. To test the crash trend and minimize the data error, this analysis focused on intersections located on the most typical highways with 2, 4, and 6 lanes.
Screening the 1999-2001 data showed that the crash involvement patterns and rates are very similar. A total of 72,912 two-vehicle crashes happened at signalized intersections in Florida. Among them, there are 20,853 left-turn crashes occupied 28.6 percent and ranked as the second most frequent crash type (the first one is angle collision type that occupied 35.2 %). Of those left-turn accidents, there are 10,359 cases (50.3%) caused by the unprotected left-turning drivers who failed yield right-of-way, of which 57.7% happened in urban area and caused 1422 (13.7%) incapacitating injuries and 84 (0.8%) deaths. According to vehicle damage, those crashes resulted in 71.2% disabling damage rate and 27.4% functional damage rate in the left-turn vehicles.

4.1.2 Quasi-induced exposure technique

In the cases of unprotected left-turn crashes, at-fault drivers were defined as those turning left who failed to yield right-of-way while not-at-fault drivers were going-straight without improper action. Further, the quasi-induced-exposure technique can also be used to develop measures of the relative accident propensity or risk of different vehicle types and analyze the interaction relationship between driver factors and environmental conditions.

However, to test the main effect of traffic environments on the left-turn crashes, the most common dilemma for the study is the difficulty in obtaining data of environmental exposure. For example, it was very difficult to obtain the percentage of the signalized intersection with 55mph speed limit in the total signalized intersections in Florida and the
related traffic volume data. In this study, all two-vehicle crashes that happened at signalized intersections were split into two groups: left-turn crashes and non left-turn crashes. Thus, those non left-turn crashes can be used as environmental exposure information, and the comparisons between environment distribution in left-turn and non left-turn crashes can be used to investigate the crash propensities.

4.1.3 Logistic regression

A binary logistic regression is proper to use when the dependent is a dichotomy (an event happened or not) and can be applied to test association between a dependent variable and the related potential factors, to rank the relative importance of independents, and to assess interaction effects. The binary logistic regression is used in this study since for driver/vehicle analyses, the dependent variable Y (driver classification) can only take on two values: Y=1 for at-fault driver, and Y=0 for not-at-fault driver; for traffic environment analyses, the dependent variable Y (crash classification) also take on two values: Y=1 for the left-turn crash, and Y=0 for non-left-turn crash.

Furthermore, the P-values generated from the logistic regression qualitatively indicate the statistical importance of those RAIR comparisons for those potential drivers/vehicles/environments variables. The SAS program procedure, LOGISTIC, was used for model development in this study and hypothesis tests are based on 0.01 significance level.
4.1.4 Decision trees

In additions, the driver age analysis was always the most important topic in traffic safety studies, but most of them grouped driver age based on the researcher’s subjective age classification, such as, the ten-year interval. The advantage of decision trees over many of the other methods used here is to construct driver classification by dividing the data set into smaller and more homogeneous groups, so as to segment age groups of drivers and make further analyses. In tree-structured representations, a set of data is represented by a node, and the entire dataset is represented as a root node. When a split is made, two child nodes are formed, which correspond to partitioned data subsets. If a node is not to be split any further, it is called a leaf; otherwise, it is an internal node. For each leaf, a decision rule is made and applied to all observations in the leaf. For more detailed discussions of tree methodology, the reader is referred to Breiman et al. (1984).

In this study, the SAS Enterprise Miner Program was used to develop the classification tree based on the Entropy measure of disparity. The Entropy algorithm begins with the entire set of data to split the data into two subsets based on the attribute with the lowest entropy value pointing to the most informative attribute among a set of attributes, and then repeatedly splits until the size of each subset reaches an appropriate level. Two previous papers (Strnad et al., 1998; Vorko et al., 2000) clearly presented the Entropy method and related mathematical expression for decision trees.
Further, proportion misclassified (default in SAS Miner) is used as the model assessment measure to choose the tree with the smallest misclassification rate, which is the total number of misclassified points divided by the total number of data points. Misclassification rates based on training data are decreasing monotonically as the number of nodes increase, while the misclassification rates based on the validation data will reach a minimum value corresponding to the best size tree model. The model was constructed using only two-thirds of the available data as the training sample and one-third the size of the original dataset was randomly selected as the validation sample.

4.2 Results and Analyses

4.2.1 Environment factors

Table 4-1 lists the significant results of environmental comparisons between the left-turn crashes and non-left-turn crashes that occurred at signalized intersections. The column of relative frequency shows crash distribution within each type of traffic environment factors. The column for RAIR (Odds of logistic regression) is equal to the relative frequency in left-turn crashes divided by that in non-left-turn crashes.
Table 4-1: Results of Environmental Comparisons between the Left-turn Crashes and Non-left-turn Crashes

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Left-turn Crashes</th>
<th>Non-Left-turn Crashes</th>
<th>Total</th>
<th>RAIR (Odds)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Level</td>
<td>Count</td>
<td>%</td>
<td>Count</td>
</tr>
<tr>
<td>Name</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of lanes</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1462</td>
<td>14.1%</td>
<td>12302</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>5776</td>
<td>55.8%</td>
<td>30649</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>3121</td>
<td>30.1%</td>
<td>19602</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highway</td>
<td>Divided</td>
<td>6711</td>
<td>64.8%</td>
<td>38207</td>
</tr>
<tr>
<td></td>
<td>Undivided</td>
<td>3648</td>
<td>35.2%</td>
<td>24346</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lighting condition</td>
<td>Daylight</td>
<td>6592</td>
<td>63.6%</td>
<td>40402</td>
</tr>
<tr>
<td></td>
<td>Dusk</td>
<td>330</td>
<td>3.2%</td>
<td>1685</td>
</tr>
<tr>
<td></td>
<td>Dawn</td>
<td>145</td>
<td>1.4%</td>
<td>765</td>
</tr>
<tr>
<td></td>
<td>Dark (Street light)</td>
<td>2798</td>
<td>27.0%</td>
<td>17305</td>
</tr>
<tr>
<td></td>
<td>Dark (No light)</td>
<td>443</td>
<td>4.3%</td>
<td>2011</td>
</tr>
<tr>
<td></td>
<td>Unknown</td>
<td>51</td>
<td>0.5%</td>
<td>385</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weather</td>
<td>Clear</td>
<td>7853</td>
<td>75.8%</td>
<td>46669</td>
</tr>
<tr>
<td></td>
<td>Cloudy</td>
<td>1782</td>
<td>17.2%</td>
<td>10076</td>
</tr>
<tr>
<td></td>
<td>Rain</td>
<td>669</td>
<td>6.5%</td>
<td>5346</td>
</tr>
<tr>
<td></td>
<td>Other</td>
<td>55</td>
<td>0.5%</td>
<td>462</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Speed Limit</td>
<td>25</td>
<td>180</td>
<td>1.7%</td>
<td>1520</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>1129</td>
<td>10.9%</td>
<td>9229</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>2063</td>
<td>19.9%</td>
<td>13320</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>2452</td>
<td>23.7%</td>
<td>11154</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td>3588</td>
<td>34.6%</td>
<td>20011</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>322</td>
<td>3.1%</td>
<td>1722</td>
</tr>
<tr>
<td></td>
<td>55</td>
<td>234</td>
<td>2.3%</td>
<td>1466</td>
</tr>
<tr>
<td></td>
<td>Other</td>
<td>391</td>
<td>3.8%</td>
<td>4131</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Injury Severity</td>
<td>No</td>
<td>2403</td>
<td>23.2%</td>
<td>20636</td>
</tr>
<tr>
<td></td>
<td>Possible</td>
<td>3231</td>
<td>31.2%</td>
<td>20240</td>
</tr>
<tr>
<td></td>
<td>Non-Incapacitating</td>
<td>3219</td>
<td>31.1%</td>
<td>15189</td>
</tr>
<tr>
<td></td>
<td>Incapacitating</td>
<td>1422</td>
<td>13.7%</td>
<td>6071</td>
</tr>
<tr>
<td></td>
<td>Fatal</td>
<td>84</td>
<td>0.8%</td>
<td>417</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>10359</td>
<td>100.0%</td>
<td>62553</td>
<td>100.0%</td>
</tr>
</tbody>
</table>
About one-half of left-turn crashes (55.8%) happened at intersections located on 4-lane highway. The second in the rank is 6-lane highway, which occupied 30.1%. For all two-vehicle accidents at signalized intersections, 4 lanes and 6 lanes also presented most common types and they constituted 50.0% and 31.2% accidents. Compared to the non-left-turn crashes, the conditional probabilities of left-turn accidents for 4-lane and 6-lane highways (15.9% and 13.7%) are significant higher than 2-lane highway. Relatively, the 4-lane and 6-lane highways were more often associated with high-volume urban locations and intersection negotiation for unprotected left-turn drivers requires more complex decisions involving more conflict vehicles and more visually distracting conditions.

More left-turn crashes (6711) happened at divided highways than those (3648) occurring at undivided highways. The relative crash risk of left-turn accidents at divided highways (1.06) is 1.18 times as high as the undivided highways. According to the previous studies that were mentioned before, for divided highways at intersections with wider medians, vehicles in the opposite left-turn lane waiting to turn left can restrict the (left-turning) driver's view of oncoming traffic in the through lanes. AASHTO (2001) reported that the typical poor visibility of opposing through traffic usually occurs at intersections with medians wider than 18 ft. In Florida, the designs of protected/permitted left-turn phase or unprotected green phase are not uncommon at intersections with very wide median. That may be the potential reason explaining why the probability that left-turn accidents happen at divided highways is higher.
For the light conditions, most accidents happened at the daytime (63.6%) followed by the dark condition with streetlight (27.0%), and their relative crash risks are similar and relatively lower than other conditions. The left-turn crashes are most likely to happen at the dark condition without streetlight (RAIR=1.33), followed by dusk and dawn surrounding (1.18 and 1.14). The trend of this analysis is fairly typical of what one would expect of that weak lighting conditions affect vision and judgment of drivers so as to contribute to left-turn accidents, especially for the older drivers.

In Florida, drivers frequently drive in the cloudy and rainy weather conditions and the corresponding left-turn crash rates (17.2% and 6.5%) are very high. The analysis shows that the RAIR for cloudy weather (1.07) is slightly higher than clear weather (1.02). The presumable reason is that for the cloudy weather, the light condition is worse and drivers may have more aggressive attitude to avoid the pending rain when it is cloudy. However, the risk for rain weather is apparently lower than clear weather presumably because drivers slow down the travel speed and are more cautious when it is raining, and the drivers with relatively weak driving ability, for example older drivers or new drivers, may possibly avoid the adverse driving environments. Moreover, compared to the left-turn crashes, the rear-end crashes are more likely to be over-presented at signalized intersections.

The most common type of left-turn collisions (34.6%) happened at intersections with 45mph speed limit and fewer crashes occurred at intersections with speed limits higher than 50mph and speed limit lower than 30mph. From the table 1, there is a tendency that
the left-turn risk for the higher speed limits (>35mph) is significantly larger than that for lower speed limits. The presumable reason is that at the lower operation speed, both left-turning and coming-through drivers can rapidly make a turn or stop to avoid the accident. Further, the risk (RAIR=1.33) for the 40 mph speed limit is highest presumably because the operation speed variance of going-straight vehicles for the 40 mph speed limit may be relatively larger so as to increase the probability of the left-turning driver’s erroneous gap judgment.

Additionally, the table 4-1 also lists the injury severity comparison between left-turn crashes and other crashes. The no-injury rate of left-turn crashes is as low as 23.2 percent of total crashes. The result shows a clear trend that the left-turn crashes more likely caused serious injury and even fatal injury than other intersection crashes.

4.2.2 Driver age segment base on the decision tree model

Based on Quasi-induced exposure, the dataset input to Enterprise Miner consisted of 20,718 observations from 10,359 unprotected left-turn crashes at signalized intersections. 13,881 (two-thirds of data) of them were included in the training sample and 6,837 (one-third of data) were randomly selected and set aside as the validation sample. Using the validation data, the misclassification rates reached a minimum value (0.3883) for the tree with 6 leaves, as shown in Figure 4-1. Figure 4-2 is the decision tree diagram for driver/vehicle characteristics. In the node boxes, the top two lines give the node number and total number of cases entering the node; the third and fourth lines show the
proportions of at-fault (coded as 1) or not-at-fault (coded as 0) drivers/vehicles in the crashes. The diamond-shaped boxes indicate important variables used to split data, which include driver age, alcohol use, and vehicle type. From the figure 2, the classification tree basically segments the data into four age groups: <=17 years, 18-55 years, 56-65 years, and >65 years. The drivers older than 65 years of age in Node 3 presented the highest risk involving left-turn crashes, which conditional crash probability (0.749) is apparently higher than other age groups. The divers younger than 18 years in Node 4 also present relatively higher crash ratio (65.1%), although their crash frequency is smaller (559 of 859 cases). The 18-55 years drivers in Node 36 have the largest crash frequency (3895 of 9362 cases), but the conditional crash probability (0.416) of them is lower than any other groups. The crash risk of the 56-65 age group is lower than the younger group but higher than the middle age group. The tree model also shows that the left-turn crashes involved in alcohol use mainly happened to drivers from 18 to 65 years. The crash probability of alcohol use in Node 11 is 0.80, higher than those in all the other nodes. Moreover, there are a total of 13 types of vehicles classified by Florida crash database. This study mainly focused on four types of vehicles including automobile (Level 1), passenger van (Level 2), pickup/light truck (Level 3), and large size vehicle (Level 4). The large size vehicle is combined with the medium truck, heavy truck, truck-tractor, motor home, and bus, since the sample size for each one is very small. The other vehicles in Node 21 such as motorcycle, moped, terrain vehicle, and train shows a different attribute from the four levels of vehicles in Node 20, but this study is not interested in them.
Figure 4-1: The best size tree model based on missing Classification

Figure 4-2: The model of tree classification for driver/vehicle characteristics based on Quasi-induced exposure
4.2.3 Logistic regression analysis for driver/vehicle characteristics

At the first stage of analysis, the main effect logistic regression analysis is used to examine driver/vehicle characteristics associated with the left-turn crashes, where the driver age was categorized into four age groups based on the tree classification result. Table 4-2 lists the model estimation and odds ratios properly adjusting other factors for significant independent variables including Driver age, Alcohol use, Physical defect, Residence code, Gender, and Type of vehicle. Figure 4-3 illustrates comparisons of relative crash involvement ratios between different levels of those factors.

Table 4-2: Model Estimation and Odds Ratios for Significant Independent Variables Related to Driver/vehicle Characteristics

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Coefficient Estimate</th>
<th>Odds Ratio</th>
<th>95% Wald confidence limits</th>
<th>Wald Chi-square</th>
<th>P-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>0.270</td>
<td></td>
<td></td>
<td>31.088</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Driver age</td>
<td></td>
<td></td>
<td></td>
<td>879.7821</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>&gt; 65 vs &lt; 18</td>
<td>0.614</td>
<td>1.847</td>
<td>1.624</td>
<td>2.101</td>
<td>87.493</td>
</tr>
<tr>
<td>&gt; 56-65 vs &lt; 18</td>
<td>-0.229</td>
<td>0.795</td>
<td>0.694</td>
<td>0.911</td>
<td>10.943</td>
</tr>
<tr>
<td>18-55 vs &lt; 18</td>
<td>-0.741</td>
<td>0.476</td>
<td>0.433</td>
<td>0.524</td>
<td>231.746</td>
</tr>
<tr>
<td>Alcohol use</td>
<td></td>
<td></td>
<td></td>
<td>195.559</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Had been drinking vs no use</td>
<td>1.270</td>
<td>3.562</td>
<td>2.652</td>
<td>4.784</td>
<td>71.234</td>
</tr>
<tr>
<td>Under influence vs no use</td>
<td>2.128</td>
<td>8.397</td>
<td>5.797</td>
<td>12.165</td>
<td>126.626</td>
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<td>Physical defect</td>
<td></td>
<td></td>
<td></td>
<td>16.7584</td>
<td>0.0002</td>
</tr>
<tr>
<td>Other defect vs No defect</td>
<td>1.100</td>
<td>3.004</td>
<td>1.661</td>
<td>5.434</td>
<td>13.232</td>
</tr>
<tr>
<td>Eyesight defect vs No defect</td>
<td>0.358</td>
<td>1.431</td>
<td>0.988</td>
<td>2.073</td>
<td>3.594</td>
</tr>
<tr>
<td>Residence code</td>
<td></td>
<td></td>
<td></td>
<td>30.0893</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Non-Local vs local</td>
<td>0.240</td>
<td>1.272</td>
<td>1.167</td>
<td>1.386</td>
<td>30.0893</td>
</tr>
<tr>
<td>Gender</td>
<td></td>
<td></td>
<td></td>
<td>72.0717</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Female vs Male</td>
<td>0.259</td>
<td>1.296</td>
<td>1.221</td>
<td>1.376</td>
<td>72.0717</td>
</tr>
<tr>
<td>Type of vehicle</td>
<td></td>
<td></td>
<td></td>
<td>99.2965</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Other vs car</td>
<td>-1.436</td>
<td>0.238</td>
<td>0.174</td>
<td>0.325</td>
<td>81.580</td>
</tr>
<tr>
<td>Large vehicle vs car</td>
<td>-0.087</td>
<td>0.916</td>
<td>0.731</td>
<td>1.148</td>
<td>0.578</td>
</tr>
<tr>
<td>Light truck vs car</td>
<td>-0.175</td>
<td>0.84</td>
<td>0.769</td>
<td>0.916</td>
<td>15.406</td>
</tr>
<tr>
<td>Van vs car</td>
<td>0.093</td>
<td>1.098</td>
<td>0.98</td>
<td>1.23</td>
<td>2.581</td>
</tr>
</tbody>
</table>
The graph for driver age shows a typical U-shape pattern, which indicates that middle age groups have lower risk and the younger and older groups have relatively higher risk. Odds ratios resulting from logistic regression showed the crash risk of the group older than 65 years could be 84.7% higher than the younger group (younger than 18 years), while the risk of the 56-65 years group could be 20.5% lower than the younger group.
The oldest group (> 75 years) presented the highest risk, presumably because of age-
related deterioration of their physical and cognitive abilities that may result in the
erroneous left-turn gap acceptance. The younger group has larger accident propensity,
presumably because of less driving experiences, risk-taking, and other attitudinal factors.
Further, the most of 15-17 years drivers are new drivers and even have only learner
licenses, and they are more likely unfamiliar with the related traffic law. They may
misunderstand the traffic meaning conveyed by the signal so as to wrongly assume they
have the right-of-way to make a left-turn. For the middle age drivers, they have the
lowest risk and may benefit from more driving experiences and better physical condition.
Moreover, statistical tests show that driver gender is also a significant variable associated
with the left-turn crash risk. Compared to the male drivers, the female drivers are 30%
more likely over-involved in the crashes.

As shown in Table 4-2 and Figure 4-3, the drivers under influence of alcohol are
substantially over-involved in crashes and their left-turn crash risk could be 8.4 times
higher than normal drivers. Alcohol reduces alertness, interferes with judgment and
impairs vision, and impairs the driving ability. Even drivers who had been drinking also
show 3.6 times higher risk than non-drinking drivers. Additionally, as not-at-fault drivers,
those with alcohol use performed worse to avoid crashes than normal ones. There is a
higher chance to underestimate their risk involving crashes based on the induced
exposure technique.
The crash database also recorded possible physical defects of drivers involved in crashes, which are coded as 1-No defect, 2-eyesight defective, 3-fatigue/sleep, 4-hearing defect, 5-illness, 6-Seizure/Epilepsy/Blackout, and 7-Other Physical defect. Since the data sample sizes in some levels are very small, so the levels 3-7 are combined as other defect (see Table 4-2). It is not surprising that drivers with some kind of physical defects present a higher left-turn crash risk. The drivers, with fatigue/sleep and Seizure/Epilepsy/Blackout defects, who may even lose basic ability to control vehicle would definitely contribute to left-turn crashes at intersections. Although drivers with eyesight defect show a higher crash involvement ratio, but the difference from the normal driver is not statistically significant based on a 0.05 significance level (P-value=0.058) possibly due to the insufficient data sample size. Generally, eyesight defective may have significant effects on driver’s judgment of distance, vehicle speed, gap acceptance, and signal information at signalized intersections.

Considering the driver residence, local drivers may benefit from their driving experiences to the familiar traffic environments so as to avoid the adverse traffic conditions. In the crash dataset, the non-local drivers include those living not in the local county where crash occurred, but elsewhere in the state of Florida, other state, or other country. The result showed that the left-turn crash risk of the non-local drivers could be 27.2% higher than the local drivers.

Based on the vehicle type, the figure 4-3 shows that the RAIR for automobile is close to that for passenger van while RAIR for light truck is very similar as the large size vehicle,
and automobile and passenger van are over-presented in the left turn crashes. Adjusting other factors, the logistic regression testified that the crash risk for light truck could be 16% lower than the automobile, but the crash risk for the large size vehicle is not significantly different from that for the automobile (see Table 4-2). Normally, larger size vehicles tend to void traffic peak, their operation speeds are relatively lower than small cars, and their drivers, who mostly are professional, are rarely involved in alcohol/drug use and extreme aggressive driving attitude. Moreover, since larger size vehicles are more noticeable at intersection, the drivers of conflicting going-straight vehicles may more likely detect the larger size left-turn vehicles before they cross the major road.

