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Studies of Driver Behaviors and Traffic Flow Characteristics at Roadway Intersections

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To the Graduate Council:

I am submitting herewith a dissertation written by Qiang Yang entitled "Studies of Driver Behaviors and Traffic Flow Characteristics at Roadway Intersections." I have examined the final electronic copy of this dissertation for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, with a major in Civil Engineering.

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Studies of Driver Behaviors and Traffic Flow Characteristics at Roadway Intersections

A Dissertation Presented for the

Doctor of Philosophy

Degree

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Qiang Yang

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Dedicated to my parents, Shaogu Yang and Rungeng Jiang, and my brother, Sheng Yang, for their endless patience, support, and love.

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ABSTRACT

The performances of intersections and driveway access points are crucial to a road network in terms of efficiency and safety. Driver behavior and traffic flow characteristics at these locations are relatively complex. To better understand these issues and potentially provide guidance to engineers in their designs, a series of studies were performed on the driver behavior and traffic characteristics at intersections and driveway access points based on field experiments or observations.

First, a countdown timers study was performed in China about their influences on driver behavior. It was found that the presence of countdown timers may encourage yellow running behavior and late entry into intersections in China. Second, a phase gradient method was proposed for the general application purpose to the studies of driver behavior and traffic characteristics at signalized intersections. A case study on red-light cameras was performed at Knoxville, TN. Third, a study was performed to learn the legal issues and arguments about the usage of red-light cameras for the purpose of generating profits. A variety of engineering measures, mainly dealing with the setting of the traffic signal, which could be potentially used by municipalities or camera vendors to trap red-light runners and thus generating more revenues from the camera system are discussed. Finally, an experiment was conducted to simulate the right-turn issues, which impact the safety and operation efficiency at intersections or driveway access points. Two turn lane geometric parameters, angle-of-turn and tangent, and their influences on driver behavior and traffic flow characteristics were studied.

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CHAPTER 1 INTRODUCTION

The performance of intersections is crucial to a road network in terms of efficiency and safety, as vehicles, pedestrians, and other roadway users interact with each other intensively at these locations. Retting et al. (1998) reported that about 1 million collisions occurred at signalized intersections in the United States annually. Intersections are also always bottlenecks in the operation of a road network due to delays in compromising the right-of-way. For this reason, plenty of studies were performed in the past to evaluate the performance of signalized intersections, and to explore potential improvements that can be implemented.

At signalized intersections, the behaviors of driver and traffic are concerned mostly at transitions of signal phases, e.g. the onset of yellow or green. Decision makings and traffic flow variations are complex at the transitions of signal phases. The concern of safety and efficiency issues are also focused on these periods of time in a signal cycle. For this reason, engineers implement some devices like red-light camera and countdown timers to improve the operation performance and safety at signalized intersections. In this dissertation, several studies were performed at signalized intersections related to these devices in China and the US. Their influences on driver behavior and traffic characteristics were researched.

Besides the effects on driver behavior and traffic operation, the operation of red-light cameras has been widely argued for its function in generating profits. While legal disputes and public debates continue regarding the implementation of red-light cameras, municipalities and camera vendors try to make more profits out of the camera systems. In this dissertation, one of the chapters also discusses different possible engineering measures, mainly related to the signal timing, which may influence the frequency red-light running and thus could be potentially employed by red-light camera providers and municipalities for making profits from the camera systems. Their impacts on safety and efficiency of intersection operation also will be discussed.

Based on the survey of previous studies at signalized intersections, it was found that video camera is the most commonly used tool for data collection and the video data analysis and processing is time-consuming and labor-intensive. Depending on the purpose of the study, video data are manually processed in a specific way. There is not such a method that can be generally used to study various driver behavior and traffic characteristics at signalized intersections. Due to the limitation of the current method, a new concept, phase gradient method, is proposed in this dissertation for the general purpose of studying driver behavior and traffic characteristics at signalized intersections. The objective is to propose a method, which simplifies data collection and processing. The method relies simply on vehicle counting data at the stop bar location.

Finally, a study is performed on the turning movements at intersections and driveway access points which also impact the safety and efficiency of the traffic operation at these locations. The objective is to investigate the influence of two turn-lane geometric parameters, angle-of-turn and tangent, on right-turn driver behavior and traffic flow characteristics. Both parameters are not considered in the latest version of Highway Capacity Manual in the calculation of right-turn traffic capacity. Therefore, the study may provide potential guidance to engineers in the design of intersections and driveway lanes.

The dissertation is organized in journal article formats since each chapter is either published, submitted, or to be submitted to an academic journal. Following this chapter, the second chapter is the study of countdown timers on driver behavior in China. The third chapter proposes the phase gradient method. The fourth chapter discusses the engineering measures for profiting red-light cameras. The fifth chapter is the study of the influence of angle-of-turn and tangent on right-turn traffic. Conclusions are drawn and future works are recommended in the sixth chapter.

CHAPTER 2 EFFECTS OF COUNTDOWN TIMERS ON DRIVER BEHAVIOR AFTER YELLOW ONSET AT CHINESE INTERSECTIONS

(This chapter is a slightly revised version of a paper published in Traffic Injury Prevention by Kejun Long, Lee D. Han, Qiang Yang)

2.1 Introduction

A traffic signal countdown timer is a device that displays the remaining time, typically in seconds, of the current signal phase. While such devices are yet to be widely deployed in the US for vehicular movement phases, they are quite common for pedestrian walk phases (Singer et al. 2005, Bundy et al. 2007, Pulugurtha et al. 2010). A green phase countdown timer can serve as a mounting warning about the imminent termination of the right of way and can arguably reduce red-light running and other potential conflicts. When implemented during the red phase, a countdown timer may also ready motorists for the forthcoming onset of the green phase and reduce start-up lost time. This paper focuses on several countdown timers deployed in Changsha, the capital of Hunan province, China. Specifically, the paper evaluates the effect of countdown timers on driver behavior after the onset of yellow and subsequent red-light running violations.

Countdown timers are thought to have the potential of improving capacity, reducing right-angle crashes, and easing driver anxiety while in the queue. However, in practice, the expected effects of countdown timers have not always been realized. Kidwai et al.

(2005) found that countdown timers have "little effect" on capacity but did reduce the number of red-light running violations by some 50% in Malaysia. By contrast, Chen et al. (2007) argued that countdown timers had negative effects on intersection safety. Traffic accident data from 187 intersections in Taiwan from 2003 to 2006 show that while the accident rate during the red phase decreased by about 50 percent after the installation of countdown timers, the accident rate during the green phase increased by almost 100 percent. Similarly, Chiou et al. (2010) found that countdown timers lowered the driver's tendency to stop, lengthened the dilemma zone by 28 meters, and resulted in higher rear-end crash rates at the end of the green phase. Even though countdown timers reduced start-up lost time and saturated headway, no significant safety improvement was observed.

Ibrahim et al. (2008) found that while countdown timers had a limited effect on start-up lost time, improvement of the queue discharging rate was found to be significant. Interestingly, the frequency of red-light violations increased after the installation of the countdown timers.

Lum and Halim (2006) studied longitudinal, or lasting, effects of countdown timers on intersection operations. Results from their before-and-after study in Singapore showed a 65% decrease in red-light violations 1.5 months after the installation of countdown timers. However, red-light violations rebounded to before-installation levels after another six months. Similarly, Puan and Ismail (2010) found that countdown timers led to inadequate yellow length and did not address dilemma zone problems.

Limanond et al. (2009) found that countdown timers in Bangkok reduced average start-up lost time by about one second, which increased intersection capacity slightly. However, the saturated flow rate showed little change. A further investigation by Limanond et al. (2010) indicated that a countdown timer could lengthen the average saturation headway during the green phase slightly and lessen the start-up lost time at the beginning of the green phase by 22%. It was also found that while countdown timers had little impact on driver behavior during the yellow phase, they reduced red-light violations. Sharma et al. (2009) studied the effect of countdown timers on headway distribution. Both start-up lost time and transition lost time were found to decrease after the installation of the timers.

Newton et al. (1997) used a driving simulator to study the effects of a flashing yellow signal, which functions somewhat similarly to a countdown timer, on driver behavior. Results indicated that the timer reduced red-light violations and increased variability in the potential of conflicting decisions between successive vehicles.

Köll et al. (2004) examined driver behavior when approaching a flashing green at ten intersections in Switzerland, Austria, and Germany. They found that early stops increased substantially with the flashing green, resulting in a minimal dilemma zone and a reduced likelihood of right-angle collisions.

Ma et al. (2010) reported that countdown timers encouraged drivers to cross the stop line at higher speeds during the yellow phase in Shanghai, China; countdown timers increased intersection capacity, smoothed driver’s responses to the phase transition, and also significantly reduced red-light violations. However, the timers increased the possibility of collisions with unexpectedly crossing vehicles or with pedestrians due to higher approach speeds before the onset of the yellow.

Wu and Juan (2009) established a logistic decision-making model, and compared driver behavior in intersections with and without countdown timers in China. Many other researchers (Genya et al. 2004, Caird et al. 2007, Yan et al. 2009, Noor et al.2010, and Rosenbloom, 2009) have conducted similar studies regarding driver's perceptions-reactions, decision-making, and red-light violations at intersections.

Thus, while the aforementioned studies report observations from particular locales, their findings tend to contradict one another. Traffic conditions and driving culture differ from one country to another, and, in many cases, from one city to another. Additional studies are necessary to provide useful perspectives. Many countdown timers have been installed in China, but their effects are seldom studied comprehensively. The present study covers four comparable intersections in China, two with countdown timers and two without. Data collected for this study include driver decisions after the onset of yellow and vehicle entry times. Driver decisions and the distribution of vehicle entry times are compared at intersections with and without countdown timers.

2.2 Methods

2.2.1 Data Collection

Four intersections in downtown Changsha, China were selected for this study. These intersections share similar traffic demands, roadway geometries, and surrounding environments. Two intersections are equipped with countdown timers while the other two are not; see Figure 2.1. Some general characteristics of the four intersections are provided in Table 2.1.

High-definition video cameras were installed at vantage points to record traffic operations and signal phasing changes throughout the study period. The location of the stop line and distances upstream of it were marked for each study site so the exact distance of a vehicle from the stop line at the onset of the yellow could be determined. This allowed the calculation of each vehicle's speed as it approached the intersection.

The four study sites were monitored simultaneously with video cameras from 8:00 AM to 11:00 AM, the normal morning peak, on two successive sunny workdays. A total of 24 hours of video footage were obtained.

2.2.2 Data Processing

Only one straight-through lane away from curbs on each approach was selected to minimize the effect of "roadside friction" on driver behaviors. Because trucks are prohibited in the study area, only passenger cars and city buses were present. Entities

Figure 2.1 Locations of the study intersections

Table 2.1 Characteristics of the study intersections

No.	Intersection Location	Layout	Count- down Timer	Number of lanes in approach	Volume per lane (pcu/h·lane)	Cycle Length (seconds)	Green Length (seconds) [a]
1	Wuyi Road at Furong Road	4-legged	Yes	5	585	105	40
2	Yingpan Road at Furong Road	4-legged	No	5	566	128	42
3	Bayi Road at Yingbin Street	4-legged	Yes	4	408	100	40
4	Dongfeng Street at Yingpan Road	4-legged	No	4	417	115	45

[a] yellow interval is exactly 3.0 seconds at all intersections.

such as pedestrians and bicycles were excluded. Other abnormal situations such as illegal parking maneuvers were identified and removed so as not to taint the data.