4.2.4 Interaction analysis between environment and driver/vehicle factors

After confirming the main effect model, the next logistic regression analysis is to explore the possible significant interactions between driver characteristics and traffic environmental conditions. Statistical tests show that there are four interaction factors associated with left-turn crashes including: Driver age & Number of lanes (P-value=0.0041), Driver age & Accident time (P-value=0.0037), Type of vehicle & Gender (P-value =0.0005), and Speed limit & Driver residence (P-value=0.0099). Figure 4-4 illustrates the effects of these interaction factors by relative accident involvement ratios.
Figure 4-4: Effects of interaction factors by relative accident involvement ratios

The results indicate that for the driver groups older than 55 years, their crash risk increases as the number of lanes in the highway increases, while the younger group shows a contrary trend. Normally, intersection negotiation for unprotected left-turn drivers requires more complex decisions involving more conflict vehicles and more visually distracting conditions for the larger intersections located on 4 or 6 lane highways. Staplin et al. (2001) report that slower reaction times for older versus younger drivers contribute to a disproportionately heightened degree of risk especially when traffic environments tend to be more complex. Therefore, the age difference in left-turn crashes becomes more exaggerated at intersections with a larger number of lanes and higher traffic volume. The driver age effect is also influenced by the light conditions. The
analysis shows that for age groups younger than 18 years or between 56 and 65 years, the left-turn crashes more likely happened during night than daytime. On the contrary, the older drivers are over-involved in the crashes during daytime compared to the night. For middle age groups, there is no apparent tendency on light conditions. The result is not consistent with previously published left-turn crash study on older drivers. Chandraratna and Stamatiadis (2003) indicated that older drivers are 1.65 times more likely to be involved in left turn crashes at nighttime as compared to daytime while younger drivers have only 1.11 times higher risk to be involved in left turn crashes compared to daytime. As shown in the figure 4-4, gender shows an interaction effect with type of vehicles: male drivers have larger crash propensities for automobile, passenger van, and light truck while female drivers are over-presented in the crashes for large size vehicle. Furthermore, the interaction analysis shows that the non-local drivers are more likely over-involved in the left turn crashes happening at intersections with 30 to 40 mph speed limits, but local drivers have no such apparent tendency in highway speed limit.

4.3 Conclusions and Discussions

Using 1999-2001 Florida traffic crash databases, this study investigated the overall characteristics of the unprotected left-turn crashes at signalized intersections based on Quasi-induced exposure technique, decision tree method, and logistic regression analysis. Through comparisons between the left-turn crashes and other two-vehicle crashes at signalized intersections, the traffic environment analyses show that the number of lanes,
divided/undivided highway, lighting condition, weather, and posted speed limit are significant variables associated with left-turn crashes. Compared to the other crashes, the conditional probabilities of left-turn accidents for 4-lane and 6-lane highways are significantly higher than the 2-lane highway. The relative crash risk at divided highways is higher than undivided highways. For divided highways with medians wider than 18 ft, sight distance of left-turn drivers may be obstructed by the simultaneous left-turn vehicles in opposing left-turn lanes. The AASHTO (2001) design guide (5) recommended two methods, parallel offset left-turn lane and tapered offset left-turn lane, to ensure required left-turn sight distance, which may effectively reduce the unprotected left-turn crash rate for divided highways with wide medians. The left-turn crashes are most likely to happen at the dark condition without streetlight, and the crash risk for nighttime with streetlight is lower than dusk or dawn surrounding. This result confirmed the precious analysis that that more street lighting may help to reduce the problems with left turns at night (Chandraratna and Stamatiadis, 2003). Compared to the clear weather, the cloudy weather is over-presented in the crashes. It was also found that that the crash risk for relatively higher speed limits (>35mph) are significantly larger than that for lower speed limits.

Based on the tree classification, drivers were segmented into four homogeneous age groups: younger than 18 years, 18-55 years, 56-65 years, and older than 65 years. Since the most of 15-17 years drivers are more likely to be learner drivers or new drivers with less driving experiences, the particular education program to emphasize the unprotected left-turn risk and related traffic policy at signalized intersections is strongly suggested for
the younger group. The analysis is consistent with many previous findings as mentioned before, the older driver present the highest left-turn crash risk. The risk for older drivers is increasing as the traffic environments become more complex at intersections with larger number of lanes and higher traffic volume. Therefore, for the area with high density of older drivers, reducing left-turn chance and protected left-turn signal phase are recommended. One of the interesting findings of this work is that the crash risk of younger drivers is over-presented during nighttime while the older drivers are under-involved in the crashes during nighttime. The result testified that under adverse vision conditions, older drivers’ conservative driving attitude could compensate for the decline in their driving capability. A prior study indicated that drivers 55 years and older, particularly those who had a previous history of crashes, avoided driving in rain, during rush hour and making left-turns across traffic (Ball et al., 1998). The examination of the relative crash propensity of driver gender indicates that the female drivers are more likely to be involved in left turn crashes compared to the male drivers. The analysis confirmed the substantial effect of alcohol use on driver’s safety. Especially, left-turn crashes involved in alcohol use mainly happened in middle age drivers from 18 to 65 years. Moreover, drivers’ physical defect and unfamiliarity with local traffic environment were also found to be significant factors associated with the left-turn crashes.

Mostly, points of impact in the crashes are in front or right side of the left-turn vehicles and the estimated speeds of oncoming-through vehicles are relatively higher. Compared to the other crashes, 71.2% disabling damage rate and 27.4% functional damage rate for left-turn vehicles is extremely high and the left-turn crashes more likely caused serious
injury and even fatal injury. Therefore, for traffic engineers and researchers, it is worthwhile to pay more attentions to the traffic countermeasures to reduce the crash occurrence rate. The crash propensity analyses in this paper may provide a better understanding of the unprotected left-turn risks and more information to seek effective countermeasures.
CHAPTER 5. DRIVING SIMULATOR EXPERIMENT FOR
TESTING PAVEMENT MARKING COUNTERMEASURE TO
REDUCE RED-LIGHT RUNNING RATE

According to literature findings, there is no related pavement marking countermeasure to provide drivers yellow phase information and diminish the likelihood of red-light running rate. In this study, a pavement marking countermeasure is proposed to help drivers make a clear stop/go decision at the onset of yellow phase to reduce red-light running and ultimately minimize intersection accident rates. A pavement marking with word message ‘SIGNAL AHEAD’ (see Figure 5-1) is placed on the pavement of the upstream approach of a signalized intersection and is sufficient to permit vehicles cruising around speed limit to stop safely before reaching the intersection stop bar. The proposed policy is that, when drivers are located upstream of the marking at the yellow onset, they are encouraged to stop at the intersection if they are cruising around speed limit. On the other hand, when drivers are located downstream the marking at the yellow onset, they are encouraged to cross the intersection if they are cruising around speed limit. To test the effectiveness of the pavement-marking countermeasure on red-light running, this section documented an experiment study based on the UCF driving simulator. The purposes of the research are to test the theory behind pavement marking countermeasure and to find the tendency of driver behaviors during the signal changing at intersections.
5.1 Driving Simulator Experimental Design

5.1.1 Experiment factors

This experiment utilized a within-subjects repeated measures factorial design to test effectiveness of the pavement-marking countermeasure on red-light running. The three treatment design factors include speed limit, pavement-markings and yellow phase onset distance. There are two levels for speed limits (30 mph and 45 mph), two levels for program types (with marking or without marking), and eight yellow phase onset distances for each speed-limit type measured from the position of the approaching vehicle when yellow phase starts to the stop bar of the intersection approach. The factorial manipulation of the three factors described above (speed, pavement-markings, and yellow onset distance) resulted in 32 unique intersection-approach types.
With the different onset distances, a total of 8 test-signalized intersections in the driving simulator's visual database were identified, as shown in Figures 5-2-a and 5-2-b. Among those, half of the intersections are along an urban street in a downtown area with 30 mph speed limit and the other 4 intersections are along a suburban arterial with 45 mph speed limit. The experimental intersections are indicated by light color in the figure. There were additional signalized intersections, intermingled with the test intersections, which display continuous green phase. These locations are displayed by dark color in the figure. The continuous green intersections are designed to keep the subject from continually expecting a signal change at every intersection.

(a) Downtown scenarios with 30 mph speed limit
Figure 5-2: Arrangement for test signalized intersection with different yellow onset distance

In a pilot study (Yan, et al., 2005), for the 30 mph speed limit, the eight points for yellow onset distances range from 49.2 to 278.8 ft with 32.8 m increment; for the 45 mph speed limit, the eight points range from 164 to 393.6 ft also with 32.8 ft increment. The results based on 12 subjects showed that for the 30 mph speed limit, there were no stops happened for yellow onset distances 49.2 ft and 114.8 ft. For the 45 mph speed limit, there were no stops happened at intersections with 164 ft yellow onset distances, and for
the 328 ft, 360.8 ft, and 393.6 ft yellow onset distances, those stop rates were very close. The pilot experiment results suggested that for this future design, the ranges of yellow onset distance for both speed limits should shrink and the yellow onset distance for each test intersection need be adjusted correspondingly.

Therefore, in this formal experiment design, for the 30 mph speed limit, the eight points for yellow onset distances range from 82 to 278.8 ft with 28.11 ft increment; for the 45 mph speed limit, the eight points range from 180.4 to 360.8 ft with 25.77 ft increment. The yellow onset distances were identical for both program types (with and without marking) and were randomly assigned to those approaches of test-signalized intersections, as shown in Figure 5-2-a and 5-2-b.

To evaluate the effect of the proposed pavement marking, a without-with study was conducted. In the "Without" scenarios, none of the intersection approaches had the pavement marking and in the "With" scenarios all had them. Since two directions of each road can be used as two routes (see Figure 5-2), totally there are 4 routes and 8 different (without-with) scenarios to test. For each scenario, the experiment elapsed time was designed not to exceed 3 minutes.

5.1.2 Yellow change interval

In the current edition of ITE’s *Traffic Engineering Handbook* (8), a standard equation is provided as a method to calculate the yellow change interval, $YT$, is as follows:
\[ YT = t + \frac{V}{2a + 64.4g} \]  (5-1-1)

Where,

\[ t = \text{reaction time (1.0 s)} \]

\[ V = \text{the 85th percentile speed or speed limit (ft/sec)} \]

\[ a = \text{gravitational acceleration (10 ft/s}^2\text{)} \]

\[ g = \text{grade of the intersection approach (g = 0, since level road is assumed).} \]

According to the equation (5-1-1), the duration time of the yellow change interval calculations for 30 mph and 45 mph intersections are shown as the following:

For 30 mph speed limit: \[ YT = 3.2 \text{ sec, round up to 3.5 sec} \]

For 45 mph speed limit: \[ YT = 4.3 \text{ sec, round up to 4.5 sec} \]

### 5.1.3 Pavement-marking position

The marking position is related to speed limit and vehicle’s deceleration rate. The distance from the marking to the intersection stop bar should be sufficient to permit vehicles to stop safely before reaching the intersection stop bar. According to the deceleration rate suggested by ITE, the distance from the marking to the stop bar is calculated by the following equation:

\[ X = vt + \frac{V^2}{2a + 64.4g} \]  (5-1-2)

Where
\[ X = \text{distance from the marking to the stop bar (ft)} \]
\[ V = \text{the 85th percentile speed or speed limit (ft/sec)} \]
\[ t = \text{reaction time (1.0 s)} \]
\[ a = \text{gravitational acceleration (10 ft/s}^2) \]
\[ g = \text{grade of the intersection approach (g = 0, since level road is assumed)} \]

According to the equation (5-1-2), the results of the marking-stop bar distance calculations for 30 mph and 45 mph intersections are shown as the following:

For 30 mph speed limit: \[ X= 140.8 \text{ ft (42.9 m)} \]
For 45 mph speed limit: \[ X= 283.8 \text{ ft (86.5 m)} \]

5.1.4 Experiment procedure

Upon arrival, the subjects were given an informational briefing about the driving simulator. Subjects were specifically advised to adhere to traffic laws, and to drive as if they were in normal everyday traffic surroundings. Then, a practice course was programmed on the driving simulator. During this process, subjects exercised driving to become familiar with the basic simulator operation.

Before proceeding to the formal experiment, each subject was informed that they would be driving under simulated conditions through a course that contained both conventional intersections and experimental intersections with pavement markings. Computer demo
and paper handouts were shown to help them understand the purpose of the pavement marking design.

Next, the subjects performed the red-light running experiment with the 8 scenarios, of which 4 scenarios have the pavement marking and 4 scenarios did not have pavement marking. Those with or without-marking scenarios were randomly loaded for each driver so as to eliminate the time order effect and bias from subjects to the experiment results. During the course of the experiment subjects were routinely checked for simulator sickness. Whenever sickness was found, the subject quit the experiment and the related data collected was removed. Finally, when subjects completed the formal experiments, a survey was used to gather information about their opinions of the proposed pavement marking and red-light running. Specifically, the survey investigated the red-light running reason and frequency of the potential violators in the real world, dilemma zone’s hazard at signalized intersections, and subjects’ attitude to the safety significance of the proposed pavement marking.

**5.1.5 Subjects**

As shown in Table 5-1, a total of 42 paid test subjects in two age groups, 18 younger subjects (<26 years), 24 middle-age subjects(26-55 years) were recruited and completed the experiment. According to gender, there were 24 male subjects and 18 female subjects for this research. The ratios of male to female and the younger group to the middle-age group closely represent Florida driver population distribution in Qausi-induced exposure.
method. As shown in Table 5-1, the ratio of male to female not-at-fault drivers is around 59% to 41% and the ratio of the younger group to the middle-age group is around 40% to 60%.

Every participant has a full driving license with a minimum of 1-year driving experience. Most of subjects were recruited from students/faculties in the University of Central Florida. Data analysis was based on the responses and decisions made by the 42 subjects approaching 32 signalized intersections. Each subject responded to 16 test signalized intersections with marking and 16 regular signalized intersections without marking for a total of 1344 driver-intersection encounters.

Table 5-1: Age and Sex Structure of the Subject Sample

<table>
<thead>
<tr>
<th>AGE</th>
<th>&lt;26 YEARS</th>
<th>26-55 YEARS</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Male</td>
<td>10</td>
<td>14</td>
<td>24 (57.1%)</td>
</tr>
<tr>
<td>Female</td>
<td>8</td>
<td>10</td>
<td>18 (42.9%)</td>
</tr>
<tr>
<td>Total</td>
<td>18 (42.9%)</td>
<td>24 (57.1%)</td>
<td>42 (100%)</td>
</tr>
</tbody>
</table>

5.2 Data Collection

Data logging includes experiment sampling time, vehicle positions, speeds, accelerations, information of driver's braking behavior, and records of signal phase status. Independent measurements include red-light running rate, probability to stop during yellow, deceleration rate, and reaction time after termination of green. To organize and easily
process data generated from the experiments, a FORTRAN program was developed to manipulate the experiment data output files (See Appendix A).

5.2.1 Red-light Running Rate

Red-light running rate is percentage of illegal entering intersections during red phase in the number of drivers meeting yellow phase onset. For example, if we hypothetically compare running red-light rate between scenarios with marking and without marking, one may observe the effect of the pavement marking countermeasure, as shown in Figure 5-3.

![Comparison of red light running rate](image)

Figure 5-3: Comparison of running red-light rate between before and after study

5.2.2 Probability to stop during the yellow phase

Another important question is how the marking influences the stopping behavior at the decision point, onset of the yellow. Does it improve the ability of the drivers to make
stop-go decision? Does it encourage safe stopping and reduce unsafe crossing? Probability of stopping as a function of the distance to the intersection from the onset of yellow will help to analyze the driving behavior with pavement marking program.

According to Logistic Regression method, probability of stopping as a function of the distance to the intersection from the onset of yellow can be developed. When drivers encounter yellow onset distances that are near or far from the intersection, most drivers will choose similar courses of action; either most will stop or most will cross the intersection. In these situations, a driver’s behavior is highly predictable, and easily anticipated by other drivers. In contrast, yellow onset distances where 50% of the drivers choose to stop may result in situations where stopping behavior is least predictable, and the likelihood of two successive drivers being in a region where they make conflicting decisions is greatest. For this reason, the region surrounding the 50% probability of stopping has been defined as the most hazardous portion of the intersection approach. Traffic signal change intervals are designed to minimize this region of uncertainty.

In a simulation study, Newton (1997) analyzed probability of stopping as a function of the distance for two traffic signal programs, with or without Traffic Light Change Anticipation System. The results are regressed as logit curves (See Figure 5-4). The uncertainty regions between the probabilities of 0.25 and 0.75 were calculated around the point of highest uncertainty. The analysis showed that for both of 40.3 km/hr and 72.5 km/hr approach speeds, larger uncertainty regions was also found in TLCAS intersection
than the regular one, which indicated that the new system increased the potential for rear-end collision between successive drivers approaching an intersection.

In another road study, Köll et al. (2002) tested the effect of TLCVAS through analyzing probability of stopping as a function of potential time to the intersection from the onset of yellow, which is the time to the stop line if the driver continues with unchanged speed from the first possible decision point (start of yellow). As shown in Figure 5-5, the uncertainty duration between 20% and 80% probability of stopping is about a second longer with TLCVAS program in comparison without ones.

Figure 5-4: Probability of stopping as a function of the yellow onset distance
For the pavement marking program, both methods of stopping probability as a function of yellow onset distance and as a function of potential time will be used to analyze the driving behavior of stop-go decisions.

5.2.3 Driver’s brake response time and deceleration rate

Another measure of effectiveness is the reaction time of the driver following the yellow onset. The time following the appearance of the yellow phase until the driver steps on the brake will also be compared for significant differences in human response attributable to the new situation. The shorter reaction time takes drivers to make decisions of deceleration or acceleration, the better effectiveness of the new countermeasures.
Deceleration rate at the yellow onset will also be compared for significant differences attributable to the presence of the markings. They are measured from vehicle’s position in which driver begins to step on brake after yellow onset to the stop bar of the intersection approach. Those values can be used to check if there will be some abnormal driving behaviors for the new program. For example, too large deceleration can contribute to rear-end collisions.

5.2.4 Dilemma zone analysis

Considering the approaching speed \( V \) of vehicles, the maximum distance \( X_c \) to safely cross the intersection is calculated by Equation 5-2-1:

\[
X_c = V * Y = V (t_r + \frac{V_{sl}}{2a})
\]

The minimum distance \( X_s \) to safely stop at the intersection is calculated by Equation 5-2-2:

\[
X_s = V (t_r + \frac{V}{2a})
\]

When a motorist is approaching to the intersection at the onset of yellow change interval, they must decide whether to stop or cross the intersection. Figures 5-6-a and 5-6-b illustrates the tendency of driver stop/go decision at onset of the yellow at signalized
intersections with 30 mph and 45 mph speed limits. The decision to stop is easy to make when the approach distance to the intersection is larger than $X_s$ at the onset of yellow change. Similarly, most of drivers tend to continue to travel through the intersection when the approach distance to the intersection is less that $X_c$. However, a vehicle can possibly execute neither crossing nor stopping maneuvers safely and comfortably if it happens to be located within the dilemma zone if the approach distance is larger than $X_s$ but less that $X_c$. There is also a possible option area as shown in the figures where the driver can either stop or cross the intersection safely. The length of the dilemma zone is dynamic and increases with the increment of approaching speeds, which can be calculated by Equation 5-2-3. So, the speeding drivers are most likely involved in the dilemma zone problem.

\[ X_s - X_c = V(t_r - YT + \frac{V}{2a}) \]  

(5-2-3)

Moreover, the length of the dilemma zone can also increase as the driver reaction time increase. As shown in Figure 5-6-a for the 30 mph speed limit, a vehicle with 15 mph approaching speed and around 80 ft from the intersection may fall within a dilemma zone if the driver reaction time is 2.5 seconds. For the 45 mph speed limit in Figure 5-6-b, the dilemma zone can happen at 200 ft to the intersection for the drivers with 2.5 seconds reaction time. However, based on the ITE standard in Equation (1), the designed perception-reaction time to the signal change is generally 1 sec. A driver’s reaction time may personally be larger than the design value, which could be affected by a number of
factors, including driver age and gender, driver experience, the distance to intersections, speed limits, and other factors.

Figure 5-6: Driver stop/go decision at onset of the yellow at signalized intersections
(Source: A conference paper of Köll et al. (2002) )
Dilemma zone analyses of comparison between with marking and without may help find the effect of the pavement marking countermeasure.

5.3 Experiment Results and Data Analyses

5.3.1 Operation speed

Operation speed is measured at each intersection at termination of the green phase. For the 30 mph speed limit, the mean of the speed was 33.26 mph; for the 45 mph speed limit, the mean of the average speed was 47.26 mph; and the histograms of the operation speed appear very close to normal distributions for both speed limits as shown in Figure 5-7. In the simulation environment, average operation speeds of drivers tend to be slightly higher than the speed limit, presumably because the simulator vehicle is always the leading vehicle in the traffic stream and the drivers were more likely to drive at free-flow speeds. Moreover, between scenarios without marking and with marking, there is no significant difference found in the operation speeds. For the 30 mph speed limit, the means of the speed without marking and with marking were 33.38 mph and 33.14 mph; for 45 mph speed limit, the means of the speed without marking and with marking were 47.47 mph and 47.05 mph (see Table 5-2). Therefore, the proposed marking design didn’t have a significant effect on the speed.
(a) For the 30mph speed limit

(b) For the 45mph speed limit

Figure 5-7: Distribution of operation speed
Table 5-2: Descriptive Statistics of Operation Speed

<table>
<thead>
<tr>
<th>Speed Limit</th>
<th>Scenario</th>
<th>Mean</th>
<th>N</th>
<th>Std. Deviation</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 mph</td>
<td>Without</td>
<td>33.3776</td>
<td>336</td>
<td>3.5269</td>
<td>23.85</td>
<td>55.68</td>
</tr>
<tr>
<td></td>
<td>With</td>
<td>33.1431</td>
<td>336</td>
<td>3.2774</td>
<td>23.98</td>
<td>49.53</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>33.2603</td>
<td>672</td>
<td>3.4039</td>
<td>23.85</td>
<td>55.68</td>
</tr>
<tr>
<td>45 mph</td>
<td>Without</td>
<td>47.4796</td>
<td>336</td>
<td>4.4003</td>
<td>32.76</td>
<td>67.87</td>
</tr>
<tr>
<td></td>
<td>With</td>
<td>47.0461</td>
<td>336</td>
<td>3.7099</td>
<td>35.96</td>
<td>61.98</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>47.2628</td>
<td>672</td>
<td>4.0725</td>
<td>32.76</td>
<td>67.87</td>
</tr>
</tbody>
</table>

5.3.2 Red-light running rate and time

Comparison of red-light running rates between scenarios with marking and without can directly reflect the effect of the pavement marking countermeasure. As shown in Table 5-3 and Figure 5-8, red-light running rate without marking information is apparently higher than that with. For 30 mph speed limit without marking, there were 15 red-light running events representing red-light running rate of 4.5 percent; for 45 mph speed limit without marking, there were 11 red-light running events representing a rate of 3.3 percent. However, with the help of marking, there were only 4 red-light running events representing a rate of 1.2 percent for 30 mph speed limit; for 45 mph speed limit with marking, there were 5 red-light running events representing a rate of 1.5 percent. Potentially, the pavement marking could results in a 74.3 percent reduction in red-light running. Chi-square test showed that the p-value is 0.005 and the reduction in red-light running rate with the marking is statistically significant based on the 0.05 significance level.
Another important measurement for a red-light runner is the travel time to the intersection after the yellow light expires. The longer the travel time is in the upstream of the intersection at the onset of the red phase, the more likely an angle crash happens. As shown in Figure 5-9, without marking, there are 4 red-light running events of which the travel time during the red phase is larger than 1 sec and that represent 15.4 percent red-light running behaviors; with marking, all of red light entries occur in the first second after the yellow light expires. The analysis shows that the pavement marking may reduce the red light running time and the probability of angle crashes. However, since the sample size of red-light running observations is very small, one can not draw a significant conclusion from such a few data.

Table 5-3: Number of Red-light Running Violations and Red-light Running Rate Without Marking and With Marking

<table>
<thead>
<tr>
<th>Speed limit</th>
<th>Marking</th>
<th>Red-light running</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>No</td>
<td>Yes</td>
<td>Total</td>
<td></td>
</tr>
<tr>
<td>30mph</td>
<td>Without</td>
<td>Count</td>
<td>321</td>
<td>15</td>
<td>336</td>
</tr>
<tr>
<td></td>
<td>% of Total</td>
<td>95.5%</td>
<td>4.5%</td>
<td>100.0%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>With</td>
<td>Count</td>
<td>332</td>
<td>4</td>
<td>336</td>
</tr>
<tr>
<td></td>
<td>% of Total</td>
<td>98.8%</td>
<td>1.2%</td>
<td>100.0%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>Count</td>
<td>653</td>
<td>19</td>
<td>672</td>
</tr>
<tr>
<td></td>
<td>% of Total</td>
<td>97.2%</td>
<td>2.8%</td>
<td>100.0%</td>
<td></td>
</tr>
<tr>
<td>45mph</td>
<td>Without</td>
<td>Count</td>
<td>325</td>
<td>11</td>
<td>336</td>
</tr>
<tr>
<td></td>
<td>% of Total</td>
<td>96.7%</td>
<td>3.3%</td>
<td>100.0%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>With</td>
<td>Count</td>
<td>331</td>
<td>5</td>
<td>336</td>
</tr>
<tr>
<td></td>
<td>% of Total</td>
<td>98.5%</td>
<td>1.5%</td>
<td>100.0%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>Count</td>
<td>656</td>
<td>16</td>
<td>672</td>
</tr>
<tr>
<td></td>
<td>% of Total</td>
<td>97.6%</td>
<td>2.4%</td>
<td>100.0%</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>Without</td>
<td>Count</td>
<td>646</td>
<td>26</td>
<td>672</td>
</tr>
<tr>
<td></td>
<td>% of Total</td>
<td>96.1%</td>
<td>3.9%</td>
<td>100.0%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>With</td>
<td>Count</td>
<td>663</td>
<td>9</td>
<td>672</td>
</tr>
<tr>
<td></td>
<td>% of Total</td>
<td>98.7%</td>
<td>1.3%</td>
<td>100.0%</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>Count</td>
<td>1309</td>
<td>35</td>
<td>1344</td>
</tr>
<tr>
<td></td>
<td>% of Total</td>
<td>97.4%</td>
<td>2.6%</td>
<td>100.0%</td>
<td></td>
</tr>
</tbody>
</table>
Figure 5-8: Red-light running rate comparison between with marking and without

Figure 5-9: Travel time to the intersection after the yellow light expires

5.3.3 Dilemma zone analyses
Table 5-4 shows the proportions of stopping and crossing decisions at intersections with markings and without markings by drivers during the simulator experiment. The situation that drivers were located in a stop zone, cross zone, optional zone, or dilemma zone are based upon a kinematics analysis using driver velocity and distance values at the onset of the yellow phase (See Section 5.2.4). In comparison, the pavement marking reduced the number of occurrences where drivers chose to continue through an intersection when it was not safe to proceed (4.36%) compared to the without marking (10.6%). This reduction in unsafe crossings appears to be due to the marking information as drivers were located upstream of the marking. Chi-square test showed that the p-value is 0.008 and the reduction in unsafe crossings with the marking is statistically significant based on the 0.05 significance level.

In the other hand, the pavement marking reduced the number of occurrences where drivers chose to stop at an intersection when it was not safe to stop (20.2%) compared to the without marking (24.3%). This reduction in unsafe stops appears to be due to the marking information as drivers were located downstream of the marking. However, the Chi-square test showed that the p-value is 0.301 so that the reduction in unsafe stops with the marking is not significant. Further, situations in which a driver could not safely stop or safely cross an intersection were defined as dilemma situations and situations in which the driver could either safely choose to stop or choose to cross the intersection were defined as option situations. It appears that when they are located in option zones, drivers are more likely stop at intersections with markings (64.7% Vs 42.9%) but the tendency is not statistically significant (P=0.601); when they are located in dilemma zones, the
drivers are more likely stop at intersections with marking (92.6% Vs 79.2%) but the difference is not statistically significant (P=0.226).

Table 5-4: Dilemma Zone Analysis

<table>
<thead>
<tr>
<th>Situation that drivers are encountering</th>
<th>Stop</th>
<th>Cross</th>
<th>Optional</th>
<th>Dilemma</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without Marking</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross</td>
<td>30</td>
<td>271</td>
<td>4</td>
<td>5</td>
<td>310</td>
</tr>
<tr>
<td>Stop</td>
<td>253</td>
<td>87</td>
<td>3</td>
<td>19</td>
<td>362</td>
</tr>
<tr>
<td>Total</td>
<td>283</td>
<td>358</td>
<td>7</td>
<td>24</td>
<td>672</td>
</tr>
<tr>
<td>With Marking</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross</td>
<td>12</td>
<td>276</td>
<td>6</td>
<td>2</td>
<td>296</td>
</tr>
<tr>
<td>Stop</td>
<td>270</td>
<td>70</td>
<td>11</td>
<td>25</td>
<td>376</td>
</tr>
<tr>
<td>Total</td>
<td>282</td>
<td>346</td>
<td>17</td>
<td>27</td>
<td>672</td>
</tr>
</tbody>
</table>

5.3.4 Driver’s stop/go decision based on yellow onset distances

Driver’s stop/go decision is the most essential behavior at signalized intersection because wrong stop/go judgments are directly related to traffic crashes happening such as red-light running or rear-end crashes. From the experiment results, generally, as the yellow onset distances increase, the cross rate decreases and the stop rate increases. Tables 5-5 and 5-6 show the comparisons of stop rates between with marking and without for different yellow onset distances at the 30 mph and 45 mph speed limits.
Table 5-5: Drivers’ Stop/cross Decision According to Yellow Onset Distance for 30 mph

<table>
<thead>
<tr>
<th>Yellow Onset Distance</th>
<th>82.00</th>
<th>110.1</th>
<th>138.2</th>
<th>166.3</th>
<th>194.4</th>
<th>222.5</th>
<th>250.6</th>
<th>278.8</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross Count</td>
<td>41</td>
<td>40</td>
<td>36</td>
<td>22</td>
<td>3</td>
<td>1</td>
<td>5</td>
<td>2</td>
<td>150</td>
</tr>
<tr>
<td>% within Distance</td>
<td>97.6%</td>
<td>95.2%</td>
<td>85.7%</td>
<td>52.4%</td>
<td>7.1%</td>
<td>2.4%</td>
<td>11.9%</td>
<td>4.8%</td>
<td></td>
</tr>
<tr>
<td>Stop Count</td>
<td>1</td>
<td>2</td>
<td>6</td>
<td>20</td>
<td>39</td>
<td>41</td>
<td>37</td>
<td>40</td>
<td>186</td>
</tr>
<tr>
<td>% within Distance</td>
<td>2.4%</td>
<td>4.8%</td>
<td>14.3%</td>
<td>47.6%</td>
<td>92.9%</td>
<td>97.6%</td>
<td>88.1%</td>
<td>95.2%</td>
<td>55.4%</td>
</tr>
<tr>
<td>Total Count</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>336</td>
</tr>
</tbody>
</table>

Table 5-6: Drivers’ Stop/cross Decision According to Yellow Onset Distance for 45 mph

<table>
<thead>
<tr>
<th>Yellow Onset Distance</th>
<th>180.4</th>
<th>206.1</th>
<th>231.9</th>
<th>257.7</th>
<th>283.4</th>
<th>309.2</th>
<th>335.0</th>
<th>360.8</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross Count</td>
<td>41</td>
<td>35</td>
<td>26</td>
<td>24</td>
<td>15</td>
<td>4</td>
<td>4</td>
<td>9</td>
<td>158</td>
</tr>
<tr>
<td>% within Distance</td>
<td>97.6%</td>
<td>83.3%</td>
<td>61.9%</td>
<td>57.1%</td>
<td>35.7%</td>
<td>9.5%</td>
<td>9.5%</td>
<td>21.4%</td>
<td>47.0%</td>
</tr>
<tr>
<td>Stop Count</td>
<td>1</td>
<td>7</td>
<td>16</td>
<td>18</td>
<td>27</td>
<td>38</td>
<td>38</td>
<td>33</td>
<td>178</td>
</tr>
<tr>
<td>% within Distance</td>
<td>2.4%</td>
<td>16.7%</td>
<td>38.1%</td>
<td>42.9%</td>
<td>64.3%</td>
<td>90.5%</td>
<td>90.5%</td>
<td>78.6%</td>
<td>53.0%</td>
</tr>
<tr>
<td>Total Count</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>336</td>
</tr>
</tbody>
</table>

Table 5-5: Drivers’ Stop/cross Decision According to Yellow Onset Distance for 30 mph

<table>
<thead>
<tr>
<th>Yellow Onset Distance</th>
<th>82.00</th>
<th>110.1</th>
<th>138.2</th>
<th>166.3</th>
<th>194.4</th>
<th>222.5</th>
<th>250.6</th>
<th>278.8</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross Count</td>
<td>41</td>
<td>40</td>
<td>36</td>
<td>22</td>
<td>3</td>
<td>1</td>
<td>5</td>
<td>2</td>
<td>150</td>
</tr>
<tr>
<td>% within Distance</td>
<td>97.6%</td>
<td>95.2%</td>
<td>85.7%</td>
<td>52.4%</td>
<td>7.1%</td>
<td>2.4%</td>
<td>11.9%</td>
<td>4.8%</td>
<td></td>
</tr>
<tr>
<td>Stop Count</td>
<td>1</td>
<td>2</td>
<td>6</td>
<td>20</td>
<td>39</td>
<td>41</td>
<td>37</td>
<td>40</td>
<td>186</td>
</tr>
<tr>
<td>% within Distance</td>
<td>2.4%</td>
<td>4.8%</td>
<td>14.3%</td>
<td>47.6%</td>
<td>92.9%</td>
<td>97.6%</td>
<td>88.1%</td>
<td>95.2%</td>
<td>55.4%</td>
</tr>
<tr>
<td>Total Count</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>336</td>
</tr>
</tbody>
</table>

Table 5-6: Drivers’ Stop/cross Decision According to Yellow Onset Distance for 45 mph

<table>
<thead>
<tr>
<th>Yellow Onset Distance</th>
<th>180.4</th>
<th>206.1</th>
<th>231.9</th>
<th>257.7</th>
<th>283.4</th>
<th>309.2</th>
<th>335.0</th>
<th>360.8</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross Count</td>
<td>41</td>
<td>35</td>
<td>26</td>
<td>24</td>
<td>15</td>
<td>4</td>
<td>4</td>
<td>9</td>
<td>158</td>
</tr>
<tr>
<td>% within Distance</td>
<td>97.6%</td>
<td>83.3%</td>
<td>61.9%</td>
<td>57.1%</td>
<td>35.7%</td>
<td>9.5%</td>
<td>9.5%</td>
<td>21.4%</td>
<td>47.0%</td>
</tr>
<tr>
<td>Stop Count</td>
<td>1</td>
<td>7</td>
<td>16</td>
<td>18</td>
<td>27</td>
<td>38</td>
<td>38</td>
<td>33</td>
<td>178</td>
</tr>
<tr>
<td>% within Distance</td>
<td>2.4%</td>
<td>16.7%</td>
<td>38.1%</td>
<td>42.9%</td>
<td>64.3%</td>
<td>90.5%</td>
<td>90.5%</td>
<td>78.6%</td>
<td>53.0%</td>
</tr>
<tr>
<td>Total Count</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>42</td>
<td>336</td>
</tr>
</tbody>
</table>
For the 30 mph speed limit, most of stop rates with marking at eight yellow onset distances except the 82, 110.11, and 194.46 feet ones were found to be higher than those without, as shown in Figure 5-10-a. Without marking, there were 44.6% crosses and 55.4% stops; with marking, there were 38.7% crosses and 61.3% stops and no stop happened at the 82 ft yellow onset distance. On the whole, drivers tend to stop at the larger onset distances with the marking compared to the without.

Totally, for the 45 mph without marking, there were 47% crosses and 53% stops; for that with marking, there were 50% crosses and 50% stops. As shown in Figure 5-10-b, there are significant differences in the stop-go decision between with markings and without at different yellow onset distances. If the distances are smaller than 270 ft, the stop rates without marking are higher than those with marking; and if the distances are larger than 270 ft, the stop rates without marking are lower than those with marking. Generally, if drivers decide to stop when they are close to the intersection at the onset of yellow phase, it is more likely to be involved in rear-end crashes since the deceleration distance tends to be insufficient. On the other hand, if drivers decide to cross the intersection when they are far from the intersection at the onset of yellow phase, it is more likely to be involved in angle crashes since they have a higher chance of red-light running. It appears that with the help of marking information, drivers tend to get better stop/go decision: stop at farther distance and cross at shorter distances.
Figure 5-10: Stop rate without-with comparison according to yellow onset distances
5.3.5 Stopping probability analysis based on logistic regression method

In this step, to more accurately analyze drivers’ behavior at intersections, two logistic regression models for the 30mph speed limit and the 45mph speed limit are developed to predict drivers’ probability based on more independent parameters related to the driver’s stop-go decision.

Logistic regression is proper to be used in this study because the stop/go decision at intersections can be described as a typical dichotomy dependent variable, Y=1 when the driver stopped and Y=0 when the driver crossed the intersection. Logistic regression can be applied to predict a dependent variable on the basis of independence; to rank the relative importance of the independent variables; to assess interaction effects; and to understand the impact of covariate control variables. Logistic regression applies maximum likelihood estimation after transforming the dependent into a logit variable (the natural log of the dependent variable). In this way, logistic regression estimates the probability of a certain event occurring.

The probability that a driver will stop or not is modeled as logistic distribution in Equation 5-3-1:

\[
\pi(x) = \frac{e^{g(x)}}{1 + e^{g(x)}}
\]  
(5-3-1)
The Logit of the multiple logistic regression model (Link Function) is given by Equation 5-3-2:

\[ g(x) = \ln \left( \frac{\pi(x)}{1 - \pi(x)} \right) = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_3 + \ldots + \beta_n x_n \]  

(5-3-2)

where, \( \pi(x) \) is conditional probability of a red-light running crash, which is equal to the number of stops divided by the total number of stop/go observations. \( x_n \) are independent variables which can be either categorical or continuous. Both main effects and interactions can generally be accommodated. \( \beta_n \) are model coefficients, which directly determines odds ratio that drivers stop at intersections.

Five independent variables (Age, Gender, Marking, Distance, and Speed) were chosen as potential factors that might be associated with the stop probability at intersections and they are described in the Table 5-7.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Variable Description</th>
<th>Variable Coding</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age</td>
<td>Subject age</td>
<td>0=younger drivers 1=middle-age drivers</td>
</tr>
<tr>
<td>Gender</td>
<td>Subject gender</td>
<td>0=Male 1=Female</td>
</tr>
<tr>
<td>Marking</td>
<td>If there is a Pavement Marking or not</td>
<td>0=Without marking 1=With marking</td>
</tr>
<tr>
<td>Distance</td>
<td>Yellow onset distance to the intersection</td>
<td>Continuous (feet)</td>
</tr>
<tr>
<td>Speed</td>
<td>Approaching speed at onset of the yellow</td>
<td>Continuous (mph)</td>
</tr>
</tbody>
</table>
Screening all 5 given variables, Table 5-8 lists the logistic regression results of main effect models for the 30 mph and 45 mph speed limits respectively. For the 30 mph speed limit, the significant independent variables include Distance, Speed, and Marking, but the Age and Gender are not significant; for the 45 mph speed limit, only Distance and Speed are significant variables.

Table 5-8: Summary of Main Effect Logistic Regression Models

<table>
<thead>
<tr>
<th>Parameter</th>
<th>DF</th>
<th>Coefficient</th>
<th>Standard Error</th>
<th>Wald Chi-Square</th>
<th>Pr&gt; ChiSq</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a) Main effect model for the 30 mph speed limit</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intercept</td>
<td>1</td>
<td>-1.3176</td>
<td>1.6027</td>
<td>0.6758</td>
<td>0.411</td>
</tr>
<tr>
<td>Speed</td>
<td>1</td>
<td>-0.2612</td>
<td>0.0511</td>
<td>26.1688</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Distance</td>
<td>1</td>
<td>0.1933</td>
<td>0.0158</td>
<td>150.5307</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Marking</td>
<td>1</td>
<td>0.8456</td>
<td>0.291</td>
<td>8.4442</td>
<td>0.0037</td>
</tr>
<tr>
<td>(b) Main effect model for the 45 mph speed limit</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intercept</td>
<td>1</td>
<td>-5.1295</td>
<td>1.3936</td>
<td>13.5485</td>
<td>0.0002</td>
</tr>
<tr>
<td>Speed</td>
<td>1</td>
<td>-0.096</td>
<td>0.0287</td>
<td>11.2006</td>
<td>0.0008</td>
</tr>
<tr>
<td>Distance</td>
<td>1</td>
<td>0.1183</td>
<td>0.00845</td>
<td>195.9297</td>
<td>&lt;.0001</td>
</tr>
</tbody>
</table>

Based on above variables, hypothesis test with a 0.05 significance level is used to decide on the significant factors for the final models. As shown in Table 5-9, all those parameters’ P-values are less than 0.05 and there is an interaction effect found between Distance and Marking for both speed limits. The model equations are shown as following:

- For the 30 mph speed limit
  \[ g(x) = 0.8199 - 0.2766 \times \text{Speed} + 0.1618 \times \text{Distance} - 3.9918 \times \text{Marking} + 0.0992 \times \text{Distance} \times \text{Marking} \]

- For the 45 mph speed limit
\[ g(x) = -2.8981 - 0.0915 \times \text{Speed} + 0.0899 \times \text{Distance} - 7.2509 \times \text{Marking} \]
\[ + 0.0851 \times \text{Distance} \times \text{Marking} \]

Table 5-9: Summary of Final Logistic Regression Models

(a) Final model for the 30 mph speed limit

<table>
<thead>
<tr>
<th>Parameter</th>
<th>DF</th>
<th>Coefficient</th>
<th>Standard Error</th>
<th>Wald Chi-Square</th>
<th>Pr&gt; ChiSq</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>1</td>
<td>0.8199</td>
<td>1.7162</td>
<td>0.2282</td>
<td>0.6328</td>
</tr>
<tr>
<td>Speed</td>
<td>1</td>
<td>-0.2766</td>
<td>0.0526</td>
<td>27.6343</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Distance</td>
<td>1</td>
<td>0.1618</td>
<td>0.0168</td>
<td>92.2608</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Marking</td>
<td>1</td>
<td>-3.9918</td>
<td>1.7334</td>
<td>5.3031</td>
<td>0.0213</td>
</tr>
<tr>
<td>Distance* Marking</td>
<td>1</td>
<td>0.0992</td>
<td>0.0354</td>
<td>7.8394</td>
<td>0.0051</td>
</tr>
</tbody>
</table>

(b) Final model for the 45 mph speed limit

<table>
<thead>
<tr>
<th>Parameter</th>
<th>DF</th>
<th>Coefficient</th>
<th>Standard Error</th>
<th>Wald Chi-Square</th>
<th>Pr&gt; ChiSq</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>1</td>
<td>-2.8981</td>
<td>1.4076</td>
<td>4.239</td>
<td>0.0395</td>
</tr>
<tr>
<td>Speed</td>
<td>1</td>
<td>-0.0915</td>
<td>0.0282</td>
<td>10.5208</td>
<td>0.0012</td>
</tr>
<tr>
<td>Distance</td>
<td>1</td>
<td>0.0899</td>
<td>0.00947</td>
<td>90.0191</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Marking</td>
<td>1</td>
<td>-7.2509</td>
<td>1.7086</td>
<td>18.0092</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Distance* Marking</td>
<td>1</td>
<td>0.0851</td>
<td>0.0206</td>
<td>17.0743</td>
<td>&lt;.0001</td>
</tr>
</tbody>
</table>

According to the final models, not only yellow onset distances are significantly related to drivers’ stopping probability at intersections, but also the approaching speed is the other important factor that influences driver’s stop-go decision. The larger approaching speeds are, the less possibly drivers stop. For the 30 mph, the odds ratio estimator for Speed is \( \text{Exp.} (-2.766) = 0.758 \) and its interval under the 95% confidence is \([0.684, 0.841]\); without considering other factors, drivers with the larger approaching speed might be 24.2% less likely to stop at the intersection compared to those with the speed that is 1 mph smaller. For the 45 mph, the odds ratio estimator for Speed is \( \text{Exp.} (-0.0915) = 0.913 \) and its interval under the 95% confidence is \([0.863, 0.964]\); without considering other factors,
drivers with the larger approaching speed might be 13.7% less likely to stop at the intersection compared to those with the speed that is 1 mph smaller.