Video footage was studied frame by frame to extract the precise timing of phase changing and stop line crossing events. As the signal turned yellow, the exact time was recorded, to a precision of frame rate of 25 Hz (1/25 second). At the same time, the vehicle nearest the stop line was identified as the target vehicle and its distance to the stop line recorded. The driver decision and the exact crossing time of the target vehicle (if the decision was to cross) were recorded. Sample data records of phase changing times and driver decisions/actions are shown in Table 2.2.

Table 2.2 A sample of data extracted from video

No	Onset of Yellow (*hh*:*mm*:*ss*,*ff*)	Distance to Stop Line at Onset of Yellow (*m*)	Onset of Red (*hh*:*mm*:*ss*,*ff*)	Vehicle Entry Time (*hh*:*mm*:*ss*,*ff*)	Driver Action [a]			Vehicle Type
					S	C	R	
1	08:11:35,24	10	08:11:38,24	08:11,37,17		1		Car
2	08:24:41,18	22	08:24:44,18	08:24,44,20			1	Car
3	08:26:15,10	23	08:26:18,10	-	1			Car

[a] S: vehicle stopped at stop line, C: vehicle crossed legally during yellow, R: vehicle ran red light.

2.2.3 Data Analysis

- Binary Logistical Regression Analysis

Binary logistical regression (BLR) analysis was employed to model driver decisions during the yellow at intersections with and without countdown timers. Driver decision type is a binary dependent variable ***Y***, where *Y*=1 denotes a decision to stop and *Y*=0 a decision to continue through the intersection. Independent variables include vehicle distance to the stop line (***D***), velocity at onset of yellow (***V***), and the presence of a countdown timer (***C***), where ***D*** and ***V*** are continuous and *C* is discrete. We use *C* = 1 and 0 to denote the presence and absence of a countdown timer, respectively. The driver's stop/go decision can then be modeled by the follow equation:

$$\text{Logit}(P) = \ln\left(\frac{P}{1-P}\right) = \beta_0 + \beta_1 V + \beta_2 D + \beta_3 C \tag{2-1}$$

where *P* is the probability of a decision to stop (*Y*=1), *Logit*( ) is the natural logarithm of the odds ratio, and $\boldsymbol{\beta}_i$s are model parameters.

- Nonparametric Test

Prior to comparing the means for vehicle entry times, Kolmogorov and Shapiro-Wilk tests were used to examine the normality of the data. Because both tests showed that the data significantly differ from a normal distribution whether a countdown timer is present (d.f.=283, sig.=0.000) or not (d.f.=149, sig.=0.000), a nonparametric test was used. (Vehicle entry histograms in Figure 2.2 show the non-normality of the data.)

2.3 Results

2.3.1 Driver Decisions at the Yellow Onset Time

Upon noticing the onset of the yellow phase, the driver of the target vehicle can either brake and stop or proceed to cross; obviously the action to cross could be by choice or by default depending on whether there is time to react. While this decision may depend on many factors, this study focuses solely on actual demonstrated behaviors, hence decisions, as observed in the field. Because we cannot know the mindset of every driver or the performance of each vehicle, speculative conjectures on these matters may be interesting, but not helpful. The following are some observations.

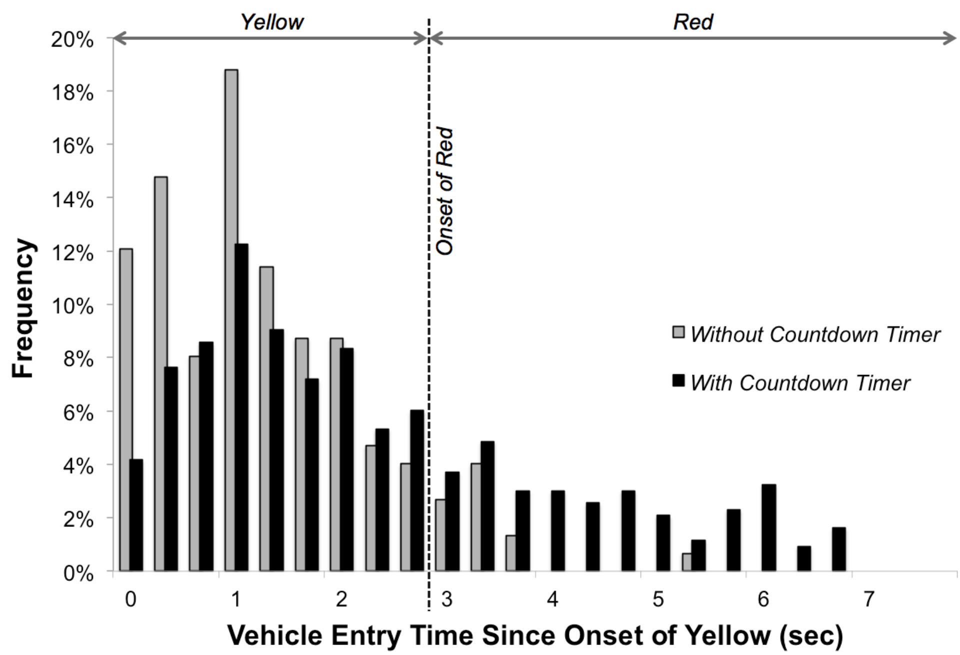


Figure 2.2 Vehicle entry times after the onset of yellow

- Field data show that after the onset of the yellow at intersections without countdown timers, vehicles that crossed the stop line outnumbered those that stopped by a ratio of just under 3 to 2 (58.7% vs. 41.2%); see Table 2.3. At intersections equipped with countdown timers, this ratio increased significantly to near 4 to 1 (78.8% vs. 21.1%). With countdown timers, the proportion of target vehicles that stopped dropped substantially from 41.2% to 21.1%, while the proportion of vehicles that crossed rose from 58.7% to 78.8%. The likelihood of red light running increased from 8.2 to 15.4 percent, or nearly doubled.

Table 2.3 Descriptive statistics of driver stop/go decisions

Statistics		***With Timer***			***Without Timer***		
		Go	Stop	Run Red	Go	Stop	Run Red
Distance to stop line (m)	Average	9	33.4	24.0	10.5	20.0	22.9
	Std. deviation	6.7	21.7	13.6	5.8	13.3	13.1
Velocity (km/h)	Average	19.9	23.3	20.1	26.0	20.9	21.1
	Std. deviation	9.35	12.4	8.0	11.9	11.8	8.6
Frequency		78	26	19	49	40	8
Percent (%)		63.4	21.1	15.4	50.5	41.2	8.2

- For all of the intersections studied, when the distance to the stop line is less than 10 meters, most drivers chose to enter the intersection regardless of the vehicle's velocity. When this distance increased to above 30 meters, most drivers chose to stop. In the zone between 10 and 30 meters, the majority of the drivers at countdown timer equipped intersections entered the intersection (see Figure 2.3), but this was not the case at intersections without countdown timers, where driver decisions were mixed (see Figure 2.4).
- Table 2.3 shows that the average distances to the stop line, for drivers who decided to stop, were 33.4 meters with countdown timers and 20.0 meters without; on the other hand, for drivers who chose to cross, the distances were 9 and 10.5 meters,

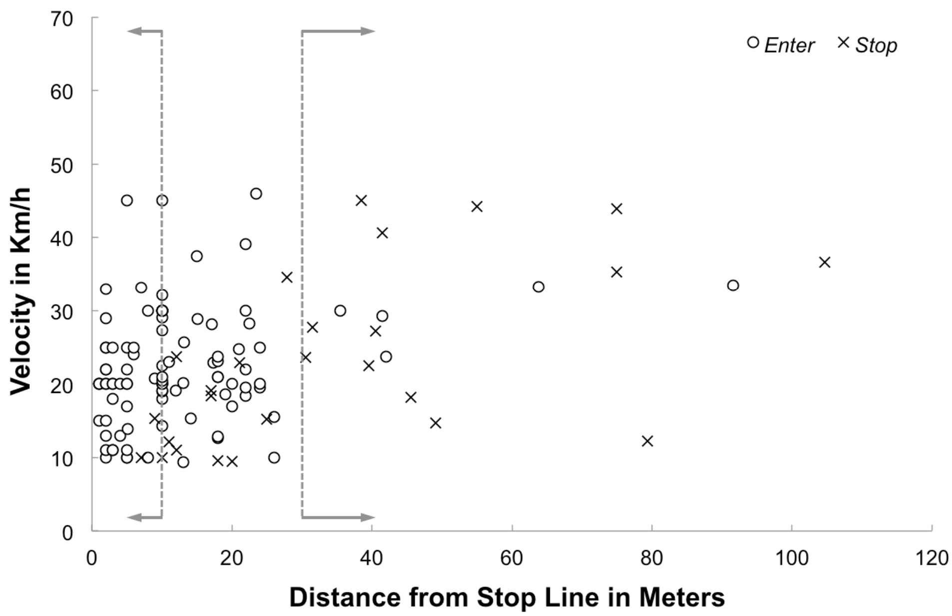


Figure 2.3 Driver stop/go decisions with countdown timers

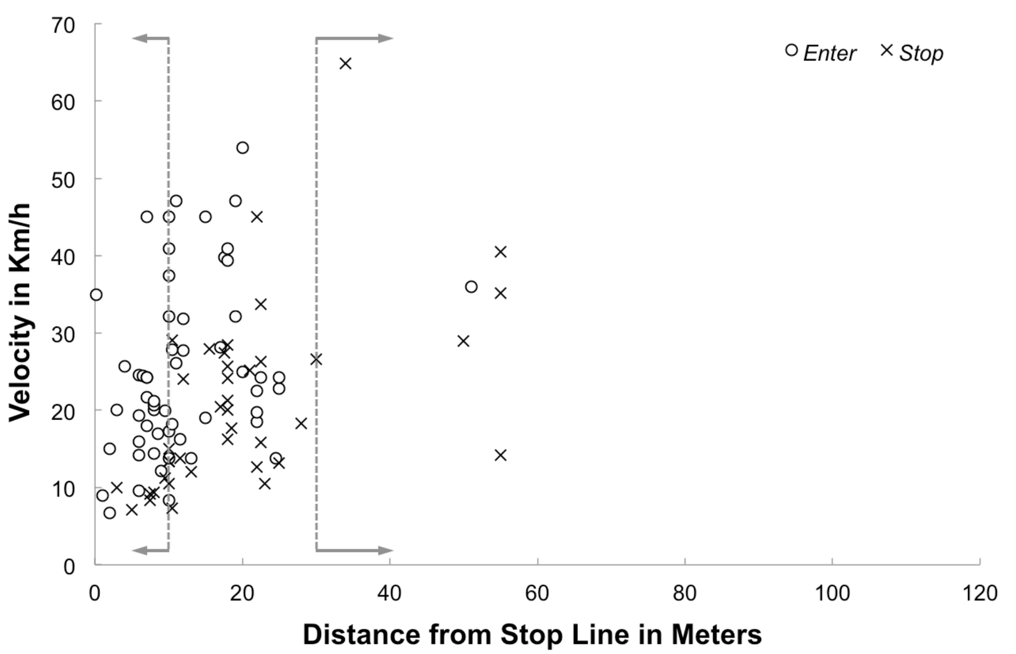


Figure 2.4 Driver stop/go decisions without countdown timers

respectively. Drivers who chose to stop at countdown timer equipped intersections were further away from the stop line than the control group; drivers who chose to cross at countdown timer equipped intersections were slightly closer to the stop line. It appears that the information provided by countdown timers led appreciable portion of drivers to different decisions at different distances from the stop line. Without countdown timers, it seems that drivers were less able to predict the changing of the phase and thus showed more randomness in their decisions. This can be verified as data points from two driver decision groups in Figure 2.3 are more clearly separated from each other than those in Figure 2.4.