For the final models with Distance* Marking interaction variable, the Marking effect on the driver stop decision is correlated to the yellow onset distance. The odds ratio estimators for the intersection with a marking could be Exp. (-3.9918+0.0992*Distance) for the 30 mph and Exp. (-7.2509+0.0851*Distance) for the 45 mph times compared to that without adjusting other factors. The distance is positively related to the odds ratio estimators for the marking, as shown in Table 5-10. For the 30 mph, if the distances are shorter than 130 ft, drivers tend to cross the intersection with the marking; for the distances larger than 130 ft meters, drivers tend to stop at the intersection with the marking. For the 45 mph, if the distances are shorter than 280 ft, drivers tend to cross the intersection with the marking; for the distances larger than 280 ft, drivers tend to stop at the intersection with the marking.

Table 5-10: Interaction Effect of Yellow Onset Distance on the Marking

<table>
<thead>
<tr>
<th>Distance (ft)</th>
<th>Coefficient of Marking</th>
<th>Odds ratio estimator for Marking</th>
<th>Distance (ft)</th>
<th>Coefficient of Marking</th>
<th>Odds ratio estimator for Marking</th>
</tr>
</thead>
<tbody>
<tr>
<td>82.00</td>
<td>-1.512</td>
<td>0.221</td>
<td>180.40</td>
<td>-2.570</td>
<td>0.077</td>
</tr>
<tr>
<td>110.11</td>
<td>-0.662</td>
<td>0.516</td>
<td>206.17</td>
<td>-1.902</td>
<td>0.149</td>
</tr>
<tr>
<td>138.23</td>
<td>0.189</td>
<td>1.208</td>
<td>231.94</td>
<td>-1.233</td>
<td>0.291</td>
</tr>
<tr>
<td>166.34</td>
<td>1.039</td>
<td>2.827</td>
<td>257.71</td>
<td>-0.564</td>
<td>0.569</td>
</tr>
<tr>
<td>194.46</td>
<td>1.889</td>
<td>6.615</td>
<td>283.49</td>
<td>0.104</td>
<td>1.110</td>
</tr>
<tr>
<td>222.57</td>
<td>2.740</td>
<td>15.481</td>
<td>309.26</td>
<td>0.773</td>
<td>2.166</td>
</tr>
<tr>
<td>250.69</td>
<td>3.590</td>
<td>36.231</td>
<td>335.03</td>
<td>1.441</td>
<td>4.227</td>
</tr>
<tr>
<td>278.80</td>
<td>4.440</td>
<td>84.792</td>
<td>360.80</td>
<td>2.110</td>
<td>8.249</td>
</tr>
</tbody>
</table>
According to the previous study, the region surrounding the 50% probability of stopping has been defined as the most hazardous portion of the intersection approach. The results showed that the uncertainty distances between 20% and 80% probability of stopping are about 23 ft for the 30 mph (56 ft Vs 33 ft) and 50 ft for the 45 mph (102 ft Vs 52 ft) shorter with markings compared to without ones as shown in Figure 5-11. The analysis indicates that the marking information can help to reduce driver hesitated region to decide to stop or cross the intersection, which possibly results in higher accident rates.

(a) For the 30 mph speed limit and assuming that approaching-vehicle speed is 30 mph
Figure 5-11: Probability of stop based on the logistic regression models.

5.3.6 Brake response time

The time following the appearance of the yellow phase until the driver steps on the brake is measured as brake response time. Four independent variables (Age, Gender, Marking, and Distance) were chosen as potential factors that might have an effect on driver brake response time and the basic descriptive results are described in the Table 5-11.
Table 5-11: Descriptive Statistical Results of Brake Response Time for Age, Gender, Marking, and Distance

(a) For the 30 mph speed limit

<table>
<thead>
<tr>
<th>Variable</th>
<th>N</th>
<th>Mean</th>
<th>Std. D</th>
<th>95% C.I.</th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Younger</td>
<td>169</td>
<td>1.0609</td>
<td>.3723</td>
<td>1.0044</td>
<td>1.1175</td>
<td>.20</td>
</tr>
<tr>
<td>Middle</td>
<td>217</td>
<td>1.1330</td>
<td>.4003</td>
<td>1.0794</td>
<td>1.1865</td>
<td>.35</td>
</tr>
<tr>
<td>Gender</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Male</td>
<td>221</td>
<td>1.0902</td>
<td>.4006</td>
<td>1.0371</td>
<td>1.1433</td>
<td>.20</td>
</tr>
<tr>
<td>Female</td>
<td>165</td>
<td>1.1164</td>
<td>.3747</td>
<td>1.0589</td>
<td>1.1740</td>
<td>.23</td>
</tr>
<tr>
<td>Marking</td>
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<td></td>
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<td></td>
<td></td>
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<tr>
<td>Without</td>
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<td>.23</td>
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<td>.3875</td>
<td>1.0324</td>
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<td>.52</td>
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<td>278.80</td>
<td>77</td>
<td>1.3794</td>
<td>.5072</td>
<td>1.2643</td>
<td>1.4946</td>
<td>.23</td>
</tr>
</tbody>
</table>

(b) For the 45 mph speed limit

<table>
<thead>
<tr>
<th>Variable</th>
<th>N</th>
<th>Mean</th>
<th>Std. D</th>
<th>95% C.I.</th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Younger</td>
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<td>.9911</td>
<td>.2949</td>
<td>.9442</td>
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</tr>
<tr>
<td>Middle</td>
<td>186</td>
<td>1.0652</td>
<td>.3352</td>
<td>1.0167</td>
<td>1.1136</td>
<td>.23</td>
</tr>
<tr>
<td>Gender</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Male</td>
<td>193</td>
<td>1.0228</td>
<td>.3253</td>
<td>.9766</td>
<td>1.0690</td>
<td>.28</td>
</tr>
<tr>
<td>Female</td>
<td>147</td>
<td>1.0432</td>
<td>.3119</td>
<td>.9924</td>
<td>1.0941</td>
<td>.23</td>
</tr>
<tr>
<td>Marking</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without</td>
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<td>1.0216</td>
<td>.3298</td>
<td>.9723</td>
<td>1.0710</td>
<td>.23</td>
</tr>
<tr>
<td>With</td>
<td>166</td>
<td>1.0421</td>
<td>.3085</td>
<td>.9948</td>
<td>1.0894</td>
<td>.50</td>
</tr>
<tr>
<td>Distance</td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>180.40</td>
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<td>.9000</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>.90</td>
</tr>
<tr>
<td>206.17</td>
<td>8</td>
<td>.8125</td>
<td>.2231</td>
<td>.6260</td>
<td>.9990</td>
<td>.53</td>
</tr>
<tr>
<td>231.94</td>
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<td>.9643</td>
<td>.3446</td>
<td>.8074</td>
<td>1.1212</td>
<td>.28</td>
</tr>
<tr>
<td>257.71</td>
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<td>.2188</td>
<td>.8322</td>
<td>.9596</td>
<td>.57</td>
</tr>
<tr>
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<td>1.0261</td>
<td>.2583</td>
<td>.9576</td>
<td>1.0946</td>
<td>.50</td>
</tr>
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<td>1.0831</td>
<td>.3699</td>
<td>.9980</td>
<td>1.1682</td>
<td>.45</td>
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<tr>
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<td>1.0067</td>
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<td>.9388</td>
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<td>.23</td>
</tr>
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<td>360.80</td>
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<td>1.1022</td>
<td>.3461</td>
<td>1.0215</td>
<td>1.1830</td>
<td>.43</td>
</tr>
</tbody>
</table>
A four-factor analysis of variance (ANOVA) for each speed limit type was conducted using the general linear model procedure (GLM) of the SAS software to determine the statistical significance of these trends. For the 30 mph speed limit, the ANOVA model was significant ($P < 0.0001$) at the 0.05 level. Table 5-12 lists the ANOVA variance analysis for independent variables from SAS, which shows that distance and two-way interaction between age and gender are significant factors but marking is not under the 95% confidence level.

Table 5-12: ANOVA Variance Analysis of Brake Response Time for the 30 mph

<table>
<thead>
<tr>
<th>SOURCE</th>
<th>DF</th>
<th>TYPE III SS</th>
<th>MEAN SQUARE</th>
<th>F VALUE</th>
<th>PR&gt;F</th>
</tr>
</thead>
<tbody>
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<td>0.258735</td>
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<td>0.1539</td>
</tr>
<tr>
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<td>0.0719</td>
</tr>
<tr>
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<td>9.468899</td>
<td>1.3527</td>
<td>10.67</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Gender*Age</td>
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<td>0.958521</td>
<td>0.958521</td>
<td>7.56</td>
<td>0.0063</td>
</tr>
</tbody>
</table>

A Scheffe test for multiple comparisons on the distance factor showed that the response time for the 278.8 ft is significantly larger than those for other distances except for the 82 ft and 110.11 ft. Although there is no significant difference between the other distances, there is an obvious tendency that the response time for farther distances is larger than those for shorter ones. This tendency does make sense because drivers at larger yellow onset distance have more space and time to decide to stop or to cross the intersection. A Scheffe test on the age factor showed that the response time for the middle group is 0.072 second significantly larger than the younger group. However, the age effect is confounded by the gender. As shown in Figure 5-12, for the younger group, the response time for male drivers is less than female; for the middle group, the response time for male drivers is larger than female.
For the 45 mph speed limit, the ANOVA model was significant ($P = 0.0104$) at the 0.05 level. Table 5-13 lists the ANOVA variance analysis for significant independent variables from SAS, which shows that distance and age are significant factors but gender, marking and any two-way interactions are not under the 95% confidence level. A Scheffe test on the age factor showed that the response time for the middle group is 0.074 second significantly larger than the younger group. A Scheffe test for multiple comparisons on the distance factor did not show any significant difference in the response time among those eight levels of yellow onset distance. However, the ANOVA analysis confirmed the trend that that the response time increases as the yellow onset distances increase.
Moreover, since the result comparisons between with marking and without are not significantly different for both speed limits, the marking did not have an effect on the human factor related to driver response time.

Table 5-13: ANOVA Variance Analysis of Brake Response Time for the 45 mph

<table>
<thead>
<tr>
<th>SOURCE</th>
<th>DF</th>
<th>TYPE III SS</th>
<th>MEAN SQUARE</th>
<th>F VALUE</th>
<th>PR&gt;F</th>
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</thead>
<tbody>
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<td>Distance</td>
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<td>1.545209</td>
<td>0.220744</td>
<td>2.24</td>
<td>0.0305</td>
</tr>
</tbody>
</table>

5.3.7 Brake deceleration rate

Deceleration rates of the stopping vehicles after the yellow onset were compared between the with-experiment and without-experiments in an attempt to explore a significant change in the drivers’ behavior attributable to the presence of the markings. The deceleration rate was measured for speeds ranging from the speed of the vehicle following the appearance of the yellow phase to a speed of 5 mph. Zero mph was not used because few drivers maintained a crawling speed until they reached the stop bar, which would bias the experiment results. Four independent variables (Age, Gender, Marking, and Distance) were chosen as potential factors that might have an effect on driver brake deceleration rate and the basic descriptive results are described in the Table 5-14.
Table 5-14: Descriptive Statistical Results of Brake Deceleration Rate for Age, Gender, Marking, and Distance

(a) For the 30 mph speed limit

<table>
<thead>
<tr>
<th>Variable</th>
<th>N</th>
<th>Mean</th>
<th>Std. D</th>
<th>95% C.I.</th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Middle</td>
<td>220</td>
<td>10.1764</td>
<td>4.2548</td>
<td>9.6111 - 10.7418</td>
<td>2.24</td>
<td>23.67</td>
</tr>
<tr>
<td>Gender</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Female</td>
<td>171</td>
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<td>9.3046 - 10.4904</td>
<td>2.24</td>
<td>21.68</td>
</tr>
<tr>
<td>Marking</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With</td>
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<td>3.9143</td>
<td>9.6077 - 10.6779</td>
<td>2.24</td>
<td>23.67</td>
</tr>
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<td>Distance</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
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<td>17.1270</td>
<td></td>
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<td>17.13</td>
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<td>16.3443</td>
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<td>8.1503</td>
<td>3.1598</td>
<td>7.4560 - 8.8446</td>
<td>3.71</td>
<td>23.67</td>
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(b) For the 45 mph speed limit

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<th>Std. D</th>
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<td>12.2639</td>
<td>4.6847</td>
<td>11.5953 - 12.9325</td>
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<td>27.19</td>
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<tr>
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<td>12.0143</td>
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<td>231.94</td>
<td>21</td>
<td>17.7077</td>
<td>5.7409</td>
<td>15.2978 - 20.1176</td>
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<td>8.78</td>
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<td>9.1073 - 10.2312</td>
<td>2.72</td>
<td>17.47</td>
</tr>
</tbody>
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A four-factor analysis of variance (ANOVA) for each speed limit type was conducted using the general linear model procedure (GLM) of the SAS software to determine the statistical significance of these trends for brake deceleration rate. For the 30 mph speed limit, the ANOVA model was significant ($P < 0.0001$) at the 0.05 level. Table 5-15 lists the ANOVA variance analysis for independent variables from SAS, which shows that distance and age are significant factors but marking, gender and any two-way interactions are not under the 95% confidence level. A Scheffe test for multiple comparisons on the distance factor showed that most of deceleration rates for the larger distance are significantly less than those for the smaller distance except for the 82.00 feet. This tendency does make sense because drivers at larger yellow onset distance have more space and time to slowly decelerate their vehicles to stop safely. A Scheffe test on the age factor showed that the deceleration rate for the middle group is 0.717 $\text{ft/s}^2$ significantly larger than the younger group. However, since the result comparisons between with marking and without are not significantly different, the marking did not have an effect on the driver behavior related to the brake deceleration rate for the 30 mph limit.

### Table 5-15: ANOVA Variance Analysis of Deceleration Rate for the 30 mph Speed Limit

<table>
<thead>
<tr>
<th>SOURCE</th>
<th>DF</th>
<th>TYPE III SS</th>
<th>MEAN SQUARE</th>
<th>F VALUE</th>
<th>PR&gt;F</th>
</tr>
</thead>
<tbody>
<tr>
<td>AGE</td>
<td>1</td>
<td>49.67349</td>
<td>49.67349</td>
<td>4.72</td>
<td>0.0304</td>
</tr>
<tr>
<td>DISTANCE</td>
<td>7</td>
<td>1542.812</td>
<td>220.4017</td>
<td>20.94</td>
<td>&lt;.0001</td>
</tr>
</tbody>
</table>

For the 45 mph speed limit, the ANOVA model was significant ($P < 0.0001$) at the 0.05 level. Table 5-16 lists the ANOVA variance analysis for significant independent variables from SAS, which shows that distance and marking are significant factors but gender, age and any two-way interactions are not under the 95% confidence level. A
Scheffe test for multiple comparisons on the distance factor showed that most of deceleration rates for the larger distances are significantly less than those for the smaller distance except for the 166.34 ft. A Scheffe test on the marking factor showed that the deceleration rate without marking is $1.959 \text{ ft/s}^2$ significantly larger than that with marking.

With the marking information, the probability that drivers make a too conservative stop will decrease if they located in the downstream of marking at the onset of yellow, which contributes to the gentler deceleration rate with marking. Generally, when drivers stop at intersections, the smaller deceleration rate is, the less likely rear-end crashes happen. Therefore, the marking countermeasure may have a positive effect on improving traffic safety for rear-end crashes at signalized intersection with the higher speed limits.

Table 5-16: ANOVA Variance Analysis of Deceleration Rate for the 45 mph Speed Limit

<table>
<thead>
<tr>
<th>SOURCE</th>
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<th>TYPE III SS</th>
<th>MEAN SQUARE</th>
<th>F VALUE</th>
<th>PR&gt;F</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAKING</td>
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<td>331.6474</td>
<td>34.01</td>
<td>&lt;.0001</td>
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<td>DISTANCE</td>
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<td>2301.959</td>
<td>328.8513</td>
<td>33.73</td>
<td>&lt;.0001</td>
</tr>
</tbody>
</table>

5.3.8 Subject survey for the pavement-marking experiment

When subjects completed the formal experiments, a survey was used to gather information about their opinions of this proposed pavement marking and red-light running. Specifically, the survey is to investigate the red-light running reason and frequency of the potential violators in the real world, dilemma zone’s hazard at signalized intersections, and subjects’ attitude to the safety significance of the proposed pavement marking. The investigation form is attached as appendix A in this report.
Based on the survey results from 42 subjects, there were 90.5% drivers who admitted that they did run a red-light in the real road before. For those red-light runners, 54% drivers run a red-light at least per month and 32% drivers run a red-light at least per week. As shown in Figure 5-13, more than 54% subjects thought that red-light running problem may result from incapability of stopping during the yellow signal phase because of poor judgment and 28% subjects thought the traffic delay is an important reason. Moreover, according to the investigation results after the experiment, all of subjects gave a positive evaluation on the pavement-marking countermeasure. All of subjects thought that the marking design can help them easily make stop-go decision at signalized intersections without any confusion. 91% of the subjects agreed that the pavement marking should be applied to the real road. Two subjects (one younger male and one younger female) who did not agree with road application explained that the marking should be helpful but their stop-go decisions would rely on the traffic situation and they might still ignore the marking information to beat a red-light so they would not be delayed. Another subject suggested that it should be necessary to conduct more related researches before the road application. Moreover, several subjects reported that they used the solid lane line to make stop-go decision: they crossed when the signal turned amber and they were within the solid lane line; otherwise, they stopped at the intersection. Additionally, subjects gave a whole evaluation on fidelity of the simulator system in the questionnaire as shown in Figure 5-14. More than 70% subjects thought that the simulator fidelity is good or excellent, 14% subjects thought it need improvement, but nobody gave “poor” evaluation.
Figure 5-13: Red-light running reason

Figure 5-14: Evaluation on fidelity of the simulator system

5.4 Conclusions and Discussions

According to the result analyses of the driving simulator experiment, the pavement-marking countermeasure has a significantly positive effect on the signalized-intersection
safety. Firstly compared to regular intersections, the pavement marking could results in a 74.3 percent reduction in red-light running because of poor stop-go judgment. In comparison, the pavement marking reduced the number of occurrences where drivers chose to continue through an intersection when it was not safe to proceed compared to the without marking, and this result is correlated to the less red-light running rate with marking. Further, for those running red-light drivers, the marking tends to reduce the red-light entry time. The results may contribute to reducing the probability of angle crashes.

Secondly, logistic regression models confirmed that the marking is helpful to improve driver stop-go decision at intersections. Compared to without marking, if the drivers located near to the stop bar, drivers tend to cross the intersection with the marking; if the drivers located farther to the stop bar, drivers tend to stop at the intersection with the marking. The results showed that the uncertainty distances between 20% and 80% probability of stopping with marking are about 23 ft for the 30 mph and 50 ft for the 45 mph shorter in comparison with regular intersections. The analysis indicates that the marking information can help to reduce driver hesitated region to decide to stop or cross the intersection, which possibly results in higher accident rates.

Thirdly, it was found that for those stopping drivers, the brake deceleration rate without marking is 1.959 ft/s² significantly larger than that with marking for the higher speed limit. With the marking information, the probability that drivers make a too conservative stop will decrease if they are located in the downstream of marking at the onset of yellow,
which resulted in the gentler deceleration rate with marking. At intersections, the smaller deceleration rate may contribute to the less probability that rear-end crashes happen.

Moreover, according to survey results, all of subjects gave a positive evaluation on the pavement-marking countermeasure and nobody felt confused or uncomfortable when they made a stop-go decision with marking. In comparison between scenarios without marking and with marking, there is no significant difference found in the operation speeds and drivers brake response time, which proved that the marking has no significantly negative effect on driver behaviors at intersections.

Although, it was found that the pavement marking is useful to improve intersection safety based on the simulator test, there are still several issues such as effect of the red-light running reduction, education method, experiment design, driver attitude, and other factors, that need to be discuss if applying the marking to the real world.

Red light runners can be divided into two categories, intentional violators and unintentional violators. The pavement marking may effectively help those unintentional drivers who may be incapable of stopping for a red signal because of poor judgment by the drivers or a deficiency in the design of the intersection. The marking may not be useful for that intentional violator at all, who are most affected by enforcement countermeasures or traffic education program. However, some previous accident studies admitted a connection between red-light cameras and rear-end accidents. Some additional rear-end crashes might result from non-uniform changes in the driver behavior. If drivers
stop more often and too conservatively for red lights, they may be struck from behind by drivers not intending to stop. The pavement marking countermeasure is a low-technology and inexpensive solution to reduce the number of motorists that run red lights. Therefore, the combination of the marking and red-light cameras may be more effective for both countermeasures.

For this experiment, a simple education and training would be required for drivers to learn the basic knowledge about the purpose of marking. In the real world, the new driver may get the knowledge from license-training procedures and the licensed driving population may get to know the marking policy from media and other drivers. Therefore, there could be a shorter or longer period that the whole driving population gets used to it. However, if installing some type of warning signs beside the marking, such as a sign with word message of “if yellow prepare to stop”, that might help to reduce the learning period. In addition, if a digital clock is installed on this sign that would display how many seconds remain in the green phase before the signal turns amber, the motorist may have additional information to help him/her make better decision.
CHAPTER 6. SAFETY ISSUES OF LEFT TURN SIGHT DISTANCE

At signalized-intersections where protected left turn signals are not provided, simultaneous opposing left-turn vehicles attempting the turn maneuver may cause insufficient sight distance for each other (see Figure 6-1). The sight distance problem can contribute to not only a serious safety problem because of driver’s misjudging gaps available in the opposing through traffic, but also increase intersection delay. This delay is created because drivers need more time to make sure that the opposing through lanes are clear.