- When the distance to the stop line is greater than 20 meters, drivers at intersections with countdown timers seem to run the red light more often than those at intersections without the device. Overall, the odds ratio of red light running at countdown timer equipped intersections is 15.4/(100-15.4)=0.182; at non-timer intersections, it is 8.2/(100-8.2)=0.089. Therefore, drivers are about twice most likely to run a red light when countdown timers are present. Even more seriously, some drivers may intrude into intersections even if their distances to the stop line are exceedingly long, e.g. 60 meters.

Statistical results from testing the effects of countdown timers on driver behavior with BLR analysis are shown in Table 2.4. Statistical significance is evident for the presence of a countdown timer (p = 0.000), the distance from the target vehicle to the stop line (p = 0.000), and vehicle velocity (p = 0.001).

Table 2.4 BLR analysis of results regarding driver stop/go decisions

Sources	β_i	***S.E.***	***Wald***	***df***	***P value***	***Exp(β_i)***
Distance to Stop Line, ***D***	.083	.014	35.756	1	.000	.282
Velocity, ***V***	-.070	.017	16.602	1	.001	.933
Countdown Timer, ***C***	-1.266	.331	14.584	1	.000	.282
Constant$\boldsymbol{\beta}_0$	-.145	.400	.132	1	----	.865

Substituting $\boldsymbol{\beta}_i$s in Eq. (2-1) with coefficients obtained from BLR analysis, the logistical regression model of the probability of braking to stop (Y=1) at the onset of the yellow is given by the following equation.

$$\ln\frac{P_{Y=1}}{1-P_{Y=1}} = -0.145 + 0.083D - 0.070V - 1.266\times C(1) \qquad (2\text{-}2)$$

For intersections with a countdown timer (C = 1), we have:

$$\ln\frac{P_{Y=1}}{1-P_{Y=1}} = -0.145 + 0.083D - 0.070V - 1.266 \qquad (2\text{-}3)$$

For cases without a countdown timer (C = 0), we have:

$$\ln\frac{P_{Y=1}}{1-P_{Y=1}} = -0.145 + 0.083D - 0.070V \qquad (2\text{-}4)$$

Subtracting Eq. (2-4) from Eq. (2-3) yields:

$$\frac{OddsRatio_{C=1}}{OddsRatio_{C=0}} = \frac{\left(\frac{P_{Y=1}}{1-P_{Y=1}}\right)_{C=1}}{\left(\frac{P_{Y=1}}{1-P_{Y=1}}\right)_{C=0}} = \exp(-1.266) = 0.282 \quad (2\text{-}5)$$

Eq. (2-5) indicates that given the same vehicle location and velocity at the onset of yellow, the odds ratio of stopping at countdown timer equipped intersections is 0.282 times that at intersections without countdown timers. That is, when countdown timers are present, drivers are 3 to 4 times more likely to cross than stop after the onset of yellow, which may potentially result in more red light violations.

The probability curve of a vehicle stopping can be generated from Eq. (2-3) and (2-4), as shown in Figure 2.5, which suggests that the average probability of a vehicle stopping is reduced by a significant percentage (10~60%) where a countdown timer is present when the vehicle is less than 40 meters from the stop line. With increasing distance to the stop line, the difference between the two curves diminishes quickly until the two curves merge at nearly 100%. The vehicle stopping probability is almost always higher when countdown timers are absent.

2.3.2 Distribution of Vehicle Entry Times

Observed vehicle entry times are charted in Figure 2.2, which charts the vehicle crossing frequency against the vehicle entry time, lumped into intervals of 1/3 of a second. If every driver tried to avoid running the red light after the onset of the yellow, all crossing activities, shown as vertical columns, should take place during the yellow. However,

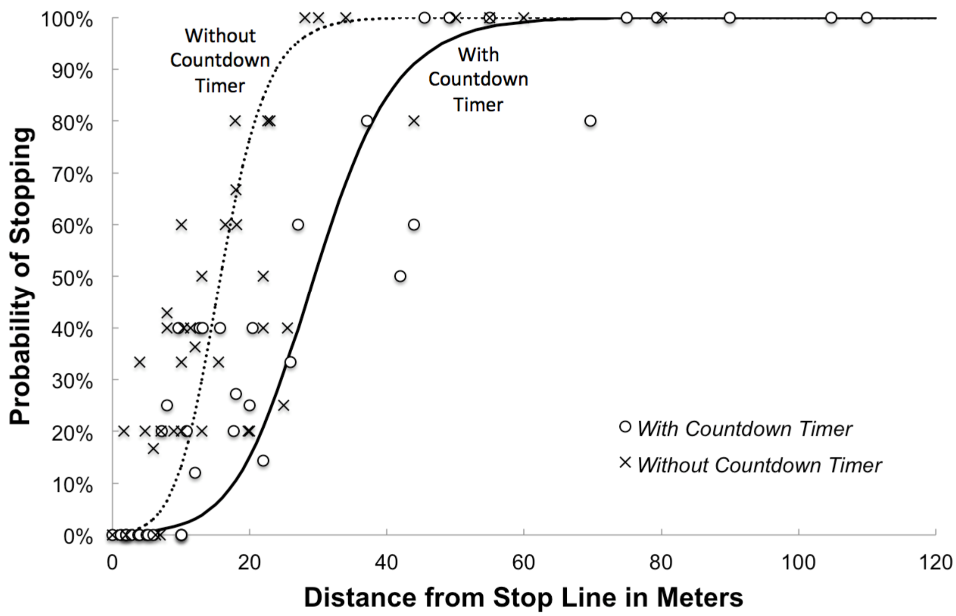


Figure 2.5 Probability of braking to stop during yellow interval

Figure 2.2 tells a very different story; it appears that red light running was quite frequent where no countdown device is present, but was far worse where countdown timers are installed. Without countdown timers, the frequency of crossing vehicles dropped quickly after the onset of yellow, and virtually no vehicle crossed the stop line 1 second into the red phase; with countdown timers, vehicles blatantly streamed into the intersection 2, 3, even 4 seconds after the onset of red.

Descriptive statistics for vehicle entry times yielded a mean vehicle entry time of 3.0s (SD = 1.79s) for countdown timer equipped intersections and 1.45s (SD = 1.00s), respectively. For countdown timer equipped intersections, vehicle entry times ranged between 0.4 to 7.8s, while for intersections without countdown timers, they ranged from 0.0 to 5.6 s. The vehicle entry time was longer lasting on average and more dispersed where countdown timers are present. Likewise, the red light violation rate was higher with countdown timers (15.4% vs. 8.2%); see Table 2.3.

A nonparametric test, the Mann-Whitney method with rank-sum test, was applied to test the effect of countdown timers on vehicle entry time with the assumption that the study sites were independent of one another. With a 2-tailed significance level of 0.05, the Z value of -8.382 ($p = 0.000$) is outside of the range from -1.96 to 1.96, which reveals a significant difference in vehicle entry time between intersections with and without countdown devices.

2.4 Discussion

This study investigated how countdown timers influence driver behavior at intersections in Changsha, China. Results of the study indicate that the driver's decisions at the onset of yellow are influenced by various factors, including vehicle distance from the stop line, vehicle velocity, and the presence of countdown timers. These findings concur with those of studies conducted in other countries (Elmitiny *et al.* 2010, Olson and Rothery 1961, and Caird *et al.* 2007). Among factors considered in this study, the distance to the stop line is the most critical. While countdown timers do influence driver decisions considerably, other factors may also influence the driver's decision making (Verghese and Alex 2010, Gates *et al.* 2007). The effects of these factors should be studied in China in the future.

Statistical results indicate that, under the same conditions, the odds that a target vehicle would choose to stop at a countdown timer equipped intersection is lower than that at intersections without timers. Furthermore, the study shows that the presence of countdown devices may contribute to more drivers crossing the stop line after the onset of yellow, which in turn can lead to an increased probability of red light violations. This is consistent with the findings of Chen (2007) and Ibrahim (2008).

A nonparametric test found a significant difference between the mean values of vehicle entry times with and without countdown timers. Vehicles approaching intersections without countdown timers typically cross the stop line within 4 seconds into the yellow, with virtually all red light violations taking place during the first second of the red phase.

In contrast, at intersections with timers, the maximum vehicle entry time extends up to 7 seconds after the onset of the yellow; in other words, red light violations were blatant and rampant for the first four seconds of the red phase. This is quite an alarming finding and something for the local traffic, safety, and enforcement agencies to look into seriously.

Thus, countdown timers were found, somewhat unexpectedly, to increase the frequency of red light violations. While Ibrahim (2008) observed similar phenomena in Malaysia, Limanond (2010) and Kidwai (2005) reached opposite conclusions in Thailand and Malaysia respectively. In cases where countdown timers reduced the frequency of red light violations, we suspect that regional driving culture, local enforcement activities, and other factors may play an important but yet to be substantiated role.

In this study, the presence of countdown timers did not seem to lead drivers to better decisions or safer behaviors. In some cases drivers maintained high speed or even accelerated to beat the red light, which obviously can lead to crashes and casualties. As such, vehicle speed profile approaching and through the intersection should be studied in the future.

2.5 Conclusions

This study investigated the influence of countdown timers on driving behavior at four urban intersections in China. Logistical regression models were built to model the driver's stop/go decisions. A nonparametric test was employed to study the influence of countdown timers on vehicle entry times. Results of the study suggest that when countdown timers are present drivers are more likely to cross the intersection after the onset of yellow. It was also found that the presence of countdown timers may contribute to late entry into intersections and, consequently, to dangerous red light running behaviors.

Future research is needed, particularly in China where countdown timers are widely deployed. This study was limited to four intersections in the same metropolitan area. Similar studies should be conducted in other locales, and, perhaps, in nonurban settings to verify the generalizability of the present findings. In addition, this study focused on driver behavior in straight-through lanes; driver behaviors might be different for turning activities. Furthermore, studies should be conducted on the influence of conflict and interference with vehicles in adjacent lanes and with bicycles/pedestrians, which are abundant at China's intersections.

CHAPTER 3 A SIMPLER TECHNIQUE FOR DRIVER BEHAVIOR STUDIES AT SIGNALIZED INTERSECTIONS – AN INTRODUCTION TO THE CONCEPT OF PHASE GRADIENTS

(This chapter is a slightly revised version of a paper to be submitted by Qiang Yang, Lee D. Han)

3.1 Introduction

Signalized intersections are crucial to the safe and efficient operation of a road network. Their performance is evaluated with many different metrics such as capacity, saturation flow rate, delay, etc. Various studies were conducted to learn traffic characteristics and driver behavior at signalized intersections. Normally, video camera or other data logging tools are used to record the behavior of individual vehicles such as position, speed, stop/go decision, etc. Researchers use these data to learn drivers' behavior individually or the characteristics of traffic or the intersection collectively. There is no uniform or standard method to collect the data and learn driver and traffic behaviors at signalized intersections. Nevertheless, almost all previous studies using video camera have to process the frame-by-frame video image either manually or automatically, which is labor- or computation-intensive. Considering a common frame rate of 30 frames per second in the U.S., one hour of video produces 108,000 video images.

At signalized intersections, the behaviors of driver and traffic are mostly concerned at the transition of signal phases, e.g. the onset of yellow or green. During signal phase transitions, the traffic flow rate changes with a pattern at a particular intersection. In this study, we define such a pattern as phase gradient. This paper introduces a method to generate phase gradients based on simple vehicle counting data at the stop bar location and use it to estimate some parameters concerning traffic or driver behavior at intersections. These parameters can be used to evaluate the performance of the intersection.

The objective of this study is to propose a method which can be used by researchers and engineers to learn traffic and driver behavior at signalized intersections. The method standardize the data collection and processing and should be generally applicable to characterize a signalized intersection, estimate commonly used traffic flow and driver behavior parameters, and help identifying potential safety and operational efficiency issues. The method also will simplify the data collection and processing for such studies at signalized intersections.

The paper is organized as follows. After this introduction, the following section presents a literature survey of several different categories of signalized intersection related studies that had been performed so far. The methodology is then developed. Subsequently, a case study is performed to demonstrate the proposed method. Potential applications also are introduced. Conclusions are drawn at the end.

3.2 Literature Review

No previous research was found that studied driver behavior or traffic flow characteristics at signalized intersections by observing the pattern of the traffic flow rate variation during signal transitions. *Qureshi and Han (2001)* employed a similar method, queuing accumulation polygon (QAP), to study right turn on red delays at signalized intersections. However, the study was based on the vehicle accumulation in queue instead of vehicle crossings at the stop location. The following section presents a survey of several different categories of driver behavior and traffic flow studies at signalized intersections.

3.2.1 Capacity/Delay Studies

The capacity and delay are among the mostly concerned metrics to evaluate operational efficiency performance at signalized intersections. *Perez-Cartagena and Tarko (2004)* deployed a video recorder and four cameras to estimate the saturation flow rate, start-up lost time, and green extension at signalized intersections in Indiana, and predicted delays with these parameters. *Li and Prevedouros (2002)* compared three different methodologies of estimating the saturation flow rate, start-up lost time, and start-up response time using data collected with videotapes in Hawaii. Aerial photos or eye-in-the-sky alternatives, which may be more costly, were also used to estimate traffic statistical profiles like the headway and queue length at intersections *(Puri et al. 2007a, Puri et al. 2007b)*. *Hadiuzzaman et al. (2008)* measured the saturation flow rate with data collected using digital video cameras at intersections in non-lane based traffic conditions, which were prevailing in developing countries, and developed the equivalent passenger car unit concept. With the data, *Hadiuzzaman and Rahman (2010)* also developed saturation flow and delay models for signalized intersections in Bangladesh. *Rahman et al. (2008)* studied the capacity of signalized intersections in Japan, with portable digital video cameras, considering the impact of professional taxi drivers in the traffic stream. *Potts et al. (2007)* investigated the relationship between the intersection discharging saturation flow rate and the lane width using video cameras.

3.2.2 Driver Response and Decision Making

Drivers' responses during signal transitions like stop/go decision making and red-light running have been widely researched in different countries and regions. *Olson and Rothery (1961)* recorded positions of vehicles at intersections by manually tripping the shutter at the onset of yellow and modeled motorist responses to the yellow phase in different distances to the intersection, speeds, and yellow phase length conditions. *Lum and Tan (2003)* reported driver responses to an amber blackout in comparison to normal conditions in Singapore with a special purpose data logger and inductance loop sensors. *Bonneson and Zimmerman (2004)* conducted a before-and-after study regarding the effect of the yellow interval duration on the frequency of red-light running. *Bonneson and Son (2003)* also performed a study to build the relationship between red-light running and flow rate, speed, and traffic arrival pattern. Some countermeasures like increasing the yellow interval duration, LED yellow indications, and adding black plates to the signal head were evaluated. *Retting et al. (2008)* also evaluated incremental effects of first lengthening the yellow phase, followed by the introduction of red-light cameras, on red-light running using unattended video cameras. *Zhang et al. (2008)* investigated characteristics of red-light running cases, including the headway of red-light runners and whether captured in dilemma zone, using multiple cameras at intersections. *Verghese and Alex (2010)* used two cameras to collect driver's stop/go decision making data at signalized intersections and modeled its relationship with driver's age and gender along with approach speed, distance to intersection, and vehicle type. *Amer et al. (2010)* built a behavioral model that can be used to simulate driver behaviors at the onset of yellow involving various influential factors and a complete decision making and action process at intersections based on data collected in the experimental road facility in Virginia. *Elmitiny et al. (2010)* modeled driver's stop/go decision and red-light violations using a decision tree based on various traffic and vehicle parameters collected by three cameras.

3.2.3 Dilemma Zone Driver Behavior

Driver's perception-reaction time and decision in dilemma zone at signalized intersections are also widely investigated. *Change et al. (1985)* studied driver responses, like the perception-reaction time and the deceleration rate, to traffic signal change intervals (yellow plus all-red) using timelapse cameras and gave suggestions regarding the timing of signal change intervals accordingly. *Caird et al. (2007)* reported a study of driver's perception-reaction time, stopping performance, and speed profiles across intersection in different age groups using a driving simulator. *Gates et al. (2007)* evaluated stop/go decisions, response times, and brake profiles of drivers who were captured in the dilemma zone using video cameras, and distinguished drivers making different decisions based on driver and vehicle characteristics, intersection conditions, and other ambient factors. *Gates and Noyce (2010)* also investigated the influence of vehicle type, time-of-day, and traffic arrival pattern on various aspects of dilemma zone driver behaviors including brake response time, deceleration rate, and red-light running occurrence.

3.2.4 Countdown Timer

Countdown timers and red-light cameras are two commonly deployed measures to improve the operation safety and efficiency at signalized intersections. Various before-and-after or comparative studies were performed to evaluate the effectiveness of these devices in different countries. *Lum and Halim (2004, 2006)* reported a before-and-after study of evaluating the impact of green signal countdown devices on the driver stop/go decision and red-light running in Singapore using loop sensors and data logger. *Huey and Ragland (2007)* explored the effect of pedestrian countdown signals on driver behavior during amber and red phases in California, US. *Sharma et al. (2009)* presented the effect of countdown timer on queue discharge characteristics at intersections under heterogeneous traffic conditions in India using two cameras. *Limanond et al. (2009, 2010)* also used video camera to investigate the impact of countdown timer on queue discharge characteristics and start-up delays of through movements in Thailand. *Ibrahim et al. (2008) and Wu et al. (2009)* conducted similar studies in Malaysia and China respectively and modeled driver's decision making with logistical models as well. A European study was performed concerning driver's behavior in flashing greens before yellow, which worked similarly as countdown timers *(Koll et al. 2004)*.

3.2.5 Red-light Camera

Kent et al. (1995) investigated signal compliances at signalized intersections with and without red-light cameras in Australia using video taping, and modeled the red-light running behavior as a function of speed, road cross section, lane type, time-of-day, and day-of-week. *Retting et al. (1999a, b)* evaluated the influence of red-light cameras on red-light violations in Virginia and California using video cameras. *Lum and Wong (2002, 2003a, b, c)* conducted a series of study in Singapore on the effect of red-light cameras on driver stopping propensity, red-light running, and vehicle entry times. *Fitzsimmons et al. (2009)* evaluated the effectiveness of red-light cameras in reducing red-light running in Iowa. *Lam et al. (2009)* studied driver behavior, dilemma zone, and capacity at red-light camera equipped intersections in Maryland.

In summary, different equipments, e.g. cameras, videos, loop sensors, were used to learn traffic or driver behavior at signalized intersection in previous studies. Most of these studies somehow relied on the time information of vehicle crossing the stop line. The common ground with these studies shed a light on the development of the phase gradient methodology.

3.3 Methodology

The pattern of the traffic flow variation can be described by the frequency of vehicles crossing the stop bar over time within a signal cycle, which is defined here as departure characteristic curve. Due to the variation of traffic arrival, the curve varies from cycle to cycle. However, after observing over many cycles, the curve should show certain pattern for a particular intersection.

3.3.1 Departure Characteristic Curve

A conceptual departure characteristic curve is shown in Figure 3.1. The curve represents the variation of the traffic flow rate entering the intersection within a signal cycle. After the onset of green, the flow rate increases until the saturation flow rate is reached. When the queue is fully discharged, the flow rate drops to the demand level at the intersection. The flow rate starts to decrease shortly after the onset of yellow and should reach zero toward the end of yellow or the red phase (including all-red if applicable) depending upon the red-light running condition at the intersection.

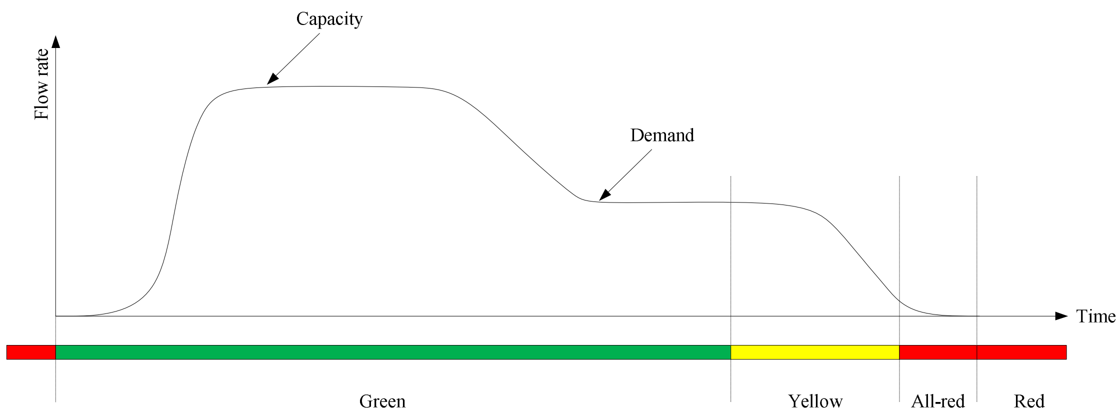


Figure 3.1 Conceptual departure characteristic curve

The curve may vary slightly due to the random arrival of traffic and the variation of ambient conditions (e.g. time-of-day, day-of-week, weather, etc.) at the intersection. To obtain a reliable characteristic curve, the data collection should ensure sufficient duration and expose to these different conditions. The curve should represent the average behavior of the traffic at the specific intersection over many cycles.