![Figure 6-1: Left turn sight distance problem at a signalized intersection](image)

A number of related studies had shown that sight distance problems at intersections usually result in a higher accident rate (Mitchell, 1972; Hanna, 1976; David, 1979).
McCoy et al. (1992) reported that in California, signalized intersections with opposing left turn lanes were found to have significantly more accidents than intersections without opposing left turn lanes, which were attributed primarily to sight distance obstructions caused by opposing left turn vehicles. The 2001 AASHTO manual pointed out that the typical poor visibility of opposing through traffic usually occurs at signal intersections with medians wider than 18 ft and recommended two improvement methods, parallel offset left-turn lanes and tapered offset left-turn lanes. However, it did not provide the specific design guideline; nor did it present the related geometric design model. Based on literature research, currently there is a lack of literatures related to sight distance models for left-turn traffic at intersections especially with curved approaches.

Previous studies had developed a sight distance computation model for intersections located on linear major road segments (see Figure 6-2-a). However, if the minor road intersects a curved major road, those models are not valid. The curve presence may contribute to or mitigate the sight distance problem, which depends on whether the driver is making left-turn toward the outside or the inside of the curve. As shown in Figure 6-2-b, the sight distance for the left-turners toward the outside of the curve is very short. On the other hand, for left-turners toward the inside of the curve, the sharpness of the curve can even result in unrestricted sight distance, since the left-turners benefit from a left-turn lane offset toward the coming traffic. Furthermore, when intersections are located at a major highway that is close to the tangent points of the linear and curved segments, the sight distance calculation models could be more complicated than the linear or the curve models. The combination of the curve and linear segments may result in four different
intersection configurations: linear approach leading a curve segment for left turn toward outside of the curve (see Figure 6-2-c), linear approach leading a curve segment for left turn toward inside of the curve (see Figure 6-2-d), curve approach leading a linear segment for left turn toward outside of the curve (see Figure 6-2-e), and curve approach leading a linear segment for left turn toward inside of the curve (see Figure 6-2-f).

Figure 6-2: Intersection configurations and left-turn sight distance
For more comprehensive and practical application to intersection design, this chapter presented a series of left-turn sight distance models to compute left-turn sight distances for different signalized-intersection configurations. They include intersections located on a linear segment, a curved segment, a linear segment but leading a curve one, a curve segment, and a curve segment but leading a linear one. The models can be used to identify a better intersection location along the major road when a curve exists, layout intersection geometric design, or evaluate the sight distance problem of an existing intersection configuration to ensure safe left-turn maneuvers by drivers.

### 6.1 Background -- Required left-turn sight distance

Procedures for determining appropriate intersection sight distance based on time gap acceptances are provided by AASHTO (5) in its revised version (2001) for various levels of intersection control and the maneuvers to be performed. There are six scenarios (A to F) in the manual, and the one that pertains to this paper is defined as Case F, left turns from the major road. The 2001 AASHTO manual recommended that the required intersection sight distance for left turns from a major road should be based on critical gap acceptance as shown in Equation 1, which is equal to the distance traversed at the design speed of the major road in the critical gap duration time accepted by the left turner.

\[
\text{ISD} = 1.47V^*G
\]  

(6-1-1)
Where: ISD = required sight distance for left turn from the major road (ft),

V = major road design speed (mph), and

G = critical gap size for left turn from the major road (sec).

In the 2001 AASHTO manual, 5.5 sec is recommended as a critical gap accepted by left turning passenger cars from the major road. For left turning vehicles that cross more than one opposing lane, 0.5 sec need be added for each additional lane to be crossed.

6.2 Geometric Model of Sight Distance Calculation for Linear-approach Intersections

6.2.1 Geometric model for parallel offset left turn lanes

Intersection Geometric Features Assumed for Left Turn Maneuver

A typical 90-degree angle of intersection with four level approaches is shown in Figure 6-3. For the major road divided by a median, there are one left turn lane and three through lanes with 12 ft lane width in both sides of the intersection. For the minor road divided by a median, three lanes with 11 ft lane width are assumed. There are also 10 ft pedestrian walk crossings to cross the major approaches and the distance from the stop bar of the major road to the edge of the minor road is supposed to be 25 ft (10’+ 10’+5’). In the opposing left turn lanes, both vehicles try to make left turn at the same time and the sight views of drivers are blocked by each other.
Figure 6-3: Intersection geometric features for parallel left turn lanes and available sight distance

**Formula to Calculate Available Sight Distance for Parallel Left Turn Lanes**

According to the definition used by McCoy et al. (4), the available sight distance is the distance from the left-turn driver’s eye to the point at which his/her line of sight intersects the centerline of the near opposing through lane. As shown in Figure 6-3 and Equation 6-2-2, the available sight distance is:

\[
SD = W + Y
\]  

(6-2-2)

Where

\[
SD = \text{available sight distance (ft)},
\]
\( W = \) the distance from the left-turn driver’s eye to the stop bar of the opposing through lanes (ft), and 
\( Y = \) the distance from the stop bar of the opposing through lanes to the front of opposing through vehicle (ft).

According to the similar triangle rule, we get equation 6-2-3:

\[
\frac{Y}{W} = \frac{B}{A} \quad \Rightarrow \quad Y = \frac{B \cdot W}{A} \tag{6-2-3}
\]

Where

\( A = \) the distance from the left-turn driver’s eye to the right edge of the opposing left turn vehicle (ft), and
\( B = \) the distance from the right edge of the opposing left turn vehicle to the centerline of the nearest opposing through lane (which is also the centerline of the opposing through vehicle (ft),

Combining equation 6-2-2 and 6-2-3, get equation 6-2-4:

\[
SD = W + \frac{B \cdot W}{A} \tag{6-2-4}
\]

The following equations 6-2-5, 6-2-6, and 6-2-7 show how to calculate the term A, B, and W, separately.
As shown in Figure 6-4, where,

\[ A = n + g + e - (m - n - g - V_w) \]
\[ A = 2n + 2g + e + V_w - m \]
\[ A = 2n - m + 12.5 \] \hspace{1cm} (6-2-5)
\[ B = L_t / 2 + (m - n - g - V_w) \]
\[ B = m - n - 3 \] \hspace{1cm} (6-2-6)
\[ W = V_f + D \]
\[ W = 8 + D \] \hspace{1cm} (6-2-7)

n = the width of the median nose (ft),
g = the distance from the left side of the left-turn vehicle to the left lane line (ft) [2 ft can be assumed for the design purpose according to the AASHTO],
e = the distance from the eye of the driver to the left side of the vehicle (ft) [1.5 ft is assumed],
\[ L_t \] = the width of the opposing through lane (ft) [12 ft is assumed],
m = the width of the median (ft),
\[ V_w \] = the width of the opposing left turn vehicle (ft) [7 ft is assumed],
\[ V_f \] = the distance from the eye of the driver to the front of the vehicle (ft) [8 ft is assumed according to the AASHTO], and
D = the distance between stop bars of the opposing left lanes, which is composed of the width of pedestrian corridors and the width of the minor road (ft).
Substituting equations 6-2-5, 6-2-6, and 6-2-7 into equation 6-2-4, a detailed available sight distance model is shown in equation 6-2-8. Equation 6-2-9 is a simplified sight-distance model using the assumed parameter values and considering intersection features, as following:

$$SD = V_f + D + \frac{(L_1/2 + m - n - g - V) \cdot (V_f + D)}{2n + 2g + e + V - m}$$  \hspace{1cm} (6-2-8)

$$SD = 8 + D + \frac{(m - n - 3) \cdot (8 + D)}{2n - m + 12.5}$$  \hspace{1cm} (6-2-9)

**Evaluation of Sight Distance Problem for Left Turn**

From equation 6-2-9, it is shown that the main factors of available sight distance are median width (m), the width of the median nose (n), and the distance between stop bars of the opposing left turn lanes (D). Of those, term D is positively related to the sight distance. Simply put, the wider the intersection, the larger sight distance available for left
turners. To emphasize the severity of the sight distance problem, a wider intersection is conservatively assumed here, as shown in the figures 6-3 and 6-4, which is equal to the width of the two pedestrian corridors plus the width of the minor road that contains three lanes, totaling 83 ft.

Terms of m and n are correlated to each other. For the traditional intersection design, especially with the median width less than 19.5 ft, there is no offset method used for opposing left turn lanes, so the median width is equal to the width of the median nose plus the left turn lane width. Table 6-1 shows a series of results of available sight distance (ASD) according to equation 9, supposing that left turn lane width (LW) is 12 ft. For the 12 ft median that composing of only left turn lane, the available sight distance is 1729 ft and there is no sight distance problem. For the 14 ft median, it can provide 419 ft sight distance, but according to the AASHTO design criteria (5), 445 ft design value is required for 55 mph design speed. For the 16 ft median, 273 ft sight distance is available, but 285 ft design value is required for 35 mph design speed. When the median is 20 ft, the available sight distance is only 187 ft, which cannot support intersections that have design speeds higher than 20 mph. According to equation 6-2-1, the corresponding major road design speed can be derived from those available sight distances, for which left turners can safely cross the opposing through traffic (see Figure 6-5). It shows that the wider the median size, the lower design speeds provided. When the median width is 13.5 ft, the major road speed should be no more than 60 mph. As the median width increases to 20 ft, the safe major road speed decreases to 23 mph.
Table 6-1: Calculated Available Sight Distance for Traditional Parallel Opposing Left Turn Lanes

<table>
<thead>
<tr>
<th>m</th>
<th>n</th>
<th>LW</th>
<th>D</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>(ft)</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(ft)</td>
<td>(ft)</td>
</tr>
<tr>
<td>12</td>
<td>0</td>
<td>12</td>
<td>83</td>
<td>1729</td>
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<td>217</td>
</tr>
<tr>
<td>19</td>
<td>7</td>
<td>12</td>
<td>83</td>
<td>200</td>
</tr>
<tr>
<td>20</td>
<td>8</td>
<td>12</td>
<td>83</td>
<td>187</td>
</tr>
</tbody>
</table>

Figure 6-5: Relationship between safe major road design speed and median width

Therefore, compared to the required sight distances for left turn from AASHTO design criteria, for traditional left turn lane design, the available sight distance may be insufficient even to 14 ft narrow median for 55 mph major-road design speed. The sight distance calculation model demonstrated that the left turn sight distance problem can also occur at medians narrower than the 18 ft figure documented by AASHTO.
Sight Distance Improvement by Offsetting Left Turn Lanes

The parallel offset design for left turn lanes is designed to improve the sight distance because there are dividers in both sides of the left lane. As shown in Figure 6-6, one is narrowed median nose in the left side of the lane, and the other is a concrete island or pavement making in the right side. Offset value (O) is defined as the distance from the outer edge of the left turn lane to the inner edge of the opposing left turn lane. Thus, the median width (m) is equal to the width of the median nose (n) plus the left turn lane width (LW) and the right divider (r). So the m term is equal to n + LW + r and the n is equal to O + r. Substituting term O and r into equation 6-2-9, equation 6-2-10 can be used to evaluate sight distance improvement by offsetting left turn lanes.

\[
SD = 8 + D + \frac{(m - O - r - 3) \cdot (8 + D)}{2O + 2r - m + 12.5}
\]  

(6-2-10)

Figure 6-6: Parameter descriptions for offset parallel left turn lanes
The value of offset is negatively related to the sight distance. The smaller the O and the more opposing left turn lanes move toward each other, the larger the sight distance available. Joshua (7) indicated that the minimum value of the offset can be zero feet, but cannot be a negative value in practical design, which would result in unsafe conditions, although negative offset can create unrestricted sight distance. If assuming that the width of the left turn lane (LW) is 12 ft and D is equal to 83 ft as the intersection geometric features were described before, a series of sight distances can be calculated according to the different offset values (see Table 6-2). When offset is 0 to 1 ft, the effect of sight distance improvement is very apparent, which can provide major road design speed higher than 70 mph, as shown in Figure 6-7. For the 2 ft offset, the sight distance cannot provide design speed higher than 65 mph. For the 3 ft offset, the benefit from the offset becomes comparatively weak and the provide design speed cannot exceed 50 mph. In Table 6-2, the related widths of dividers (n and r) in both sides of the left turn lane are also listed as the references for intersection design. For other concrete intersection geometric features, different values of m, r, O, and D can be substituted into equation 10 to search the proper offset and create the sufficient sight distance for the major road design speed.
Table 6-2: Calculated Sight Distance for Parallel Offset Opposing Left Turn Lanes

<table>
<thead>
<tr>
<th>Offset (O) = 0 ft, n = r</th>
<th>Offset (O) = 1 ft, n = r + 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>m</td>
</tr>
<tr>
<td>(ft)</td>
<td>(ft)</td>
</tr>
<tr>
<td>1</td>
<td>14</td>
</tr>
<tr>
<td>1.5</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>16</td>
</tr>
<tr>
<td>2.5</td>
<td>17</td>
</tr>
<tr>
<td>3</td>
<td>18</td>
</tr>
<tr>
<td>3.5</td>
<td>19</td>
</tr>
<tr>
<td>4</td>
<td>20</td>
</tr>
<tr>
<td>Offset (O) = 2 ft, n = r + 2</td>
<td>Offset (O) = 3 ft, n = r + 3</td>
</tr>
<tr>
<td>R</td>
<td>m</td>
</tr>
<tr>
<td>(ft)</td>
<td>(ft)</td>
</tr>
<tr>
<td>0</td>
<td>14</td>
</tr>
<tr>
<td>0.5</td>
<td>15</td>
</tr>
<tr>
<td>1</td>
<td>16</td>
</tr>
<tr>
<td>1.5</td>
<td>17</td>
</tr>
<tr>
<td>2</td>
<td>18</td>
</tr>
<tr>
<td>2.5</td>
<td>19</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
</tr>
</tbody>
</table>

Note: (--) means that data are not applicable.

Figure 6-7: Effect of sight distance improvement of offset parallel left turn lanes
6.2.2 Geometric model for tapered offset left turn lanes

Formula to Calculate Available Sight Distance for Tapered Left Turn Lanes

Till now, there is no related literature available about a geometric model to analyze and evaluate the effect of sight distance improvement by the method of tapered offset left turn lanes. In fact, the model for tapered offset is a little different from the parallel one, since the blockage points of driver’s view may shift from the right front corner of the opposing left turn vehicle to the right back corner due to the rotation of the vehicle’s position. This concept is illustrated in Figure 6-8. As far as this model is concerned, it can still be resolved by the similar triangle rule to get equation 11, if the right front corner of the opposing left turn vehicle blocks driver’s view, or equation 12, if the right back corner does.

\[
SD = W + \frac{B_1 \cdot W}{A - B_1} \quad (6-2-11)
\]

\[
SD = W + V_L + \frac{B_2 \cdot (W + V_L)}{A - B_2} \quad (6-2-12)
\]

As shown in Figure 6-8, where

- \(SD\) = available sight distance (ft),
- \(W\) = the distance from the left-turn driver’s eye to the stop bar of the opposing through lanes (ft),
- \(V_L\) = the length of the opposing left turn vehicle (ft) [20 ft is assumed],
A = the distance from the left-turn driver’s eye to the centerline of the nearest opposing through lane (which is also the centerline of the opposing through vehicle (ft),

B₁ = the distance from the right front corner of the opposing left turn vehicle to the centerline of the nearest opposing through lane (which is also the centerline of the opposing through vehicle (ft), and

B₂ = the distance from the right back corner of the opposing left turn vehicle to the centerline of the nearest opposing through lane (which is also the centerline of the opposing through vehicle (ft)

Figure 6-8: Intersection Geometric Features for tapered left turn lanes and available sight distance

For the above equations 6-2-11 and 6-2-12, it is necessary to note that the term W, the sight distance in advance of the opposing left-turn vehicle, can be approximately
calculated by equation 6-2-7, since no more than 1 ft error of W due to the very small angle rotation of the left turn vehicles have a negligible effect on sight distance calculation. For the same reason, $V_L$ can be approximately assumed as the length of the opposing left turn vehicle. For the other terms, the following equations 6-2-13, 6-2-14, and 6-2-15 show how to calculate $A$, $B_1$, and $B_2$.

$$A = m + L_r/2 - d$$

$$T = (m - n)/\cos\alpha$$

$$b_1 = V_f * \text{tg}\alpha$$

$$n = m - S * \text{tg}\alpha$$

$$d = (T - g - e - b_1) * \cos\alpha$$

$$A = m + L_r/2 - d$$

$$A = m + L_r/2 - [S * \text{tg}\alpha / \cos\alpha - g - e - V_f * \text{tg}\alpha] * \cos\alpha \quad (6-2-13)$$

$$B_1 = L_i/2 + d_1$$

$$T = (m - n)/\cos\alpha$$

$$d_1 = (T - g - V_w) * \cos\alpha$$

$$B_1 = L_i/2 + [S * \text{tg}\alpha / \cos\alpha - g - V_w] * \cos\alpha \quad (6-2-14)$$

$$B_2 = L_i/2 + d_2$$

$$b_2 = V_L * \text{tg}\alpha$$

$$d_2 = (T - g - V_w - b_2) * \cos\alpha$$
\[ B_z = \frac{L_t}{2} + \left[ S \cdot \tan \alpha / \cos \alpha - g - V_w - V_L \cdot \tan \alpha \right] \cdot \cos \alpha \]  

(6-2-15)

All above parameters are shown in Figure 6-9, where

\( n \) = the width of the end of the median nose (ft),
\( m \) = the width of the median (ft),
\( g \) = the distance from the left side of the left-turn vehicle to the left lane line (ft) [2 ft can be assumed for the design purpose according to the AASHTO],
\( e \) = the distance from the eye of the driver to the left side of the vehicle (ft) [1.5 ft is assumed],
\( L_t \) = the width of the opposing through lane (ft) [12 ft is assumed],
\( V_w \) = the width of the opposing left turn vehicle (ft) [7 ft can be assumed for the design purpose according to the AASHTO],
\( V_L \) = the length of the opposing left turn vehicle (ft) [20 ft is assumed],
\( V_f \) = the distance from the eye of the driver to the front of the vehicle (ft) [8 ft is assumed according to the AASHTO],
\( S \) = Storage length of left turn lane (ft), and
\( \alpha \) = Taper Angle (degree)
Substituting equations 6-2-13, 6-2-14, 6-2-15 and assuming values into equations 6-2-11 and 6-2-12, equations 6-2-16 and 6-2-17 can be derived to calculate the available sight distance for tapered offset left turn lanes. Equation 6-2-16 is applied to the case that the right front corner of the opposing left turn vehicle blocks driver’s view, and equation 6-2-17 is applied to the case that the right front corner of the opposing left turn vehicle blocks driver’s view.

\[
SD = V_f + D + \frac{(L_t / 2 + S \cdot \tan \alpha - g \cdot \cos \alpha - V_w \cdot \cos \alpha) \cdot (V_f + D)}{m - 2S \cdot \tan \alpha + 2g \cdot \cos \alpha + e \cdot \cos \alpha + V_w \cdot \cos \alpha + V_f \cdot \sin \alpha}
\]
As shown in Figure 6-8, $\beta$ is defined as the angle between the driver’s sight line passing the right front corner of the opposing left turn vehicle and the parallel line to the major road. If $\beta \geq \alpha$, it can be concluded that the right front corner of the opposing left turn vehicle blocks driver’s view; otherwise ($\beta < \alpha$), the right back corner of the opposing left turn vehicle blocks driver’s view. Since approximately $\tan \beta = (A - B_t)/W$, values of $\beta$ and $\alpha$ can be compared to check which one of equation 16 and 17 should be used.

**Sight Distance Improvement by Tapered Left Turn Lanes**

Evaluation on tapered offset model only focus on medians wider than 18 ft, because the taper angle for the narrow median would be too small and it is adverse to traffic operation. In the following case, the same value as the parallel offset case, 83 ft is assumed for the term $D$, the same value as the parallel offset case, and 250 ft is assumed for the storage length, $S$. According to the calculation results using equation 16 (since $\beta < \alpha$), Table 6-3 shows a sequence of sufficient sight distance for larger medians.
with tapered offset left turn lanes. For 18-23 ft medians, 4-degree taper angle is appropriate to create adequate sight distance even for 80 mph major road speed. For 24-27 ft medians, 4.5-degree taper angle is needed. For 28-30 ft medians, at least 5-degree taper angle should be used. It can be concluded that, as the sight distance problem deteriorates with the increase of the median width, the larger taper angle need be correspondent.