The traffic arrival pattern is influential to the shape of the curve. Ideally, traffic arrives randomly and the intersection is not coordinated with nearby traffic signals upstream or downstream in the network. Otherwise, platoon traffic arrival pattern may appear and impose undesirable impact on the shape of the curve, as the vehicle arrival pattern, rather than the characteristics of the intersection, may become dominant in shaping the curve.

The variation of the flow rate during signal phase transitions, i.e. the slopes of the curve at the beginning of green and of yellow, is of special interest in the paper. The following sessions will introduce the yellow and green phase gradients respectively.

3.3.2 Yellow Phase Gradient

The yellow phase gradient part of the departure characteristic curve is shown in Figure 3.2. The flow rate starts to decrease a few seconds (or a faction of one second) after the onset of yellow. The point on the curve, from which the flow rate starts to decrease, represents the earliest time that a driver can decelerate to a complete stop at the stop bar. The time difference between the onset of yellow and this point is defined as the inertia time (*IT*) of crossing traffic, which includes driver's perception-reaction time and brake-deceleration time. The perception-reaction time of drivers at a specific intersection depends on the characteristics of driver population traversing this intersection, and some other factors like visibility, weather, roadside distraction, etc. Assuming the brake-deceleration time reflects the maximum vehicle brake capacity in ideal pavement surface condition, the perception-reaction time of drivers can be derived with Eq. (3-1).

$$t_{perception-reaction} = IT - \frac{v}{15(f \pm G)} \tag{3-1}$$

where, v is the initial vehicle speed in mph. f is the friction factor. G is the approach grade in percentage *(AASHTO 2004)*.

The point, from which the flow rate drops dramatically, represents the time that most drivers choose to stop after the onset of yellow. The time difference between the onset of yellow and this point is defined as the yellow-entry time. The yellow-entry time is an important parameter representing vehicle departure characteristics. The shorter the yellow-entry time, the lower the possibility the drivers run a red light, but also the lower the traffic throughput in a cycle. The length of the yellow-entry time depends on driver characteristics (e.g. aggressiveness), and a number of intersection factors (e.g. yellow duration, pedestrian crosswalk availability, intersection width, etc.). Of special interest, the installation of countdown timers may impact the yellow-entry time significantly, as the device provides drivers the information of the remaining time before the onset of yellow *(Koll et al. 2004, Long et al. 2011)*.

The rate of traffic flow decrease, labeled as slope α in Figure 3.2, represents the average deceleration of vehicles at the intersection. The larger the angle α, the higher the deceleration of vehicles. The slope α can be used to evaluate the deceleration behavior of drivers and associated safety attributes of an intersection. For instance, if the yellow duration of an intersection is insufficient, many drivers may brake hard and decelerate quickly at the end of the yellow phase to avoid running a red which may lead to rear-end collisions.

As shown in Figure 3.2, the red-entry time is defined as the time difference between the termination of yellow and the time the last red-light running vehicle crossing the stop bar. If no vehicle enters the intersection after the onset of the red phase (including all-red if available), the red-entry time is zero. Red-entry time is an important indicator of the intersection safety, since it is associated with the occurrence of right-angle collisions with vehicles on the crossing street. Similar to the yellow-entry time, the red-entry time is also related to driver and intersection characteristics, and can be used to evaluate the effects of red-light camera, signal timing, etc., on intersection safety.

The areas under the yellow phase gradient curve are also indicators of intersection safety and efficiency. The entire area surrounded by the characteristic curve, the horizontal axis, and the vertical axis (area A in Figure 3.1, including area B) represents the total amount of time used by vehicles after the onset of yellow. On one hand, the larger the area, the higher the traffic throughput in a signal cycle; on the other hand, the smaller the area, the lower the possibility of right-angle collisions at the intersection. The shaded area B in the figure indicates the frequency and the severity of red-light running at the intersection.

So far, the description of the phase gradient curve has been conceptual and qualitative. To quantify the phase gradient, a mathematical function could be developed assuming that traffic cutoff time follows a certain distribution. Traffic cutoff time is defined as the time the first stopping vehicle stops at the stop bar in a continuous traffic stream due to the onset of yellow. For each signal cycle, one cutoff time could be obtained for each lane. If observing many cycles, the traffic cutoff times may show a certain distribution with the expected mean value located between the onset of yellow and the onset of red, since drivers are neither willing to stop right after the onset of yellow nor likely to take the risk of running a red to stop right before the onset of red. The exact distribution is unknown before further study is conducted. Assume the probability distribution function of the traffic cutoff time is $f(x;\mu,\sigma^2)$, where *f*() is the probability that the first stopping vehicle appears *x* seconds after the onset of yellow, μ is the mean value of the cutoff time at a particular intersection over many cycles, σ is the standard deviation of the cutoff time. Then, the number (or flow rate) of stopping vehicles over time after the onset of yellow at the intersection can be estimated with a cumulative distribution function of the traffic cutoff time multiplied by the traffic demand.

$$N(x;\mu,\sigma^2) = D * F(x;\mu,\sigma^2) \tag{3-2}$$

where *N*() is the number of stopping vehicles at the time point *x* seconds after the onset of yellow, *D* is the traffic demand, *F*() is the cumulative distribution function of the traffic cutoff time. Then the yellow phase gradient curve, i.e. the number (or flow rate) of crossing vehicles at the stop bar, can be described as

$$Y(x;\mu,\sigma^2) = D - N(x;\mu,\sigma^2) \tag{3-3}$$

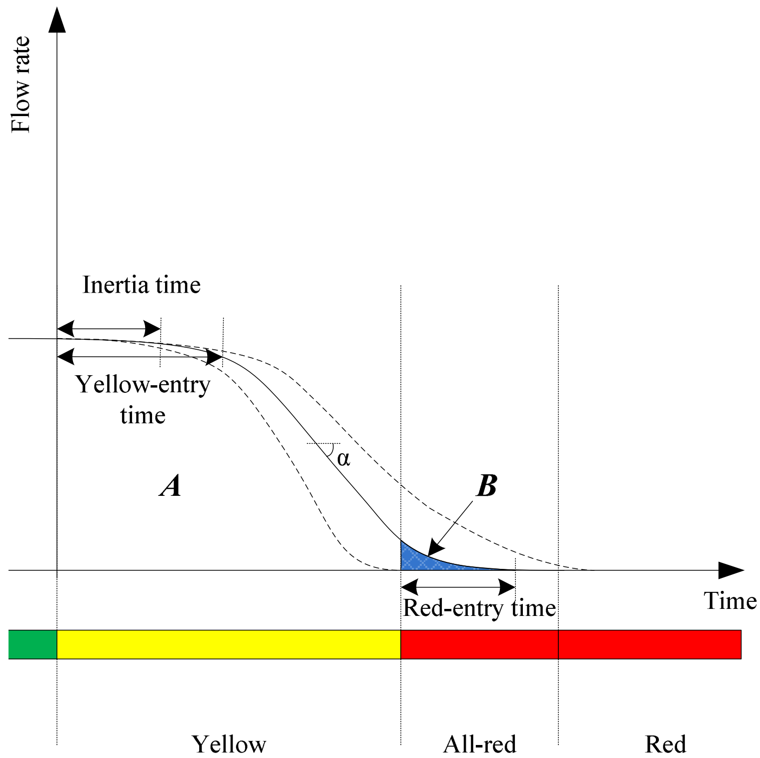


Figure 3.2 Conceptual yellow phase gradient

where $Y()$ is the number of crossing vehicles at the stop bar at the time point x seconds after the onset of yellow. It should be noted that N and Y functions describe the number of stopping and crossing vehicles at any specific time point after the onset of yellow, rather than the cumulative number of stopped and crossed vehicles until that point since the onset of yellow.

3.3.3 Green Phase Gradient

The green phase gradient part of the departure characteristic curve is shown in Figure 3.3. The flow rate starts to increase from zero a few seconds (or a fraction of one second) after the onset of green. The point on the curve, from which the flow rate rises dramatically, represents the start-up of most leading vehicles at the intersection. The time difference between the onset of green and this point approximates the start-up lost time of leading vehicles. The start-up lost time of the leading vehicle is associated with the drivers' perception-reaction time and vehicles' acceleration performance. It is possible that some vehicles enter the intersection before the onset of green if drivers start up by watching the traffic light of the crossing approach especially when countdown timers are installed *(Wu et al. 2009)*.

The rate of traffic flow increase, i.e. the slope β in Figure 3.3, represents the acceleration of vehicles following the leading vehicle. Its value is related to the acceleration performance of vehicles, as well as the alertness of following drivers in the queue. The earlier the flow rate reaches the discharging saturation flow, the less the traffic delay in queue discharging. HCM stated that the discharging saturation flow rate is mostly reached when the fourth to sixth vehicle crosses the stop bar *(TRB 2000)*, but it varies among different countries *(Li and Prevedouros 2002, Perez-Cartagena and Tarko 2004, Limanond et al. 2009, Hadiuzzaman and Rahman 2010)*. The area A in the figure, which is surrounded by the departure characteristic curve, the vertical axis, and the saturation flow rate line, represents the total delay to vehicles in the start-up process.

Similar to the yellow phase gradient, a function also could be developed to describe the green phase gradient curve. The traffic start-up process after the onset of green is different from the aforementioned traffic cutoff process after the onset of yellow. Drivers are expected to try their best to leave as early as possible. However, due to the reaction of following drivers, gaps are formed between start-up vehicles. If observing at the stop bar location, the gap changes over time until the saturation flow rate is reached. Assume the headway can be described as a function of time, $h(t, h_0)$, where h is the vehicle headway observed at the stop bar t seconds after the onset of green, h_0 is the time lapse of the leading vehicle crossing the stop bar (i.e. the initial headway). Then the number of vehicles crossing the stop bar can be described with the following equation.

$$N = \frac{1}{h(t, h_0)} \tag{3-4}$$

where N is the number of crossing vehicles (or flow rate) at t seconds after the onset of green. Further study is needed to building the relationship between the headway and the time into green. Then, the flow rate curve, as the reciprocal of the headway, can be quantitatively modeled.