Table 6-4 shows the sensitivity analysis of relationship between taper angles and available sight distances. By holding the value of \( m \) at 30 ft, the value of \( S \) at 250 ft, and the value of \( D \) at 83 ft, if \( \alpha \) is gradually increased by 0.5-degree, the corresponding available sight distances greatly grow bigger, especially when \( \alpha \) is larger than 4.5 degree. Another sensitivity analysis tested that storage length \( S \) also contributes to the larger sight distance, as shown in table 6-4. By holding \( m \) at 30-ft, \( D \) at 83 ft, and \( \alpha \) at 5-degree, if increasing \( S \) with 10 ft, available sight distance will extend very rapidly, especially when \( S \) exceeds 230 ft. In Tables 6-3 and 6-4, most analyses of sight distance for tapered offset left turn lanes involved equation 6-2-17. The reason is that the assumed intersection width (83 ft) is very wide, which causes a very small \( \beta \), even a negative value due to the taper effect. If in some cases, \( D \) is comparatively smaller, equation 6-2-16 may be used more to calculate sight distance. Therefore, using these models, it is possible for traffic engineers to layout the tapered offset lanes to satisfy the sight distance requirement of left turners, through balancing the relationship between parameters, \( m, S, D, n, \) and \( \alpha \).
Table 6-3: Suggested Taper Angle Corresponding to Median Width for Sufficient Sight Distance

<table>
<thead>
<tr>
<th>m</th>
<th>S</th>
<th>D</th>
<th>n</th>
<th>α</th>
<th>β</th>
<th>SD</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.0</td>
<td>250.0</td>
<td>83.0</td>
<td>0.5</td>
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<td>-2.5</td>
<td>Unlimited</td>
</tr>
<tr>
<td>19.0</td>
<td>250.0</td>
<td>83.0</td>
<td>1.5</td>
<td>4.0</td>
<td>-1.8</td>
<td>Unlimited</td>
</tr>
<tr>
<td>20.0</td>
<td>250.0</td>
<td>83.0</td>
<td>2.5</td>
<td>4.0</td>
<td>-1.2</td>
<td>Unlimited</td>
</tr>
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<td>4.0</td>
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<td>3278.8</td>
</tr>
<tr>
<td>22.0</td>
<td>250.0</td>
<td>83.0</td>
<td>4.5</td>
<td>4.0</td>
<td>0.0</td>
<td>1108.1</td>
</tr>
<tr>
<td>23.0</td>
<td>250.0</td>
<td>83.0</td>
<td>5.5</td>
<td>4.0</td>
<td>0.7</td>
<td>702.6</td>
</tr>
<tr>
<td>24.0</td>
<td>250.0</td>
<td>83.0</td>
<td>4.3</td>
<td>4.5</td>
<td>-1.4</td>
<td>Unlimited</td>
</tr>
<tr>
<td>25.0</td>
<td>250.0</td>
<td>83.0</td>
<td>5.3</td>
<td>4.5</td>
<td>-0.8</td>
<td>5574.6</td>
</tr>
<tr>
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<td>250.0</td>
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<td>6.1</td>
<td>5.0</td>
<td>-1.6</td>
<td>Unlimited</td>
</tr>
<tr>
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<td>83.0</td>
<td>7.1</td>
<td>5.0</td>
<td>-1.0</td>
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</tr>
<tr>
<td>30.0</td>
<td>250.0</td>
<td>83.0</td>
<td>8.1</td>
<td>5.0</td>
<td>-0.4</td>
<td>1769.8</td>
</tr>
</tbody>
</table>

Table 6-4: Relationship between the Taper Angle and Available Sight Distance

<table>
<thead>
<tr>
<th>m</th>
<th>S</th>
<th>D</th>
<th>n</th>
<th>α</th>
<th>β</th>
<th>SD</th>
</tr>
</thead>
<tbody>
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<td>3.0</td>
<td>10.4</td>
<td>146.1</td>
</tr>
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<td>3.5</td>
<td>7.7</td>
<td>181.4</td>
</tr>
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<td>30.0</td>
<td>250.0</td>
<td>83.0</td>
<td>12.5</td>
<td>4.0</td>
<td>5.1</td>
<td>254.7</td>
</tr>
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<td>10.3</td>
<td>4.5</td>
<td>2.4</td>
<td>427.5</td>
</tr>
<tr>
<td>30.0</td>
<td>250.0</td>
<td>83.0</td>
<td>8.1</td>
<td>5.0</td>
<td>-0.4</td>
<td>1769.8</td>
</tr>
<tr>
<td>30.0</td>
<td>250.0</td>
<td>83.0</td>
<td>5.9</td>
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<td>-3.1</td>
<td>Unlimited</td>
</tr>
<tr>
<td>30.0</td>
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<td>6.0</td>
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<tr>
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<td>250.0</td>
<td>83.0</td>
<td>1.5</td>
<td>6.5</td>
<td>-8.5</td>
<td>Unlimited</td>
</tr>
</tbody>
</table>

6.3 Geometric Model of Sight Distance Calculation for Intersections with Curve Approaches

For sight obstruction by apposite left-turn vehicles, the basic traffic scenario at an intersection on a horizontal curve is similar to that at an intersection with linear major approaches, as shown in Figure 6-10-a and 6-10-b. The curve scenario may have an increased probability of sight blockage, especially for the driver making left-turn toward
outside of the curve into the minor road approach. The likelihood and severity of this problem will increase with the sharpness of the curve. As shown in the figure 6-10-a, the sight distance for the left-turners toward the outside of the curve is very short. On the other hand, for left-turners toward the inside of the curve, the sharpness of the curve can mitigate the vision problem and even contribute to unrestricted sight distance, since the left-turners benefit from a left-turn lane offset toward the coming traffic. In the figure 6-10-a, the sight line doesn’t intersect with the route of through vehicles in the nearest coming through lane, which mean the sight distance is unrestricted. However, at a linear-approach intersection as shown in the figure 6-10-b, the available sight distances for both opposite left-turners are same, which are related the median width of major approaches. Therefore, if an intersection is located on a curve major road, the left-turn sight distance problem maybe becomes more complex, which is needed to be evaluated by developing special geometric models.

a. Intersection with linear major road approaches
Figure 6-10: Comparison of sight obstructions between a linear-approach intersection and a curve-approach one

6.3.1 Intersection geometric features

A four-approach intersection located on a typical horizontal curve major road is shown in Figure 6-11. Since circular curves are used most often for horizontal curves, because of their simplicity and ease of design, the sight distance models developed in this paper focus on intersections with a single circular curve only. For the curved major road divided by a median, there are one left-turn lane and two directional through lanes on both sides of the median, and each lane is 12 ft wide. It is assumed that the concrete geometric features for both curved major-approaches of the intersection are the same as each other, including median width, median nose width, and left-turn lane width. Also,
the intersection design is assumed that the far left edge of the median curb is aligned with the opposing left-turn lane line in the same radius curve. For the undivided linear minor road, two 12 ft lanes are assumed. There are also 10 ft pedestrian walk crossings to cross the major approaches and the distance from the stop bar of the major road to the edge of the minor road is assumed to be 25 ft (10’ + 10’+5’). The stop bars of major approaches are designed to be parallel to the minor road direction.

Figure 6-11: Basic descriptions of parameters to calculate sight distance for left-turn maneuver toward the outside of the curve
6.3.2 Model for left-turn maneuver toward outside of the curve

According to the definition used by McCoy et al. (1992), the available sight distance (SD) is the distance from the left-turn driver’s eye to the point at which his/her line of sight intersects the centerline of the near opposing through lane. For the curve road, it should be the curve length of the centerline in the near opposing through lane along which the opposing through vehicles will traverse (see Figure 2). This definition is documented in the 2001 AASHTO manual. The geometric features of the intersection are shown in Figure 6-11.

All related parameters to calculate the sight distance for left-turn traffic toward outside of the curve are defined in Figure 6-11. Based on the simple geometric rules, the basic sight distance model is shown in following Equations 6-3-2, 6-3-3, and 6-3-4:

\[
SD = R_3(\Delta + \delta) \tag{6-3-2}
\]

\[
\Delta = \alpha - \beta \tag{6-3-3}
\]

Where, \( \alpha = \arcsin\left(\frac{X + D_1}{R_1}\right) \),

\[
\beta = \arcsin\left(\frac{X - D_2}{R_2}\right)
\]

\[
\delta = \Omega - \gamma \tag{6-3-4}
\]
Where, $\Omega = \arcsin\left(\frac{R_1 \sin \Delta}{L_1}\right)$

$\gamma = \arcsin\left(\frac{R_1 R_2 \sin \Delta}{L_1 R_3}\right)$

$L_1 = \sqrt{R_1^2 + R_2^2 - 2R_1 R_2 \cos \Delta}$

As shown in Figure 6-11, where,

SD = available sight distance (ft),

$\Delta$ = the angle between the curve radius to the left-turner’s eye and the curve radius to the right front corner of the opposing left-turn vehicle (radian),

$\alpha$ = the angle between the curve radius to the left-turner’s eye and the parallel line to the minor road (radian),

$\beta$ = the angle between the curve radius to the right front corner of the opposing left-turn vehicle and the parallel line to the minor road (radian),

$\delta$ = the angle between the curve radius to the right front corner of the opposing left-turn vehicle and the curve radius to the point at which left-turner’s line of sight intersects the centerline of the near opposing through lane (radian),

$\Omega$ = the angle between the curve radius to the right front corner of the opposing left-turn vehicle and left-turner’s sight line (radian),

$\gamma$ = the angle between the curve radius to the point at which left-turner’s line of sight intersects the centerline of the near opposing through lane and left-turner’s sight line (radian),
1D = the distance from the left-turn driver’s eye to the centerline of the minor road (ft),

D₂ = the distance from the right front corner of the opposing left-turn vehicle to the centerline of the minor road (ft),

X = the distance from the curve center point to the centerline of the minor road (ft),

L₁ = the distance from the left-turn driver’s eye to the right front corner of the opposing left-turn vehicle (ft),

R = the curve radius to the inside edge of the median (ft),

R₁ = the curve radius to the left-turner’s eye (ft),

R₂ = the curve radius to the right front corner of the opposing left-turn vehicle (ft),

R₃ = the curve radius to the point at which left-turner’s line of sight intersects the centerline of the near opposing through lane (ft),

Based on Equations 6-3-2, 6-3-3, and 6-3-4, the available sight distance can be calculated by D₁, D₂, X, R₁, R₂ and R₃. Of those parameters, D₁, D₂ and X are fixed since they represent unique features of the intersection; R₁, R₂ and R₃ can be calculated by Equations 5, 6, and 7, respectively.

\[ R₁ = R + m − n − g − e \]  \hspace{1cm} (6-3-5)

\[ R₂ = R + n + g + V_w \]  \hspace{1cm} (6-3-6)
As shown in Figure 6-12, where,

\[
R_3 = R + m + \frac{L_t}{2}
\]  

(6-3-7)

\[
m = \text{the width of the median (ft)},
\]
\[
n = \text{the width of the median nose (ft)},
\]
\[
g = \text{the distance from the left side of the left-turn vehicle to the left lane line (ft) [2 ft can be assumed for the design purpose according to the AASHTO]},
\]
\[
e = \text{the distance from the eye of the driver to the left side of the vehicle (ft) [1.5 ft is assumed for passenger car]},
\]
\[
V_w = \text{the width of the opposing left-turn vehicle (ft) [7 ft is assumed]},
\]
\[
L_t = \text{the width of the opposing through lane (ft) [12 ft is assumed]},
\]

Figure 6-12: Geometric features of intersection with horizontal curve approaches
According to the concrete geometric features of the intersection and appropriate assumed values for left-turning maneuver, the available sight distance is dependent on median width (m), the width of the median nose (n), the curve radius (R), and terms $D_1, D_2$ and $X$, since the other terms are constant values. The stepwise calculation relationships between the sight distance and all parameters are shown in Figure 6-13.

![Figure 6-13: Calculation relationships between the sight distance and all parameters](image)

Of those terms, m and n are correlated. For the traditional intersection design, especially with the median width less than 19.5 ft, there is no offset method used for opposing left-turn lanes, so the median width (m) is equal to the width of the median nose (n) plus the left-turn lane width. Supposing that left-turn lane width on the major road is 12 ft, the n is equal to m-12.
According to the Equation 6-2-1, the available sight distances can be translated into safe major road design speeds. Figure 6-14 shows a sensitive analysis of median width and curve radius on sight distance and the major road design speed for left-turn maneuver toward the outside of the curve, if holding other parameters, X=0 ft, D₁=45 ft, and D₂=37 ft, and only increasing the radius from 500 ft to 12000 ft. The figure indicates that: 1) the median width m is negatively related to the sight distance, while the curve radius R is positively related to the sight distance; 2) the sight distance is more sensitive to curve radius for the narrower medians than that for the wider medians, since the former curves in the figure are steeper than the latter ones; 3) as the curve radius increases, the increasing rate of the sight distance decreases; and 4) the sensitivity of the sight distance to the median width increase with the increment of the curve radius, which means that curve radius is more important factor to the sight distance for the relatively sharper road curve, while median width is more important factor to that for the relatively flatter road curve.
Figure 6-14: Sensitive analysis of sight distance for left-turn maneuver toward the outside of the curve

The figure also illustrates that, for the median widths less than 20 ft, almost all of the safe major road speeds are less than 55mph. Even for the 12 ft median that is composed of only one left-turn lane, it still cannot satisfy sight distance requirement that is 445 ft for the 55 mph design speed according to the AASHTO criteria unless the curve radius is larger than 12000 ft. If the major road curve is relatively sharper (R less than 2000 ft), the available sight distance are less than 230 ft, which can not support 30 mph major road design speed. Obviously curved intersections present more serious sight distance problem for unprotected left-turn traffic toward outside of the curve than linear type ones that had been studied by prior researchers.
6.3.3 Model for left-turn maneuver toward inside of the curve

Figure 6-15 shows all basic parameters to calculate sight distance for left-turning toward inside of the curve. However, for this situation, it is very possible that there is no sight distance problem because of the geometry of the site, as shown in the figure. Only if the left-turner’s eyesight intersects the centerline of the near opposing through lane, the opposing vehicle can be a potential sight obstruction. Therefore, the first step for the model is to check if the opposing left-turn vehicle is a sight obstruction. Comparison between the radius \( R_3 \) of the centerline of the near opposing through lane and the distance \( T \) from the curve center to the drivers sight line can be used to check that, as shown in the figure.
Figure 6-15: Basic descriptions of parameters to calculate sight distance for left-turn maneuver toward the inside of the curve

*Step 1: Check if there is possible sight distance problem*

If \( T = R_2 \sin \Omega > R_3 \) \( \Rightarrow \) There is no intersection between eyesight and the centerline of the nearest opposing through lane, and no sight distance problem.
If \( T = R_2 \sin \Omega \leq R_3 \implies \) There is intersections between them and possible sight distance problem, then go to Step 2 to calculate available sight distance, which also shows how to calculate the terms \( R_2, \Omega, \) and \( R_3. \)

**Step 2: Calculate available sight distance**

Figure 6-16 shows the related geometric relationship when there is intersection between eyesight and the centerline of the nearest opposing through lane. The available sight distance (SD) can be calculated by Equations 6-3-8, 6-3-9, and 6-3-10.

\[
SD = R_3(\Delta + \delta) \quad (6-3-8)
\]

\[
\Delta = \alpha - \beta \quad (6-3-9)
\]

Where, \( \alpha = \arcsin\left(\frac{X + D_1}{R_1}\right), \)

\[
\beta = \arcsin\left(\frac{X - D_2}{R_2}\right)
\]

\[
\delta = \pi - \gamma - \Delta - \Omega \quad (6-3-10)
\]

Where, \( \Omega = \arcsin\left(\frac{R_1 \sin \Delta}{L_1}\right) \)

\[
\gamma = \pi - \arcsin\left[\frac{R_1 \sin(\Omega + \Delta)}{R_3}\right]
\]
\[ L_1 = \sqrt{R_1^2 + R_2^2 - 2R_1R_2 \cos \Delta} \]

Figure 6-16: Geometric relationship for the model for left-turn maneuver toward the inside of the curve

As shown in Figure 6-15 and 6-16, where,

- SD = available sight distance (ft),
- \( \Delta \) = the angle between the curve radius to the left-turner’s eye and the curve radius to the right front corner of the opposing left-turn vehicle (radian),
\( \alpha = \) the angle between the curve radius to the right front corner of the opposing left-turn vehicle and the parallel line to the minor road (radian),

\( \beta = \) the angle between the curve radius to the left-turner’s eye and the parallel line to the minor road (radian),

\( \delta = \) the angle between the curve radius to the right front corner of the opposing left-turn vehicle and the curve radius to the point at which left-turner’s line of sight intersects the centerline of the near opposing through lane (see Figure 6-15) (radian),

\( \Omega = \) the angle between the curve radius to the left-turner’s eye and left turner’s sight line (radian),

\( \gamma = \) the angle between the curve radius to the point at which left-turner’s line of sight intersects the centerline of the near opposing through lane and left-turner’s sight line (see Figure 6-16) (radian),

\( D_1 = \) the distance from the right front corner of the opposing left-turn vehicle to the centerline of the minor road (ft),

\( D_2 = \) the distance from the left-turn driver’s eye to the centerline of the minor road (ft),

\( X = \) the distance from the curve center point to the centerline of the minor road (ft),

\( L_1 = \) the distance from the left-turn driver’s eye to the right front corner of the opposing left-turn vehicle (ft),

\( R = \) the curve radius to the inside edge of the median (ft),
\( R_1 = \) the curve radius to the right front corner of the opposing left-turn vehicle (ft),
\( R_2 = \) the curve radius to the left-turner’s eye (ft),
\( R_3 = \) the curve radius to the point at which left-turner’s line of sight intersects the centerline of the near opposing through lane (see Figure 6-16) (ft),

Where, \( R_1, R_2 \) and \( R_3 \) can be calculated by Equations 6-3-11, 6-3-12, and 6-3-13, respectively, based on the same definitions of \( R, m, n, g, e, V_w \), and \( L_t \) in the previous model.

\[
R_1 = R + m - n - g - V_w \quad (6-3-11)
\]
\[
R_2 = R + n + g + e \quad (6-3-12)
\]
\[
R_3 = R - L_t / 2 \quad (6-3-13)
\]

For the model for left-turn maneuver toward the inside of the curve, both of the median width \( m \) and curve radius \( R \) are negatively related to the sight distance. Figure 6-17 shows a sensitivity analysis of median width and curve radius on sight distance and the major road design speed, if holding other parameters, \( X=0 \text{ ft}, D_1=45 \text{ ft}, \) and \( D_2=37 \text{ ft}, \) and gradually increasing the radius from 1000 ft to 14000 ft with 1000 ft increments. The figure indicates that: 1) both curve radius and median width are negatively related to the sight distance; 2) if the curve radius is less than 1000 ft, there will be no sight distance problem for most of median types, unless medians widths are greater than 24 ft; 3) once
the opposing vehicle become the sight obstruction, the available sight distance can be small, which may not support higher major road speed; and 4) as the curve radius increases, its sensitivity to the sight distance decreases and median width is more important factor to the sight distance for the relatively flatter road curve.

Figure 6-17: Sensitive analysis of sight distance for left-turn maneuver toward the inside of the curve

Another interesting phenomenon is that there is a threshold for the sight distance if holding other parameters constant and only increasing the curve radius. It happens when the left-turner’s sight line is the tangent of the nearest coming through lane’s centerline (\( T = R_3 \)). The threshold is the maximum available sight distance once the opposing vehicle can be a sight obstruction, and then the sight distance becomes less and less as the curve radius increases. Table 6-5 lists a series of the threshold values of sight distance
and the corresponding curve radii, which decrease with the increment of the median width. However, for medians narrower than 16 ft, although there are sight distance thresholds existing, the corresponding curve radii are larger than 10000 ft. For that situation, the curves are so flat that the related available sight distances are very close to the results from the models for linear approach intersections.

Table 6-5: Threshold Values of Sight Distance and the Corresponding Curve Radiiuses

<table>
<thead>
<tr>
<th>X (ft)</th>
<th>D₁ (ft)</th>
<th>D₂ (ft)</th>
<th>M (ft)</th>
<th>R (ft)</th>
<th>SD (ft)</th>
<th>SPEED (mph)</th>
</tr>
</thead>
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<tr>
<td>0</td>
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<td>12</td>
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<td>540.3</td>
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<td>229.9</td>
<td>28.5</td>
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</tbody>
</table>

6.4 Geometric Models of Sight Distance Calculation for Intersection with Curve-linear Combined Approaches

The section 6.2 and 6.3 discussed the method of sight distance calculation for intersections with linear and curve approaches separately. If an intersection is located at a
major highway that is close to the tangent points of the linear and curved segments, the
above models for linear or curve approaches may result in inaccurate calculations. For
this situation, the models should be developed according to four specific cases:

- Case 1: Linear approach leading a curve segment for left turn toward outside of
  the curve,
- Case 2: Linear approach leading a curve segment for left turn toward inside of the
  curve,
- Case 3: Curve approach leading a linear segment for left turn toward outside of
  the curve, and
- Case 4: Curve approach leading a linear segment for left turn toward inside of the
  curve.

6.4.1 Linear approach leading a curve segment for left turn toward outside of the
curve

As shown in Figure 6-18, firstly, the validity of the linear model should be checked. If the
linear segment length of the intersection approach \(Y_1\) is less than the part of sight
distance \(Y\) calculated by the linear model, the left turner’s line of sight will intersect the
track of the coming-through vehicle on the curved segment.
Based on the simple geometric rules, the procedure to derivate SD is shown as follows:

\[ \delta = \arctan(\frac{A}{W}) \]

\[ \gamma = \arctan(\frac{R_1}{Y_2}) \]

\[ \theta = \gamma - \delta = \arctan(\frac{R_1}{Y_2}) - \arctan(\frac{A}{W}) \]

Since, \( \frac{\sin \Omega}{R_2} = \frac{\sin \theta}{R_1} \)  \[\Rightarrow\] \( \Omega = \pi - \arcsin(\frac{R_2}{R_1} \sin(\arctan(\frac{R_1}{Y_2}) - \arctan(\frac{A}{W}))) \)

\[ \beta = \pi - \Omega - \theta \]

\[ \nabla = \frac{\pi}{2} - \gamma \]
\[ \alpha = \nabla - \beta = \Omega - \delta - \frac{\pi}{2} \]

\[ C = R_1 \times \alpha \]

\[ SD = W + Y_1 + R_1 \times \left( \frac{\pi}{2} \right. - \arcsin\left( \frac{R_2}{R_1} \sin\left( \arctan\left( \frac{R_1}{Y_2} \right) - \arctan\left( \frac{A}{W} \right) \right) \right) - \arctan\left( \frac{A}{W} \right) \left. \right) \]

\[ R_2 = \sqrt{(Y - Y_1)^2 + R_1^2} \]

\[ SD = W + Y_1 + R_1 \times \left( \arcsin\left( \frac{\pi}{2} \right. - \sqrt{(Y - Y_1)^2 + R_1^2} \right) \left. \right) - \arcsin\left( \frac{R_1}{Y - Y_1} \right) - \arctan\left( \frac{A}{W} \right) \left. \right) - \arctan\left( \frac{A}{W} \right) \left. \right) \]

(6-4-2)

Equation 6-4-2 is used to calculate the sight distance at the linear-curve approach for left turn toward outside of the curve. Where, the definitions of W and A are the same as those in the model for the intersection with only linear approaches. \( Y_1 \) is defined as the linear segment length of the intersection approach. \( R_1 \) is the curve radius to the point at which left-turner’s line of sight intersects the centerline of the near opposing through lane, which is equal to the curve radius (R) plus median width plus half of the through lane width.