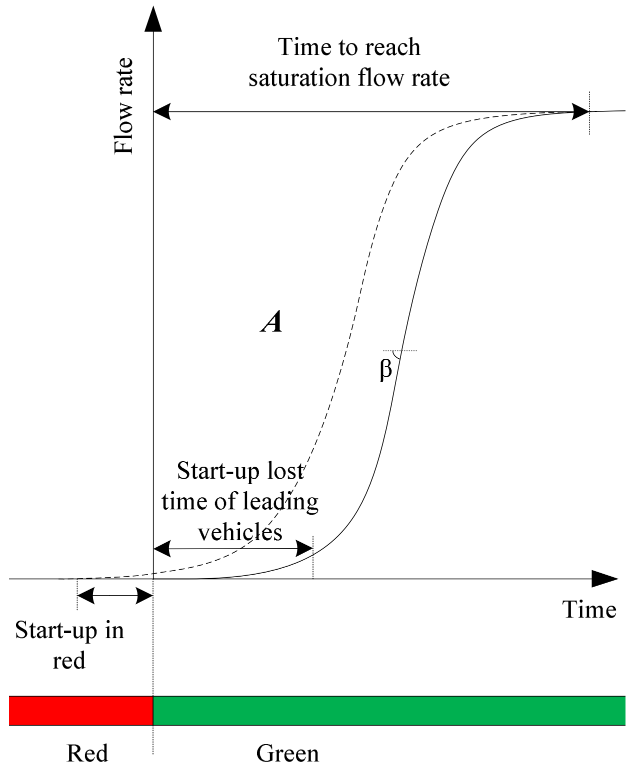


Figure 3.3 Conceptual green phase gradient

3.4 Case Study

A case study was performed to apply the proposed phase gradient method to evaluate the influence of red-light cameras on driver behavior at signalized intersections. Five comparable intersections in Knoxville, TN were selected for the case study. Among these five intersections, two were equipped with red-light cameras while the other three were not.

3.4.1 Site Characterization

Since these red-light cameras were already in place at the time the study was performed, before-after studies could not be conducted at the same site. Instead, a comparative study was performed between intersections with and without red-light cameras. Five comparable intersections in Knox County were selected. The locations of these sites on Google map are shown in Figure 3.4. Table 3.1 lists some key geometric and traffic metrics of these five intersections.

3.4.2 Data Collection

One video camera was installed at the study approach of each intersection at a vantage point with a reasonable view angle of the signal light display and vehicles crossing the stop bar. To avoid conspicuity so as not to affect the subjects we observed, the locations of cameras were typically away from travel lanes. Traffic stream and signal phase data were videotaped in a frequency of 30 frames per second. After the traffic was videotaped, the video was digitized and time code information was added onto each individual frame. Each location was videotaped for two hours on average.

As previously discussed, the proposed method relies on simple vehicle counting data at the stop bar location. As videotaping data was readily usable to the authors, to demonstrate the proposed method, only traffic counting data (i.e. the time each vehicle crossing the stop bar) were extracted from these videos for this case study.

3.4.3 Results

The time each vehicle crossing the stop bar was extracted from the video frame by frame in the precision of 1/30 of a second. Traffic signal timing of each intersection was obtained as well. Subsequently, the number of vehicles crossing the stop bar in each lane was complied in the increment of 0.5 second. Each lane at each intersection was processed separately. However, since there is limited number of data points at each intersection, the generated phase gradient curve shows significant random variations. To construct a more reliable curve, data points were aggregated for all red-light camera equipped sites and non-camera sites. To study the yellow phase gradient, 10 seconds of traffic flow data, centering on the onset of yellow, were examined during each signal cycle, since almost no vehicle crossed the stop bar after this period (right turn on red was excluded); for green phase gradient, 5 seconds of traffic data after the onset of green were examined for each cycle. No start-up vehicle before that was noticed.

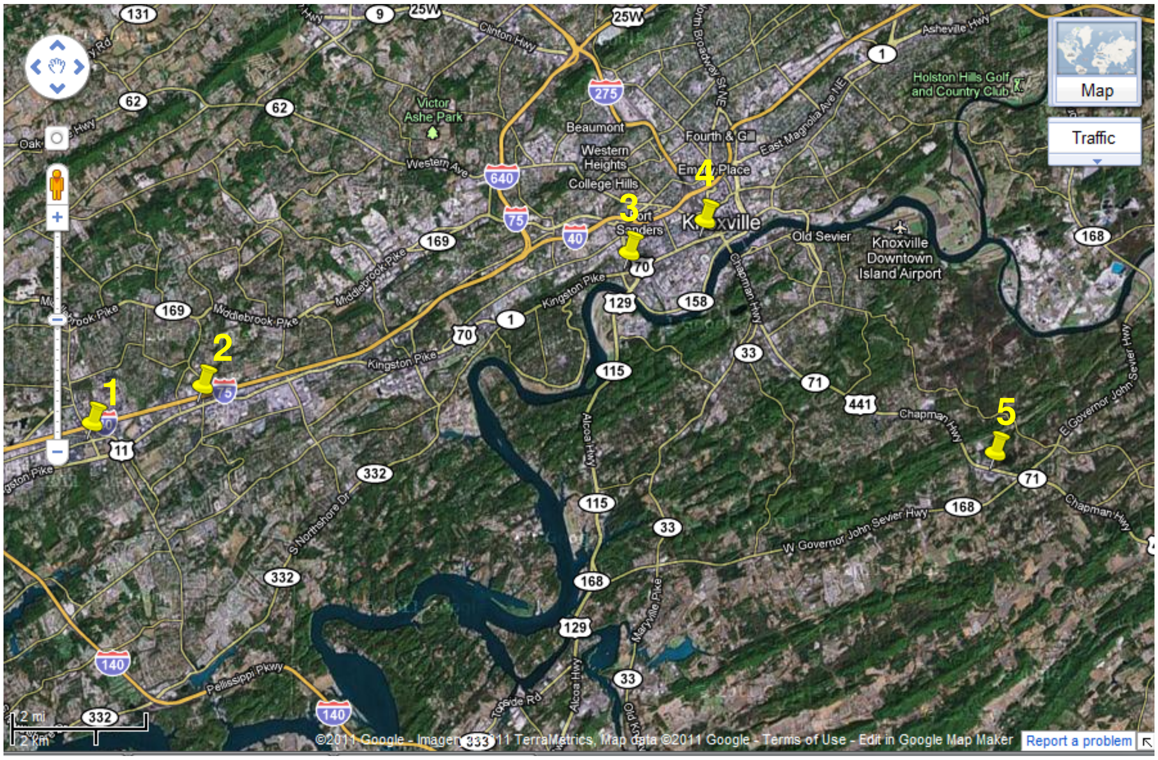


Figure 3.4 Study intersection site locations in Knox County, TN

Table 3.1 Geometric and traffic parameters of study intersections

ID	Intersection name	Red-light camera	# of lanes	AADT	Speed limit (mph)	Cycle length (sec)
1	Cedar Bluff at North Peters	Yes	3	15,360	45	110
2	Gallaher View at Kingston Pike	No	4	19,161	40	120
3	Kingston Pike at Alcoa	No	4	23,998	45	140
4	Cumberland[1] at 11th Street	No	3	16,249	35	150
5	Chapman Highway at Greene Lane	Yes	4	27,566	50	130

[1] Cumberland is an extension of Kingston Pike

All intersections have 4 seconds long yellow phase

Yellow phase gradient

The yellow phase gradient curves fitting the 10-second traffic flow rate data in the interval of 0.5 second are shown in Figure 3.5 and 3.6, for non-camera and red-light camera equipped intersections respectively.

Since the total aggregated traffic volume is different between the two groups of intersections, the traffic flow rate data are normalized in order to show a fair comparison. As seen in Figure 5 and 6, these two yellow phase gradient curves show a similar trend in general after the onset of yellow. Since the 0.5-second data segmentation is not fine enough due to limited available data points, it is unable to accurately obtain the inertia time as well drivers' perception-reaction time from the curves. The yellow-entry time is roughly between 3.0 and 3.5 seconds whether a red-light camera is present or not.

At non-camera intersections, the decrease of traffic flow rate is quite smooth after the onset of yellow, see Figure 3.5. Besides, there are a few red-light runners although the occurrence rate is quite low. The red-entry time is about 1.0 second. At red-light camera equipped intersections, some drivers would accelerate at two stages of yellow, see Figure 3.6. The first stage is about 1.0 to 1.5 seconds into yellow when drivers have perceived the onset of yellow and finished the acceleration of their vehicle. The time difference into yellow here could be defined as the acceleration time, from which driver's perception-reaction time can be also derived if vehicle's acceleration performance is assumed. The second stage of acceleration is about 1.0 second later, i.e. 2.0 to 2.5 seconds into yellow,

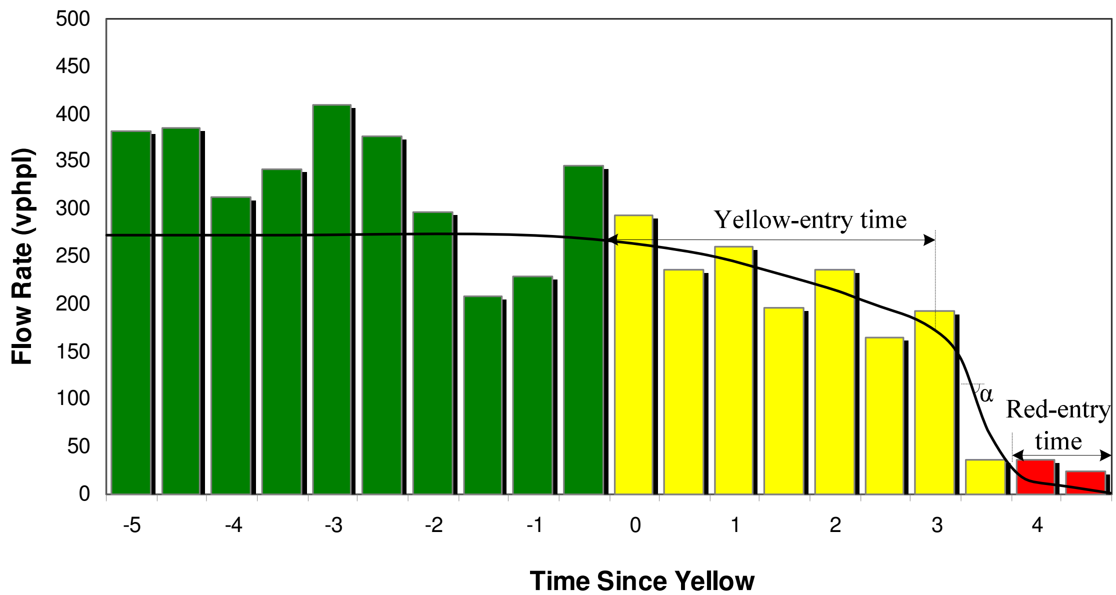


Figure 3.5 The yellow phase gradient curve for intersections without red-light cameras

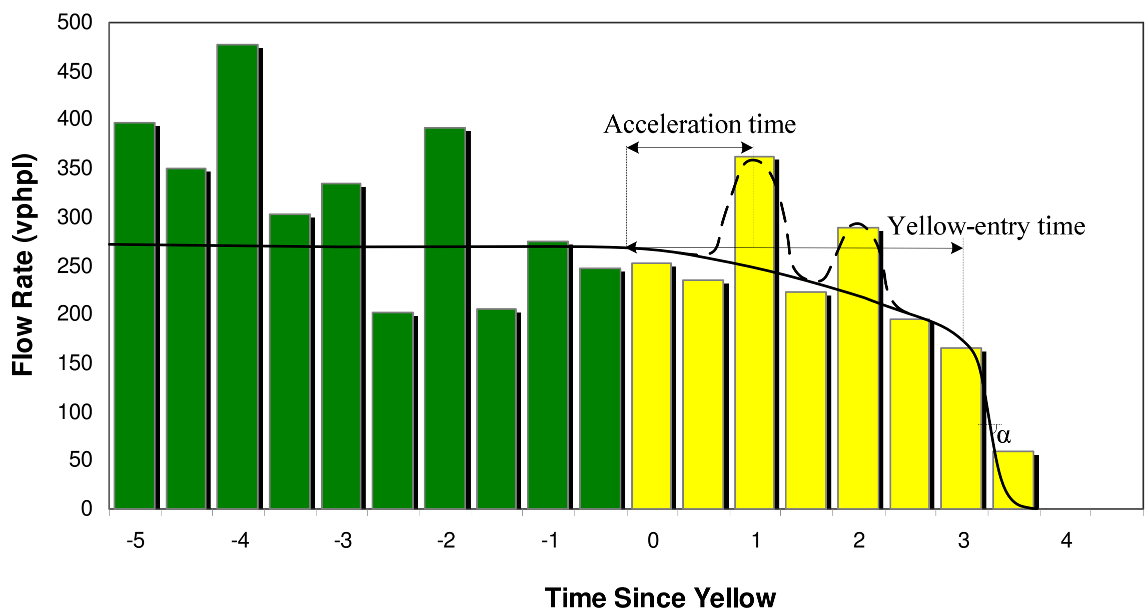


Figure 3.6 The yellow phase gradient curve for intersections with red-light cameras

when another surge, of less extent, of flow increase is shown. Drivers more familiar with the intersection and the signal setting know that the red is imminent and had to accelerate to beat the red light and avoid being caught by the red-light camera. The second and smaller surge may or may not occur depending on the familiarity of drivers in the study population. No red-light running, excluding right turn on red, was observed at red-light camera equipped intersections during the study period, which could be attributed to the deployment of red-light cameras.

An interesting point that would help understanding driver's reaction to red-light cameras is to examine the rate of traffic flow decline towards the end of yellow and the beginning of all-red. To compare driver's behavior in two different situations, the two phase gradient curves are drawn in the same chart, see Figure 3.7. At the earlier portion of the yellow phase, the rate of decline is almost the same for intersections with and without red-light cameras. Also, the two curves reach the turning point almost at the same time, i.e. 3.0 to 3.5 seconds into yellow. Toward the end of yellow, the decline rate for the case with red-light cameras is more pronounced than the other one. It appears that the traffic flow slows down more quickly at red-light camera equipped intersections and is terminated shortly before the onset of red. Apparently, drivers try hard to avoid running red when red-light cameras are installed.

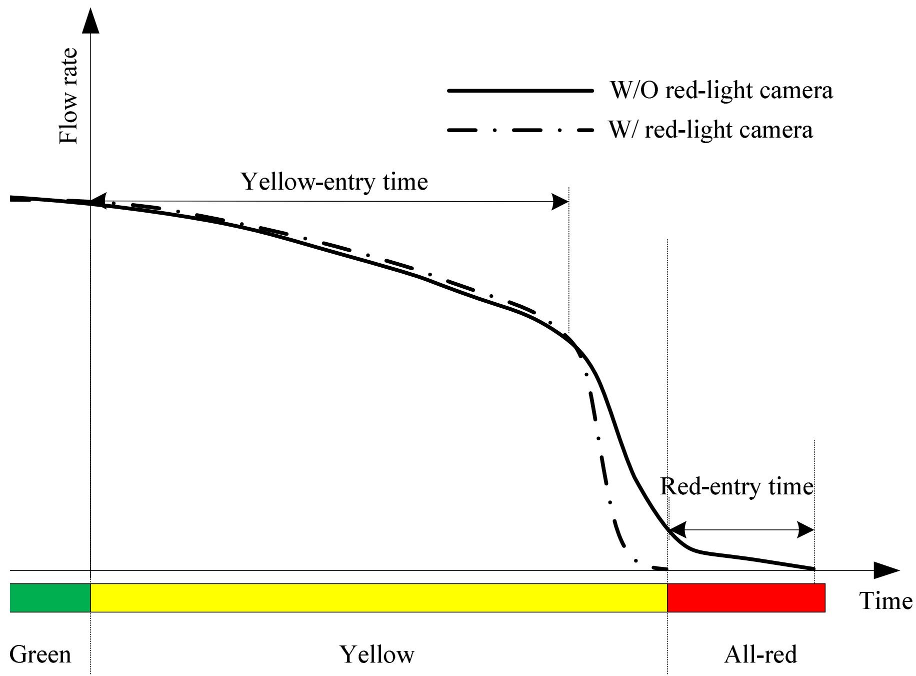


Figure 3.7 The comparison of phase gradient curves between two situations

Green phase gradient

Green phase gradient curves for intersections with and without red-light cameras, are shown in Figure 3.8 and 3.9 respectively. The curves also are generated with normalized traffic flow data.

In both situations, no early start-up before the onset of green was noticed. The start-up lost time of leading vehicles is between 0 and 0.5 second whether red-light cameras are present or not. The discharging flow rate increases dramatically about 0.5 to 1.0 second into green for both cases. Apparently, the discharging saturation flow rate, as defined by HCM, is not reached yet, since HCM states that the discharge saturation flow rate typically reaches its maximum about 10 to 14 s into green, which corresponds to the front axle of the fourth to sixth vehicle in the queue (TRB 2000). The surge of discharging flow rate between 0.5 and 1.0 second into green in this study is likely because the majority of leading vehicles start up in this time range. However, the surge in Figure 9 is much more significant than in Figure 8. At red-light camera equipped intersections, most vehicles enter the intersection earlier than non-camera intersections. Based on field investigations, it could be explained by a few reasons.

1. It is possible that drivers at red-light camera equipped intersections feel safe to start-up earlier since the crossing street is also under the surveillance of red-light cameras. Vehicles on the crossing street are less likely to run a red-light or interfere with the start-up traffic. For intersections without red-light cameras,

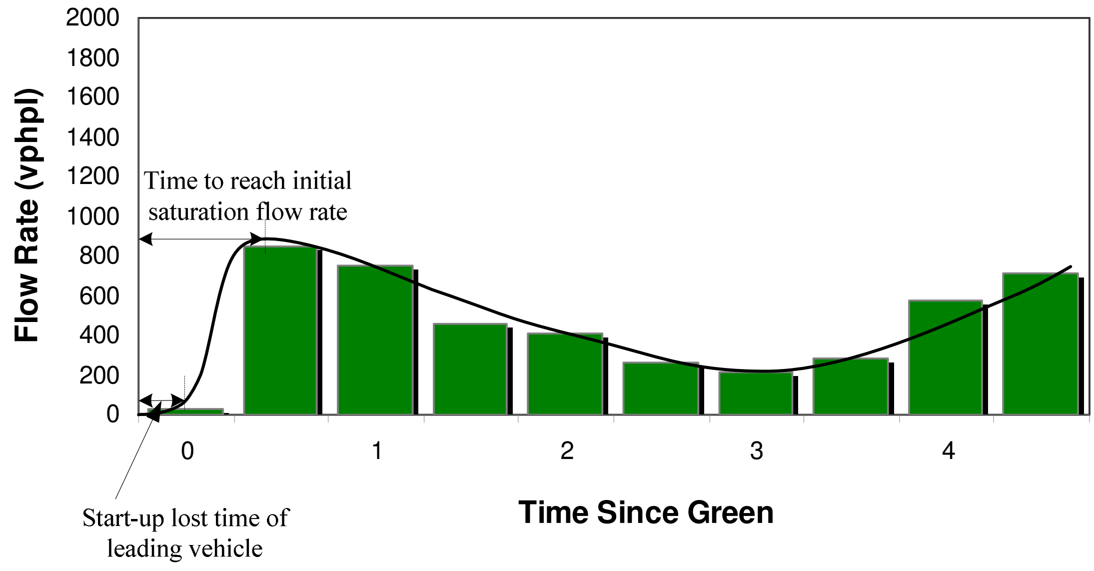


Figure 3.8 The green phase gradient for intersections without red-light cameras

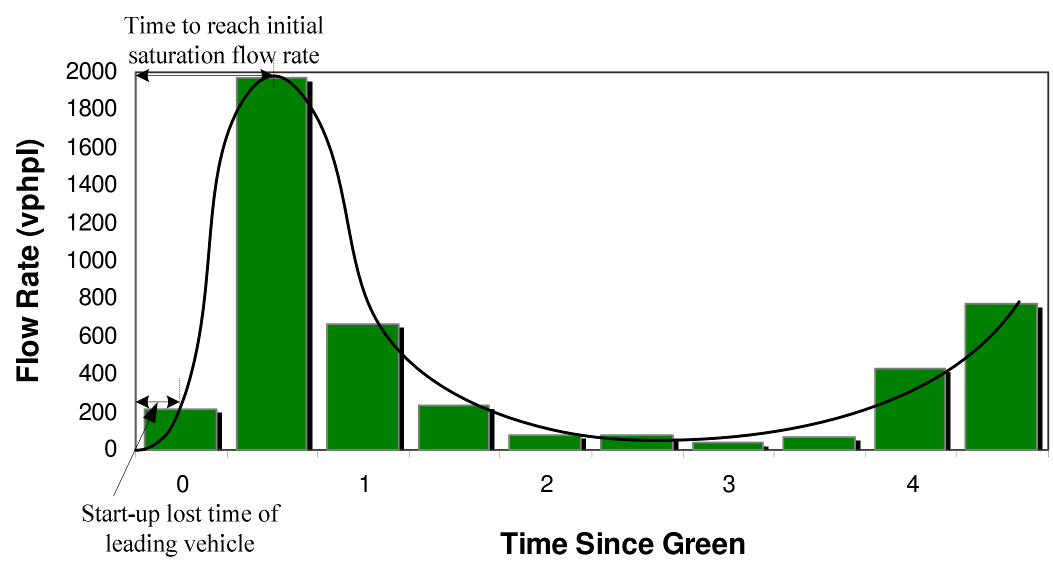


Figure 3.9 The green phase gradient for intersections with red-light cameras

2. vehicles on the crossing street may enter the intersection after the onset of red. If they are unable to clear the intersection before the end of all-red, it may lead to side collision with start-up vehicles. Therefore, start-up drivers are more cautious at these sites and most of them choose to enter the intersection slightly later, as shown in Figure 8.
3. The traffic demand at intersection 4 (non-camera intersection) is quite low, while its cycle length is fairly long. There are quite few vehicles accumulated in queue during the red phase. Many cycles/greens even start with no vehicle in queue. Traffic arrives at the intersection during the first 5 seconds of green randomly at the demand level. In comparison, the demand is high at intersection 1 (with red-light camera) while the cycle length is fairly short. There are always vehicles in queue and cross the stop bar in the first few seconds during the start-up process. Since data are aggregated with all intersections in each category and normalized based on the total traffic volume, the traffic surge in Figure 9 appears much more significant than in Figure 8.
4. Both intersections 2 and 4 (non-camera intersections) have a neighboring signal with very short distance (less than 200 ft) downstream and these signals are not coordinated. When the green signal commences at intersection 2 or 4, the signal downstream may still be red. Drivers are less willing to accelerate hard and leave quickly since they have to stop shortly at the next intersection. As a result, the leading vehicle loss times and the headways between vehicles at these two intersections are larger than other intersections. After the data aggregation, start-up vehicles spread out more with non-camera intersections, as shown in Figure 8.

After the surge, the discharging flow rate drops in the time range from 0.5 to 3.0 seconds into green. In this period, it is likely that following vehicles are reacting to the gap from the start-up vehicle in front. Then, following vehicles gradually fill up the gap by accelerating and shortening the headway and the discharging flow rate starts to increase again. In this 5-second observation window, it seems like the discharging saturation flow rate has not been reached yet since the increase of flow rate is expected to continue afterwards. Further study is still needed to obtain more information regarding the discharging of the queue.

Due to limitations of available data, the quantitative modeling of yellow and green phase gradients, as expressed by Eq. (3-3) and (3-4), cannot be validated in this paper. With more either real or simulated data available in the future, some further studies will be performed.

3.5 Potential Applications

The case study demonstrated an example of the application of the phase gradient method to an intersection study. In fact, the purpose of proposing the phase gradient method is to provide engineers and researchers a tool to learn the characteristics or identify the safety or operation issues at a specific intersection. Traffic engineers can generate a phase gradient curve for each local intersection based on traffic flow data and use it as a new data element in their traffic database. By looking into the phase gradient curve or comparing curves at different intersections, the experienced engineers will have an intuitive judgment on the safety and operation performance of a particular intersection. For instance, if the deceleration rate (angle α in Figure 3.2) of an intersection is abnormally high, the engineers may want to question the safety performance of the intersection and identify the reasons that cause the problem. The high deceleration rate implies drivers' abrupt stopping behavior and potential risk of rear-end collisions. It could be caused by an insufficient yellow length or other reasons. In such a way, engineers can use these curves from their database to identify potentially problematic intersections or help troubleshooting intersections that already show safety or efficiency issues.

The phase gradient method can provide information on various driver behavior and traffic characteristics, which are applicable to design, analysis, and research of signalized intersections. For instance, the capacity and traffic demand information can be used for the intersection design and planning; the yellow- and red-entry time can be used for the evaluation of intersection safety and the design of traffic timing; the start-up lost time and time to reach saturation flow rate can be used for the evaluation of intersection traffic operation; the driver perception-reaction time and deceleration can be used for driver behavior studies.

3.6 Conclusions

The paper proposes a phase gradient method to learn traffic and driver characteristics at signalized intersections during the transitions of traffic signal phases. The method relies on the vehicle counting data at the stop bar location, which simplifies data collection and processing. The method provides engineers and researchers a tool to characterize any specific intersection and identify the safety and operational issues at the intersection. A case study was conducted on the influence of red-light cameras on driving behavior to demonstrate the proposed method.

The phase gradient curve may vary among different locations and in different conditions. The use of the phase gradient method should be based on sufficient observations bearing a variety of traffic and environmental conditions at the particular intersection. Based on the case study, it is found that the presence of red-light cameras did influence the driving behavior after the onset of yellow. Drivers are more likely to accelerate to cross the stop bar during the early portion of the yellow phase and decelerate more quickly toward the end to avoid running a red. As a result, there is much less red-light running occurrence when red-light cameras are deployed.

Future studies should be performed to build the quantitative models of yellow and green phase gradients, which may better assist engineers to use the phase gradient method in their practice. The case study also showed limitations in terms of data sufficiency and variations in study site characteristics. A more controlled study, ideally before-after study, should be performed in the future to further investigate the effects of red-light cameras on driver behavior at signalized intersections. Also, more demonstration studies should be performed with the phase gradient method.

CHAPTER 4 HOW TO REALLY PROFIT FROM YOUR RED-LIGHT CAMERAS BY CLEVER SIGNAL SETTINGS – AND WHY YOU SHOULD NOT DO IT

(This chapter is a slightly revised version of a paper to be submitted by Qiang Yang, Lee D. Han, Christopher Cherry)

4.1 Introduction

Since invented in 1960s, automated enforcement red-light cameras (RLC) have been widely adopted by municipalities around the world as a measure to curb red-light running (RLR) at signalized intersections and reducing the cost of law enforcement. Technological improvements have made RLC much more effective in recent years, increasing their adoption in the past decade. Most RLCs are installed with dual, conflicting purposes, reduce RLR and maximize private sector (and public) revenue from RLR citations. Harmonizing these two purposes is challenging resulting in substantial backlash against RLC. Many studies were performed in different countries on the effects of RLC on driver behavior, intersection safety, and traffic capacity. These studies show mixed results on whether the installation of RLC benefits the safety at signalized intersections. In some cases, RLCs were shown to reduce severe crash rates and improve overall intersection safety. In other cases, RLCs have been installed at intersections that might not have the most severe safety challenges, but rather have higher incidence of RLR and thus more violations and revenue. RLCs are also implemented for the purpose of generating revenue for municipalities and profit for private-sector industries from RLR citations. Indeed, as municipal budgets are threatened, the temptation to identify RLC as a revenue generation source or substantial is increasing. Protests and lawsuits concerning the use of RLC have occurred around the nation and, in many instances, lawmakers have restricted their use.

Most RLC vendors provide the equipment and installation to municipalities for free. Both parties share the revenue of citations generated from RLC and a certain amount of citations have to be issued to recover the cost for equipments, installation, and maintenance. Camera providers have a clear profit motive, install RLC at intersections with the highest probability of violation. With shrinking municipal budgets, municipalities are looking for ways to augment their revenue. As a result, some municipalities have manipulated the setting of the traffic signal (e.g. shorten the duration of the yellow phase to trap more red-light runners) to generate more revenue at RLC equipped intersections. Engineering best practices allow for some flexibility in signal timing. While some of these measures are a violation of engineering codes others violate best practices. Perhaps more importantly the public may criticize or even file lawsuit against local municipalities, which harm the credibility and public image of related agencies and elected officials.

This paper provides a background of the safety and policy issues related to the use of RLC. The main focus of the paper is to identify potential engineering measures that may be employed to increase the number of RLR and associated revenue. The engineering measures are mainly related to the signal timing that are relatively inexpensive to implement. We close with reasons not to implement these strategies, focusing on safety and operational efficiency of signalized intersections.

4.2 Background

4.2.1 Effect of RLC on Intersection Safety

The primary motivation for installing RLC is safety through the consistent expectation of enforcement. As such, a number of studies have evaluated the effectiveness of RLC as an enforcement mechanism to reduce red-light violations and associated severe crashes. Several studies found a significant difference in crash rate and an improvement in overall safety attributable to RLC. Retting et al. (1999a) and Ruby and Hobeika (2003) investigated the RLC in Virginia and found a 36% and 69% reduction in RLR over the first three and six months of camera operation. The accident rates also showed a 40% reduction. Similar RLC positive effects on RLR and accident rates were observed in California (Fleck and Smith, 1999; Retting et al., 1999b), North Carolina (Cunningham, 2004), and Iowa (Fitzsimmons, 2009). Lum and Wong (2003a, b, c) found that the RLC installation at three intersections in Singapore reduced RLR by more than 40% while non-camera approaches did not experience such a reduction during the same period. Huang et al. (2006) modeled the accident risk at 15 signalized intersections in Singapore and found that RLCs were effective in reducing RLR and right-angle collisions. However, it had a mixed effect on rear-end collisions depending on speed of the trailing vehicle and the headway between vehicles. Persaud et al. (2005) reported a similar effect of RLC on right-angle and rear-end collisions in the US. Radalj (2001) investigated the same issue at 58 RLC and 447 non-RLC intersections in Australia and found that the installation of RLCs reduced fatalities by over 50% but increased rear-end crashes by 17%. The reduction in the total number of crashes was 3%.

Some other studies indicated no difference or even negative effects on safety after the installation of RLC. Burkey and Obeng (2004) analyzed reported accidents occurring near 303 intersections over a 57-month period. They found RLCs increased the accident rates by 40% while the overall time trend during the same period indicated that accidents at all intersections were becoming less frequent. The study reported a large increase in rear-end accidents due to RLCs. Regarding crash severity, RLCs were found to increase property damage only and possible injury crashes, but have insignificant effect on severe accidents. A study in Arizona (Washington and Shin, 2005) found that the total number of crashes was unchanged as a result of RLCs at 10 intersections in the City of Phoenix (14% reduction in angle crashes and 20% increase in rear-end crashes). Total crashes were slightly reduced by 11% in the City of Scottsdale. Garber et al. (2007) also observed an increase in rear-end crashes and a reduction in RLR crashes associated with RLCs. However, when the comprehensive costs for different types of crashes were monetized, RLCs were associated with a net increase in crash costs considering six jurisdictions in Virginia. Kent et al. (1995) investigated RLR data at three RLC intersections and concluded that there was no difference in RLR between camera and non-camera approaches (at the same intersection).

4.2.2 Policy Response to RLC

While a consensus has not yet been reached about whether RLC can benefit intersection safety by reducing RLR and crashes, many believe that transportation agencies and vendors install RLC for the purpose of maximizing revenue. Some RLC vendors provide the equipment and installation to municipalities for no cost, and share with local agencies the revenue from ticketing red-light runners with their camera systems. Political backlash has occurred in many cases because of this perception. A survey of news articles on *theNewspaper.com* highlighted many anti-camera referendums in cities of Texas, Washington, Missouri, California, and Illinois between 2010 and 2012. States like Massachusetts, South Dakota, Mississippi, Maine, Nevada, Virginia, Alabama, Kentucky, and cities, like Albuquerque, NM and San Jose, CA, even voted to reject the use of RLC since 2006.

In regions where RLCs were implemented, many lawsuits were filed challenging the use of RLC across the nation. Insurance Institute for Highway Safety (IIHS 2010) reported lawsuits resulted from both citizens and RLC vendors against municipalities. In 2009, a number of cities were sued for the installation of RLC in Florida, where automated enforcement was still illegal in the state at the time of installation. To enable the usage of RLC, these cities created their own type of ordinance, which was not allowed by the Florida Constitution and became the primary argument in the lawsuit against the cities (theNewspaper.com 2009). Similar cases questioning RLC legality were reported in Minneapolis, MN; Hazelwood, MO; Lafayette, LA; Miami-Dade County, FL; Santa Ana and South San Francisco, CA; and Clive, IA since 2007. As a result, some of the illegally collected fines had to be refunded to drivers.

Since 2005, many cities shut down their RLC system after a few months or years of operation mainly because of 1) public pressure and legality issues, 2) failure to generate profits, and 3) failure to improve safety. A report shows that Atlanta is likely to join Los Angeles, CA and Houston, TX as major cities that have recently shut down photo ticketing programs (theNewspaper.com 2012). Many other cities in Georgia, California, Colorado, Washington, Missouri, North Carolina, and