### 6.4.2 Linear approach leading a curve segment for left turn toward inside of the curve

Figure 6-19-a and figure 6-19-b show all basic parameters to calculate sight distance for left-turn toward the inside of the curve. However, for this situation, it is very possible that
there is no sight distance problem if there is no intersection between left turner line of sight and the curve track of the apposing through vehicle. Therefore, when $Y_1$ is less than $Y$, the first step for the model is to check if the opposing left-turn vehicle is a sight obstruction.

Figure 6-19: Geometric model for linear approach leading a curve for left turn toward inside of the curve
As shown in Figure 6-19-b, if \( T = (R_1 + B_2) \cdot \sin \beta > R1 \), there is no sight distance problem; otherwise, special calculations are needed for sight distance. Where, \( R_1 \) is the curve radius to the point at which left-turner’s line of sight intersects the centerline of the near opposing through lane, which is equal to the curve radius (R) minus half of the through lane width. Terms \( \beta \) and \( B_2 \) can be calculated as follows:

\[
\delta = \arctan\left(\frac{A}{W}\right)
\]

\[
\beta = \frac{\pi}{2} - \delta = \frac{\pi}{2} - \arctan\left(\frac{A}{W}\right)
\]

\[
B_1 = \frac{A \cdot Y_1}{W}
\]

\[
B_2 = 0.5L_t + m - n - g - V_w - B_1 = 0.5L_t + m - n - g - V_w - \frac{A \cdot Y_1}{W}
\]

Where, all the related parameters had been defined in previous models. Then, based on the simple geometric rules, the procedure to calculate C and SD is shown as follows:

\[
\frac{B_2 + R_1}{\sin \gamma} = \frac{R_1}{\sin \beta} = \frac{R_1}{\cos \delta} \quad \Rightarrow \quad \gamma = \arcsin\left(\frac{(B_2 + R_1) \cdot \cos \delta}{R_1}\right)
\]

\[
\alpha = \pi - \beta - \gamma = \frac{\pi}{2} + \arctan\left(\frac{A}{W}\right) - \arcsin\left(\frac{(B_2 + R_1) \cdot \cos(\arctan\left(\frac{A}{W}\right))}{R_1}\right)
\]

\[C = R_1 \cdot \alpha\]
\[
SD = W + Y + C = W + Y + R1 \left( \frac{\pi}{2} + \arctan \left( \frac{A}{W} \right) - \arcsin \left( \frac{B2 + R1}{R1} \cos(\arctan \left( \frac{A}{W} \right)) \right) \right)
\]

Equation 6-4-3 is the model to use for calculating the sight distance at the linear-curve approach for left turn toward inside of the curve.

6.4.3 Curve approach leading a linear segment for left turn toward outside of the curve

Figure 6-20 shows the basic geometric relationship and corresponding parameters needed to calculate sight distance at an intersection with curve approach leading a linear segment curve-linear approach for left turn toward outside of the curve. If the central angle \( \eta \), corresponding to the curve segment length of the intersection approach \( C_1 \), is less than \( \delta \), the left turner’s line of sight will intersect the track of the coming through vehicle on the linear segment. For this case, based on the simple geometric rules, the procedure to derive SD is shown as follows:

\[
\eta = \frac{C_1}{R}
\]

\[
\lambda = \Omega - \eta
\]

\[
\frac{R1}{\sin(\lambda)} = \frac{D_1}{\sin(\pi - \Omega - \Delta)} \quad \Rightarrow \quad D_1 = \frac{\sin(\Omega + \Delta) \ast R1}{\sin \lambda}
\]

\[
D_2 = R_3 - D_1
\]
\[ Y = D_2 \cdot \tan \lambda \quad \Rightarrow \quad Y = (R_3 - D_1) \cdot \tan \lambda \]

\[ Y = (R_3 - \frac{\sin(\Omega + \Delta) \cdot R_1}{\sin(\Omega - \frac{C_1}{R})}) \cdot \tan(\Omega - \frac{C_1}{R}) \]

\[ SD = C + Y = R_3 \cdot (\Delta + \frac{C_1}{R}) + \left( R_3 - \frac{\sin(\Omega + \Delta) \cdot R_1}{\sin(\Omega - \frac{C_1}{R})} \right) \cdot \tan(\Omega - \frac{C_1}{R}) \]  \hspace{1cm} (6-4-4)

Figure 6-20: Geometric model for curve approach leading a linear segment for left turn toward outside of the curve

Equation 6-4-4 is the eventual model to calculate the sight distance at the linear-curve approach for left turn toward outside of the curve. Where, except \( C_1 \), all the other terms have the same definitions as those in the previous curve model for left-turn toward
outside of the curve. \( C_1 \) is the curve segment length of the intersection approach along the inside of the median.

### 6.4.4 Curve approach leading a linear segment for left turn toward inside of the curve

When the curve approach of the intersection leads to a linear segment, firstly, it is necessary to check if the left turning driver’s line of sight intersects the track of the opposing through vehicle on the linear segment. As shown in Figure 6-21, if \( \lambda = \Omega + \Delta + \eta \geq \pi / 2 \), there is no sight distance problem; otherwise, special calculations are needed for sight distance. For this case, based on the simple geometric rules, the procedure to derive SD is shown as follows:

\[
\eta = \frac{C_1}{R} \]

\[
\lambda = \Omega + \Delta + \eta
\]

\[
\frac{R_2}{\sin(\pi - \eta - \Delta - \Omega)} = \frac{D_1}{\sin(\Omega)} \quad \Rightarrow \quad D_1 = \frac{R_2 \sin(\Omega)}{\sin(\lambda)}
\]

\[
D_2 = D_1 - R_3 \quad \text{and} \quad Y = D_2 * \tan\lambda
\]

\[
Y = \left( \frac{R_2 \sin(\Omega)}{\sin(\lambda)} - R_3 \right) * \tan(\lambda)
\]

\[
SD = C + Y = R_3 * (\Delta + \frac{C_1}{R}) + \left( \frac{R_2 \sin(\Omega)}{\sin(\Omega + \Delta + \frac{C_1}{R})} - R_3 \right) * \tan(\Omega + \Delta + \frac{C_1}{R})
\]  \quad (6-4-5)
Equation 6-4-5 is the eventual model to calculate the sight distance at the linear-curve approach for left turn toward inside of the curve. Where, except $C_1$, all the other terms have the same definitions as those in the previous curve model for left-turn toward inside of the curve. $C_1$ is the curve segment length of the intersection approach along the inside of the median.

6.4.5 Evaluation of sight distance problem and parameter analyses
For linear type intersections, the study indicated that the available left-turn sight distance is inversely related to the median width (m). The sight distance problem could even occur on the traditional left-turn lane design with 14-18 ft medians at high major-road design speed. If the intersection is located on or near a horizontal curve, besides the effect of the median width (m), the curve presence may contribute to or mitigate the sight distance problem, which depends on whether left-turn maneuvers toward outside or inside of the curve. To compare sight distance calculation by different type of models, the same basic geometric features assumed as follows:

- median width (m) is 16 ft,
- median nose width (n) is 4 ft,
- the width of left-turn lane is 12 ft,
- the width of the opposing through lane (L_2) is 12 ft, and
- the distance between stop bars of the opposing left lanes (term D in linear model) is 74 ft (2-lane minor road assumed).

Figure 6-22-a and 6-22-b illustrates a sensitivity analysis of curve radius (R) and the linear segment length (Y_1), and sight distance calculation comparison between linear model and those related to curves. Figure 6-22-c and 6-22-d illustrates the analysis of curve radius (R) and the curve segment length (C_1), and sight distance calculation comparison between curve model and those related to linear segments.
a. Linear approach leading a curve segment for left turn toward outside of the curve

b. Linear approach leading a curve segment for left turn toward inside of the curve
c. Curve approach leading a linear segment for left turn toward outside of the curve

d. Curve approach leading a linear segment for left turn toward inside of the curve

Figure 6-22: Evaluation of Sight Distance Problem and Parameter Analysis for different models
As shown in Figure 6-22-a, if opposite linear approach leading a curve segment and drivers making left turn toward outside of the curve (scenario illustrated in Figure 6-2-c), the presence of curve always deteriorates the sight distance problem. Both R and Y1 are positively related to the SD. The sight distance is more sensitive to curve radius for the shorter length of Y1, and as the Y1 and R increase, the sight distance is closer to the value (246 ft) attained from the linear model calculation and shown as a dash line. However, even 246 ft sight distance can only provide 30.5 mph safe design speed according to equation 1. Therefore, for this case, the sight distance may not be sufficient for major road with relatively higher speed limit.

Figure 6-22-b shows the sight distance calculation for opposite linear approach leading a curve segment when drivers making left turn toward the inside of the curve (scenario illustrated in Figure 6-2-d) and the presence of curve can always mitigate sight distance problem. Both R and Y1 are inversely related to the SD. The sight distance is more sensitive to curve radius for the shorter length of Y1, and as the Y1 and R increase, the sight distance gets closer to the result from the linear model calculation. When Y1 is beyond 150 ft, the curve is not under consideration. If the curve radius is smaller, especially less than 4000 ft, the curve will even result in unrestricted sight distance. However, if the curve radius is larger than 8000 ft, the available sight distances are less than 300 ft, which can provide 37.2 mph design speed and may still not be sufficient for higher speed limit.
Figure 6-22-c shows the sight distance calculation for opposite curve approach leading a linear segment when drivers making left turn toward outside of the curve (scenario illustrated in Figure 6-2-e). The reference dash line in the figure is the sight distance for a pure curve approach. For this situation, the major road curve can result in serious sight distance problem; the R is positive related sight distance; and for the 16 ft median, the available sight distance might be very short, less than 225 ft, even if the curve radius is very large, which cannot provide 30 mph design speed. Compared to the results from the curve model, the presence of a linear segment can slightly mitigate the sight distance problem; and the curve segment length to the tangent point ($C_1$) is inversely related to the sight distance.

Figure 6-22-d shows the available sight distance for opposite curve approach leading a linear segment when drivers making left turn toward inside of the curve (scenario illustrated in Figure 6-2-f). The presence of curve can greatly mitigate the sight distance problem and the radius R is inversely related to the SD. An interesting phenomenon for left turn toward inside is that there is a threshold for the sight distance if holding other parameters constant and only increasing the curve radius. It happens when the left-turner’s sight line is the tangent of the nearest coming through lane’s centerline. The threshold is the maximum available sight distance once the opposing vehicle can be a sight obstruction, and then the sight distance decreases as the curve radius increases. For 16 ft medians, the sight distance threshold is 445.3 ft, and the corresponding curve radius is 7406.8 ft for the pure curve approach. However, the presence of a linear segment can decrease the sight distance benefit from the curve; the curve segment length to the
tangent point ($C_1$) is positively related to SD; when $C_1$ is zero, the SD calculation results are closer to that from the linear model as the R increases.

6.5 Conclusions and Discussions

Left-turn sight distance is an important geometric design factor for traffic turning left during the unprotected green phase at signalized intersections. The challenge that left-turners confront is the blockage of view introduced by vehicles located in the opposing left-turn lane. Inadequate sight distance can cause drivers to aggressively accept small gaps to cross the opposing through traffic, even contribute to their illegal traffic performance. To maximize available sight distance, left-turning drivers might move the vehicle out beyond the stop bar of the left-turn lane, and encroach into the pedestrian crossing, or drive the vehicle as near as possible to the median. These behaviors can both reduce the driving comfort level and increase the probability of traffic crashes. This study introduced six types of geometric models to calculate left turn sight distance for intersection with different configurations. They include intersections located on a linear road, a curve road, a linear segment leading a curved segment, and a curved segment leading a linear segment. These geometric models presented in this paper can be used to identify a better intersection location along the major road when a curve exists, layout intersection geometric design, or evaluate the sight distance problem of an existing intersection configuration to ensure safe left-turn maneuvers by drivers.
For linear-type intersections, the study focused on the design aspect of left-turning drivers and geometric models were developed to evaluate the improvement effects of the two offset methods for opposing left-turn lanes. Using reasonable values assumed for geometric parameters of the intersection, the models testified that the sight distance problem for left-turners could even occur on the traditional left-turn lane design with 14-18 ft medians at high major-road design speed. AASHTO has indicated that medians narrower than 18 ft should not have sight distance problems. Through the use of the developed models, sensitivity analyses illustrated the relationship between the sight distance and the offset value for parallel left lanes, as well as the effect of the left-turn lane length and taper angle on sight distance improvement. The models can be adjusted to accommodate special features. For example, in Equations 5 and 13 the term A assumes that the far left edge of the median curb is aligned with the opposing left-turn lane line. In fact, sometimes there is a small offset by one or two feet between two major approaches. This value can be added into the term A in both equations, so that the model is still valid. In addition to 90-degree intersections, the models can also be applied to skewed intersections since there is no essential change in the relationship among all parameters used for the major approaches, but it should be cautioned that the required sight distances maybe differ from the normal ones due to the change of drivers’ gap acceptance behavior. However, the models can only be applied to the intersections with a straight major road approach. If the minor road intersects a curved the major road, the models are not valid.

For intersections located on a horizontal curve, sight distance calculation models for left-turn maneuver toward the outside of the curve and inside of the curve are presented
respectively in this study. The former model concluded that the major road curve can result in sight distance problem and its radius is positively related to the sight distance; for the 12 ft median, the available sight distance might be insufficient for the higher design speed on the major road even if the curve is not sharp. The later model indicated that the curve radius is negatively related to the sight distance and there is a threshold of the radius for different median widths. Only if the radius is larger than the threshold value, the opposing left-turn vehicle could be a potential sight obstruction. For 12 to 16 ft medians, there is normally no sight distance problem for curved major road; for median width beyond 16 ft, if the curve radius is large, the sight distance is possibly insufficient for the higher design speed on the major road. In additions, for both models, the curve radius lacks sensitivity to the available sight distance if it is larger than 10000 ft. Instead, median width plays a more important role on that. The models and related analyses can be used to layout intersection design or evaluate the sight distance problem of an existing intersection configuration to ensure safe left-turn maneuvers by drivers. Especially, if the left-turn traffic volumes in both left-turn lanes are relatively heavy, protected phase is suggested for left-turn traffic toward the outside of the curve. In the models, the term X is the distance from the curve center point to the centerline of the minor road, which is related to the intersection type. If X is equal to zero, it means that the minor road intersects the major road curve at a right angle; if X is unequal to zero, the intersection is skewed. According to the calculation analysis, X is not sensitive to the sight distance. Compared to zero X, a value of 200 ft for X can cause only 1.3 ft sight distance difference for 1000 ft curve radius. In addition, the models and related analyses assume that the far left edge of the median curb is aligned with the opposing left-turn lane line in
the same radius curve. Otherwise, the terms $R_1$, $R_2$ and $R_3$ need to be adjusted.

Additionally, the model development focused on the situation that the major road of the intersection is a single circular curve. For an intersection with special geometric features, such as two-circular curve combination or a spiral curve, the models will be invalid.

Based on the sight distance calculation for opposite linear approach leading a curve segment, the presence of curve can always mitigate sight distance problem when drivers making left turn toward the inside of the curve. In the other hand, if drivers making left turn toward outside of the curve, the major road curve can result in serious sight distance problem. Compared to the results from the curve model, the presence of a linear segment can slightly mitigate the sight distance problem; and the curve segment length to the tangent point is inversely related to the sight distance.

When an opposite curve approach leads a linear segment and drivers make left turn toward outside of the curve, the major road curve can result in serious sight distance problem; but the presence of a linear segment can slightly mitigate the sight distance problem; and the curve segment length to the tangent point is inversely related to the sight distance. On the other hand, if drivers make left turn toward inside of the curve, the presence of curve can greatly mitigate the sight distance problem and the curve radius is inversely related to the sight distance. However, the presence of a linear segment can reduce the sight distance benefit from the curve; the curve segment length to the tangent point is positively related to sight distance; when the curve length is zero, the SD calculation results are closer to that from the linear model.
Additionally, the models and related analyses only focus on permitted left-turn phasing scheme. Similar issues related to sight visibility for left-turning vehicles could potentially occur for non-signalized intersections such as a two-way stop controlled intersection, but it is not suggested to directly apply the above models to non-signalized situations because without stop bars in the left-turn lanes, the left-turning vehicles’ positions may be more flexible before crossing the opposing through traffic.

The model development was only based on the behaviors and features of passenger cars, but it is believed that trucks may pose serious problems to both operation and safety. A truck with its larger dimensions would definitely result in much worse blockage of opposing left-turn vehicles and therefore aggravate the situation further. Also left-turning trucks need larger gaps to cross the opposing through traffic which could deteriorate operation conditions.

It is noticeable that two studies (Alexander et al., 2002; Darzentas et al., 1980) indicated that major road traffic speed has an important effect on driver’s gap acceptance. Those studies are challenging the constant gap time applied to Equation 1 so that the required sight distance may be different from the AASHTO guideline. In addition, AASHTO did not consider driver age difference as a variable for study on gap acceptances and intersection sight distances. However many related researches (Alexander et al., 2002; Lerner et al., 1995; Tarawneh and McCoy, 1996) showed that older drivers usually accept gaps apparently longer than younger drivers. Therefore, in areas with high-density older
drivers’ population, the threshold of the intersection sight distance design needs to be investigated.
CHAPTER 7. SUMMARY

Crashes categorized as running red light or turning left to cross oncoming traffic are most likely to occur at signalized intersections and resulted in substantial severe injuries and property damages. This dissertation mainly focused on the two types of vehicle crashes and the research methodology involved several perspectives. Firstly, based on 1999-2001 Florida Crash Database, this study thoroughly examined the overall characteristics of red-light running and left-turning crashes, and further investigated the accident propensity of three aspects of risk factors related to traffic environments, driver characteristics, and vehicle types. Secondly, to reduce the red-light running rate, a driving simulator experiment was applied to test the effect of a proposed new pavement marking countermeasure which purpose is to reduce the red-light running rate at signalized intersections. Thirdly, to ensure safe unprotected left-turn maneuvers by drivers, geometric models to compute sight distance of unprotected left-turns were developed for different signalized intersection configurations, including a straight approach leading to a straight one, a straight approach leading to a curved one, and a curved approach leading to a curved one.

7.1 Characteristics of Red-light Running Crashes
The analysis showed that the risk of red-light running crashes for 6-lane highways is higher than 2-lane and 4-lane highways. The relative crash involvement ratio for night is apparently lower than daytime and the crash ratio for weekend is higher than weekdays. Compared to clear weather, crashes more likely to occur under cloudy weather and are less involved in rainy weather. Geometric configuration of the intersection can also influence the crash occurrence. Especially during daytime, complex intersection geometric conditions such as up/downgrade and horizontal curve may contribute to the higher crash involvement rate. Moreover, red-light running crashes are more relevant to urban areas and most likely happen with 30 and 35 mph speed limits.

The results indicated that the younger and older drivers (55 years and over) are over-involved in red-light running crashes. Although the driver gender is not a main effect factor associated with crash risk, it has interaction effects with driver age, vehicle type, and alcohol/drug use. Young male and old female groups are over-presented in the crashes, and for middle age groups (26-45 years), gender has no apparent crash propensity. Based on vehicle type, vans and light trucks have relatively higher crash risk and large size vehicles have the smallest crash propensity. Considering the interaction effect with gender, male drivers have larger crash propensities for light trucks and large size vehicles.

The analysis confirmed the substantial effect of alcohol/drug use on driver’s safety. Even drivers who had been drinking could be 3.56 times more likely involved in red-light running crashes than non-drinking drivers. Drivers with physical defects, especially those
with fatigue/asleep or Seizure/Epilepsy/Blackout problems, were identified as highest risk group at signalized intersections. Medical countermeasures for those drivers are very necessary, especially when they drive in urban area with higher density of traffic-controlled intersections and larger traffic volumes. In addition, non-local driver tend to be over-presented in red-light running crashes.

### 7.2 Characteristics of Unprotected Left-turning Crashes

Mostly, points of impact in the crashes are in front or right side of the left-turn vehicles and the estimated speeds of oncoming-through vehicles are relatively higher. Compared to the other crashes, 71.2% disabling damage rate and 27.4% functional damage rate for left-turn vehicles is extremely high and the left-turn crashes more likely caused serious injury and even fatal injury.

Through comparisons between the left-turn crashes and other two-vehicle crashes at signalized intersections, the traffic environment analyses show that the number of lanes, divided/undivided highway, lighting condition, weather, and posted speed limit are significant variables associated with left-turn crashes. Compared to the other crashes, the conditional probabilities of left-turn accidents for 4-lane and 6-lane highways are significantly higher than the 2-lane highways. The relative crash risk at divided highways is higher than undivided highways. The left-turn crashes are most likely to happen at the dark condition without streetlight, and the crash risk for nighttime with streetlight is
lower than dusk or dawn surrounding. Compared to the clear weather, the cloudy weather is over-represented in the crashes. It was also found that that the crash risk for relatively higher speed limits (>35mph) are significantly larger than that for lower speed limits.

Based on the tree classification, drivers were segmented into four homogeneous age groups: younger than 18 years, 18-55 years, 56-65 years, and older than 65 years. Since the most of 15-17 years drivers are more likely to be learner drivers or new drivers with less driving experiences, the particular education program to emphasize the unprotected left-turn risk and related traffic policy at signalized intersections is strongly suggested for the younger group. The risk for older drivers is increasing as the traffic environments become more complex at intersections with larger number of lanes and higher traffic volume. Therefore, for the area with high density of older drivers, reducing left-turn chance and protected left-turn signal phase are recommended. The result testified that under adverse vision conditions, older drivers’ conservative driving attitude could compensate for the decline in their driving capability. The examination of the relative crash propensity of driver gender indicates that the female drivers are more likely to be involved in left turn crashes compared to the male drivers. The analysis confirmed the substantial effect of alcohol use on driver’s safety. Especially, left-turn crashes involved in alcohol use mainly happened in middle age drivers from 18 to 65 years. Moreover, drivers’ physical defect and unfamiliarity with local traffic environment were also found to be significant factors associated with the left-turn crashes.
7.3 Effect of the Pavement Marking Countermeasure

According to the results of the driving simulator experiment, the pavement-marking countermeasure has a significantly positive effect on signalized-intersection safety. Firstly compared to regular intersections, the pavement marking could results in a 74.3 percent reduction in red-light running. In comparison, the pavement marking reduced the number of occurrences where drivers chose to continue through an intersection when it was not safe to proceed compared to the without marking, and this result is correlated to less red-light running rate with marking. Furthermore, for those running red-light drivers, the marking tends to reduce the red-light entry time. The results may contribute to reducing the probability of angle crashes.

Secondly, logistic regression models attest that the marking is helpful to improve driver stop-go decision at intersections. Compared to without marking, if the drivers located near to the stop bar, drivers tend to cross the intersection with the marking; if the drivers located farther to the stop bar, drivers tend to stop at the intersection with the marking. The results showed that the uncertainty distances between 20% and 80% probability of stopping with marking are about 23 ft for the 30 mph and 50 ft for the 45 mph shorter in comparison with regular intersections. The analysis indicates that the marking information can help to reduce driver hesitated region to decide to stop or cross the intersection, which possibly results in higher accident rates.
Thirdly, it was found that for those stopping drivers, the brake deceleration rate without marking is 1.959 ft/s$^2$ significantly larger than that with marking for the higher speed limit. With the marking information, the probability that drivers make a too conservative stop will decrease if they are located in the downstream of marking at the onset of yellow, which resulted in the gentler deceleration rate with marking. At intersections, the smaller deceleration rate may contribute to the less probability that rear-end crashes happen.

Moreover, according to survey results, all of subjects gave a positive evaluation on the pavement-marking countermeasure and nobody felt confused or uncomfortable when they made stop-go decision. In comparison between scenarios without marking and with marking, there is no significant difference found in the operation speeds and drivers brake response time, which proved that the marking has no significantly negative effect on driver behaviors at intersections.

Therefore, the pavement-marking countermeasure may contribute to reducing the number of red light running violations and improving traffic safety situation related to both angle and rear-end crashes at signalized intersections.

### 7.4 Geometric Models to Calculate Unprotected Left-turn Sight Distance

This study introduced six types of geometric models to calculate left turn sight distance for intersection with different configurations. They include intersections located on a
linear road, a curve road, a linear segment leading a curved segment, and a curved segment leading a linear segment. These geometric models presented in this study can be used to identify a better intersection location along the major road when a curve exists, layout intersection geometric design, or evaluate the sight distance problem of an existing intersection configuration to ensure safe left-turn maneuvers by drivers.

For linear-type intersections, the study focused on the design aspect of left-turning drivers and geometric models were developed to evaluate the improvement effects of the two offset methods for opposing left-turn lanes. The models testified that the sight distance problem for left-turners could even occur on the traditional left-turn lane design with 14-18 ft medians at high major-road design speed. Through the use of the developed models, sensitivity analyses illustrated the relationship between the sight distance and the offset value for parallel left lanes, as well as the effect of the left-turn lane length and taper angle on sight distance improvement.

For intersections located on a horizontal curve, sight distance calculation models for left-turn maneuver toward the outside of the curve and inside of the curve are presented respectively in this study. The former model concluded that the major road curve can result in sight distance problem and its radius is positively related to the sight distance; for the 12 ft median, the available sight distance might be insufficient for the higher design speed on the major road even if the curve is not sharp. The later model indicated that the curve radius is negatively related to the sight distance and there is a threshold of the radius for different median widths. Only if the radius is larger than the threshold value,
the opposing left-turn vehicle could be a potential sight obstruction. For 12 to 16 ft medians, there is normally no sight distance problem for curved major road; for median width beyond 16 ft, if the curve radius is large, the sight distance is possibly insufficient for the higher design speed on the major road. In additions, for both models, the curve radius lacks sensitivity to the available sight distance if it is larger than 10000 ft. Instead, median width plays a more important role on the sight distance. Especially, if the left-turn traffic volumes in both left-turn lanes are relatively heavy, protected phase is suggested for left-turn traffic toward the outside of the curve. However, the model development focused on the situation that the major road of the intersection is a single circular curve. For an intersection with special geometric features, such as two-circular curve combination or a spiral curve, the models will be invalid.

Based on the sight distance calculation for opposite linear approach leading a curve segment, the presence of curve can always mitigate sight distance problem when drivers making left turn toward the inside of the curve. In the other hand, if drivers making left turn toward outside of the curve, the major road curve can result in serious sight distance problem. Compared to the results from the curve model, the presence of a linear segment can slightly mitigate the sight distance problem; and the curve segment length to the tangent point is inversely related to the sight distance.

When an opposite curve approach leads a linear segment and drivers make left turn toward outside of the curve, the major road curve can result in serious sight distance problem; but the presence of a linear segment can slightly mitigate the sight distance
problem; and the curve segment length to the tangent point is inversely related to the sight distance. On the other hand, if drivers make left turn toward inside of the curve, the presence of curve can greatly mitigate the sight distance problem and the curve radius is inversely related to the sight distance. However, the presence of a linear segment can reduce the sight distance benefit from the curve; the curve segment length to the tangent point is positively related to sight distance; when the curve length is zero, the SD calculation results are closer to that from the linear model.

It is noticeable that the model development was only based on the behaviors and features of passenger cars, but it is believed that trucks may pose serious problems to both operation and safety. A truck with its larger dimensions would definitely result in much worse blockage of opposing left-turn vehicles and therefore aggravate the situation further. Also left-turning trucks need larger gaps to cross the opposing through traffic which could deteriorate operation conditions. In addition, this study did not consider driver age difference as a variable for study on gap acceptances and intersection sight distances. In areas with high-density older drivers’ population, the required left-turn sight distance may need to be increased.
APPENDIX A. FORTRAN PROGRAM TO PROCESS EXPERIMENT DATA
The following code for the ‘45_go_up’ scenario as an example shows how to find the key parameters from the experiment output file:

```
IMPLICIT REAL*8(A-H,O-Z)
CHARACTER*70 A,dataname,outname

DIMENSION STIME(100000),X(100000),Y(100000),STEER(100000)
DIMENSION ACCE(100000),BRAKE(100000),SPEED(100000)
DIMENSION NSECTION(7),NAMESECTION(4),ISTOP(4),IPASS(4)
DIMENSION TREACTION(4),DECELERATION(4),DISTANCE(4)
DIMENSION SMINCROSS(4),SMAXSTOP(4),IDILEMMA(4)
DIMENSION XPASS(4),TPASS(4),NSTOPBAR(4)

C NOTE:
C IDILEMMA(I)=1, STOP ZONE
C 2, CROSS ZONE
C 3, OPTION ZONE
C 4, DILEMMA ZONE
C ISTOP=0, VEHICLE ACTUALLY STOPPED
C 1, VEHICLE ACTUALLY CROSSED
C IPASS=0, VEHICLE NON-RUNNING RED LIGHT
C 1, VEHICLE RUNNING RED LIGHT

write(*,*)'Please input data file name.'
read(*,*) dataname
C DATANAME ='Yan_1000_45_up_a.LOG'
write(*,*)'Please input output file name.'
read(*,*) outname
C outname ='Yan_1000_45_up_a.TXT'

OPEN(1,FILE=dataname,STATUS="UNKNOWN")
OPEN(2,FILE=outname,STATUS="UNKNOWN")

DO I=1,7
READ(1,*)A
  IF(I.LE.6)write(2,*)(A
  IF(I.EQ.7)write(2,*),' ') ENDDO
DO NP=1,100000
READ(1,*)STIME(NP),X(NP),Y(NP),STEER(NP),
  *  ACCE(NP),BRAKE(NP),SPEED(NP)
  IF(NP.GT.1.AND.Y(NP).GE.6496.71.AND.Y(NP-1).LE.7356.14)THEN
    NSECTION(1)=NP
    NSECTION(2)=NP
```

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ELSEIF(NP.GT.1.AND.Y(NP).GE.8162.43.AND.Y(NP-1).LE.8162.43)THEN
NSECTION(3)=NP
ELSEIF(NP.GT.1.AND.Y(NP).GE.8637.86.AND.Y(NP-1).LE.8637.86)THEN
NSECTION(4)=NP
ELSEIF(NP.GT.1.AND.Y(NP).GE.6350.0.AND.Y(NP-1).LE.6350.0)THEN
NSECTION(5)=NP
ELSEIF(NP.GT.1.AND.Y(NP).GE.6950.0.AND.Y(NP-1).LE.6950.0)THEN
NSECTION(6)=NP
ELSEIF(NP.GT.1.AND.Y(NP).GE.7810.0.AND.Y(NP-1).LE.7810.0)THEN
NSECTION(7)=NP
ENDIF
IF(NP.GT.1.AND.Y(NP).GE.6591.0.AND.Y(NP-1).LE.6591.0)THEN
NSTOPBAR(1)=NP
ELSEIF(NP.GT.1.AND.Y(NP).GE.7419.0.AND.Y(NP-1).LE.7419.0)THEN
NSTOPBAR(2)=NP
ELSEIF(NP.GT.1.AND.Y(NP).GE.8241.0.AND.Y(NP-1).LE.8241.0)THEN
NSTOPBAR(3)=NP
ELSEIF(NP.GT.1.AND.Y(NP).GE.8740.0.AND.Y(NP-1).LE.8740.0)THEN
NSTOPBAR(4)=NP
ENDIF
IF(Y(NP).GE.8780.0)THEN
NP=NP
GOTO 10
ENDIF
ENDDO
10 CONTINUE

C*****CALCULATE AVERAGE SPEED
AVESPEED=0.0
DO I=1,7
   AVESPEED=AVESPEED+SPEED(NSECTION(I))/7.0
ENDDO
WRITE (2,*)'THE AVERAGE SPEED = ',AVESPEED
WRITE(2,*)''

C*****ANALYZE INTERSECTION INFORMATION
WRITE(2,80)'IN# DIST(M) ISTOP IPASS TREAC(S) SPEED(MILE/H)
*DECEL(S^2/FT) IDILEMMA(I) GAP(S) XPASS(M) TPASS(S)'
NAMESECTION(1)=134
NAMESECTION(2)=131
NAMESECTION(3)=135
NAMESECTION(4)=136
SMINCROSS(1)=48.91
SMINCROSS(2)=32.60
SMINCROSS(3)=40.75
SMINCROSS(4)=52.98
SMAXSTOP(1)=47.11
SMAXSTOP(2)=37.39
SMAXSTOP(3)=42.49
SMAXSTOP(4)=49.28
DISTANCE(1)=94.29
DISTANCE(2)=62.86
DISTANCE(3)=78.57
DISTANCE(4)=102.14
DELT=1.0/6.0
NDELT=DELT*60
DO I=1,4
C======INITIALLY, DECIDE IF THERE IS DILEMMA ZONE.
C========THERE IS OPTION ZONE
   IF(SMINCROSS(I).LT.SMAXSTOP(I))THEN
     IF(SPEED(NSECTION(I)).LT.SMINCROSS(I))IDILEMMA(I)=1
     IF(SPEED(NSECTION(I)).GT.SMAXSTOP(I))IDILEMMA(I)=2
     IF(SPEED(NSECTION(I)).GE.SMINCROSS(I).AND.*
      SPEED(NSECTION(I)).LE.SMAXSTOP(I))IDILEMMA(I)=3
   ENDIF  
C========THERE IS DILEMMA ZONE
   ELSE
     IF(SPEED(NSECTION(I)).LT.SMAXSTOP(I))IDILEMMA(I)=1
     IF(SPEED(NSECTION(I)).GT.SMAXSTOP(I))IDILEMMA(I)=2
     IF(SPEED(NSECTION(I)).GE.SMINCROSS(I).AND."
      SPEED(NSECTION(I)).LE.SMAXSTOP(I))IDILEMMA(I)=3
   ENDIF  
C======FIRST DECIDE IF THE VEHICLE STOPPED BEFORE THE INTERSECTION OR NOT
C--------NP1 IS THE YELLOW BEGINNING
NP1=NSECTION(I)
DO NP=NP1,NP
   IF(SPEED(NP).EQ.0.0)THEN
      C-----------NP2 IS THE FIRST STOP AFTER THE YELLOW BEGINNING
      NP2=NP
      IF(NP2.GT.NP)Y(NP2)=Y(NP)
      GOTO 20
   ENDIF
ENDDO
C-------IF THERE IS NO STOP AFTER THE YELLOW BEGINNING, ISTOP=0
GOTO 30
20   CONTINUE
C-------IF STOP IS BEFORE THE INTERSECTION, ISTOP=1, OTHERWISE, ISTOP=0
   IF(Y(NP2).LT.(Y(NP1)+DISTANCE(I)+15.0))ISTOP(I)=1
   IF(IPASS(I).EQ.0.0)THEN
      XPASS(I)=Y(NSTOPBAR(I))-Y(NP3)
      TPASS(I)=(NSTOPBAR(I)-NP3)/60.0
   ENDIF
   DO NP=NP1,NNP
      IF(ACCE(NP+NDELT).GT.ACCE(NP))THEN
         IF(ACCE(NP+2*NDELT).GT.ACCE(NP+NDELT))THEN
            IF(ACCE(NP+3*NDELT).GT.ACCE(NP+2*NDELT))THEN
               IF(ACCE(NP+4*NDELT).GT.ACCE(NP+3*NDELT))THEN
                  IF(ACCE(NP+5*NDELT).GT.ACCE(NP+4*NDELT))THEN
                     C-----------------------NP IS THE FIRST CONTINUOUS ACCELERATION AFTER YELLOW
                     BEGINING
                  ENDIF
               ENDIF
            ENDIF
         ENDIF
      ENDIF
   ENDIF
C=======CALCULATE REACTION TIME FOR THE NON-STOPPED VEHICLE
   IF(ISTOP(I).EQ.0.0)THEN
      C---------NP3 IS THE YELLOW ENDING
      NP3=NP1+4.5*60
      IF(NP3.GT.NP)Y(NP3)=Y(NP)
      C---------DECIDE THE NON-STOPPED VEHICLE PASSED THE INTERSECTION BEFORE RED
      OR NOT
      IF(Y(NP3).LE.(Y(NP1)+DISTANCE(I)))IPASS(I)=1
      IF(IPASS(I).EQ.1)THEN
         XPASS(I)=Y(NSTOPBAR(I))-Y(NP3)
         TPASS(I)=(NSTOPBAR(I)-NP3)/60.0
      ENDIF
   DO NP=NP1,NNP
      IF(RE(C(NP+NDELT)).GT.RE(C(NP)))THEN
         IF(RE(C(NP+2*NDELT)).GT.RE(C(NP+NDELT)))THEN
            IF(RE(C(NP+3*NDELT)).GT.RE(C(NP+2*NDELT)))THEN
               IF(RE(C(NP+4*NDELT)).GT.RE(C(NP+3*NDELT)))THEN
                  IF(RE(C(NP+5*NDELT)).GT.RE(C(NP+4*NDELT)))THEN
                     C-----------------------NP IS THE FIRST CONTINUOUS ACCELERATION AFTER YELLOW
                     BEGINING
                  ENDIF
               ENDIF
            ENDIF
         ENDIF
      ENDIF
   ENDIF
C---------CALCULATE REACTION TIME FOR THE VEHICLE WHICH STOPPED BEFORE THE INTERSECTION
   IF(IPASS(I).EQ.1)THEN
      XPASS(I)=Y(NSTOPBAR(I))-Y(NP)
      TPASS(I)=(NSTOPBAR(I)-NP)/60.0
   ENDIF
   DO NP=NP1,NNP
      IF(RE(C(NP+NDELT)).GT.RE(C(NP)))THEN
         IF(RE(C(NP+2*NDELT)).GT.RE(C(NP+NDELT)))THEN
            IF(RE(C(NP+3*NDELT)).GT.RE(C(NP+2*NDELT)))THEN
               IF(RE(C(NP+4*NDELT)).GT.RE(C(NP+3*NDELT)))THEN
                  IF(RE(C(NP+5*NDELT)).GT.RE(C(NP+4*NDELT)))THEN
                     C-----------------------NP IS THE FIRST CONTINUOUS ACCELERATION AFTER YELLOW
                     BEGINING
                  ENDIF
               ENDIF
            ENDIF
         ENDIF
      ENDIF
   ENDIF
C---------CALCULATE REACTION TIME FOR THE VEHICLE WHICH STOPPED AFTER THE INTERSECTION
   IF(IPASS(I).EQ.0.0)THEN
      XPASS(I)=Y(NSTOPBAR(I))-Y(NP)
      TPASS(I)=(NSTOPBAR(I)-NP)/60.0
   ENDIF
   DO NP=NP1,NNP
      IF(RE(C(NP+NDELT)).GT.RE(C(NP)))THEN
         IF(RE(C(NP+2*NDELT)).GT.RE(C(NP+NDELT)))THEN
            IF(RE(C(NP+3*NDELT)).GT.RE(C(NP+2*NDELT)))THEN
               IF(RE(C(NP+4*NDELT)).GT.RE(C(NP+3*NDELT)))THEN
                  IF(RE(C(NP+5*NDELT)).GT.RE(C(NP+4*NDELT)))THEN
                     C-----------------------NP IS THE FIRST CONTINUOUS ACCELERATION AFTER YELLOW
                     BEGINING
                  ENDIF
               ENDIF
            ENDIF
         ENDIF
      ENDIF
   ENDIF
C---------CALCULATE REACTION TIME FOR THE VEHICLE WHICH STOPPED IN THE INTERSECTION
   IF(IPASS(I).EQ.1)THEN
      XPASS(I)=Y(NSTOPBAR(I))-Y(NP)
      TPASS(I)=(NSTOPBAR(I)-NP)/60.0
   ENDIF
   DO NP=NP1,NNP
      IF(RE(C(NP+NDELT)).GT.RE(C(NP)))THEN
         IF(RE(C(NP+2*NDELT)).GT.RE(C(NP+NDELT)))THEN
            IF(RE(C(NP+3*NDELT)).GT.RE(C(NP+2*NDELT)))THEN
               IF(RE(C(NP+4*NDELT)).GT.RE(C(NP+3*NDELT)))THEN
                  IF(RE(C(NP+5*NDELT)).GT.RE(C(NP+4*NDELT)))THEN
                     C-----------------------NP IS THE FIRST CONTINUOUS ACCELERATION AFTER YELLOW
                     BEGINING
                  ENDIF
               ENDIF
            ENDIF
         ENDIF
      ENDIF
   ENDIF
   GOTO 40
ENDIF
ENDIF
ENDDO
ENDIF
ENDIF
ENDIF
ENDDO
40 CONTINUE
C==========CALCULATE REACTION TIME AND DECELERATION RATE FOR THE STOPPED VEHICLE
ELSE
C----------CALCULATE REACTION TIME
DO NP=NPI,NNP
  IF(BRAKE(NP).GT.0.0)THEN
C----------NP4 IS THE FIRST BRAKE AFTER THE YELLOW BEGINNING
    NP4=NP
    TREACTION(I)=STIME(NP4)-STIME(NP1)
  GOTO 50
ENDIF
ENDDO
50 CONTINUE
C----------CALCULATE DECELERATION RATE
DO NP=NP4,NNP
  IF(SPEED(NP).LT.5.0)THEN
    DECELERATION(I)=-(SPEED(NP4)-SPEED(NP))
  GOTO 60
ENDIF
ENDDO
60 CONTINUE
ENDIF
WRITE(2,70)NAMESECTION(I),DISTANCE(I),ISTOP(I),IPASS(I),
  TREACTION(I),SPEED(NP1),DECELERATION(I)*1.467,IDILEMMA(I),
  DISTANCE(I)/SPEED(NP1)*3.281/1.467,XPASS(I),TPASS(I)
ENDDO
70 FORMAT(X,I3,X,F7.3,4X,I1,5X,I1,X,F8.3,4X,F8.3,4X,F8.3,
  5X,F8.3,12X,I1,2X,F8.3,2X,F8.3,2X,F8.3)
80 FORMAT(A100)
STOP
END
APPENDIX B. INVESTIGATION FORM OF SIMULATOR

EXPERIMENT
1. Did you experience red-light running before in the real world?

   Yes ______   No______

2. If you did, how often?

   Per day ___   Per several days___   Per week ___   Per several weeks___
   More than one month___   Never Run Red Light ___

3. If you did, what is your reason to do that?

   To avoid traffic delay at the intersection___
   Inattentive driving___
   Incapable of stopping during the yellow signal phase because of poor judgment___
   Others (please specify the reason)

                                                                                   

4. Is red-light running a dangerous behavior?

   Yes ______   No______

5. In the real world, when you are approaching a signalized intersection, if traffic light turns yellow, do you feel sometimes it is not easy to decide whether stop or cross the intersection?

   Yes ______   No______

6. If the above situation described in question #5 happened to you before, did it result in your red-light running?

   Yes ______   No______

6. If the above situation described in question #5 happened to you before, did it result in your uncomfortable stop?

   Yes ______   No______

7. Do you think additional information to help drivers decide whether stop or go at signalized intersections is significant?

   Yes ______   No______

8. Do you think the pavement marking design can help you make stop-go decision at signalized intersections?
9. During the course of the experiments, when you encountered yellow phases, did the pavement marking cause you confused or uncomfortable when you made stop-go decision?

Yes ______   No______

10. Do you agree that the pavement marking should be applied to the real road?

Yes ______   No______

11. How do you evaluate the fidelity of the whole simulation experiment?

1____   2____   3____   4____   5____
Poor Needs Improvement Satisfactory Good Excellent
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VITAE
Xuedong Yan

Education

2001-2005 Ph.D. in Transportation Engineering, May 2005, University of Central Florida, Orlando, Florida
2001-2003 M.S. in Transportation Engineering, May 2003, University of Central Florida, Orlando, Florida
1994-1999 B.S. in Urban Planning, June 1999, Xi’an University of Architecture & Technology, Xi’an, China

Experience

1999-2000 Teacher Assistant, in Beijing Polytechnic University
2001-2005 Research Assistant, in University of Central Florida

Selected Publications


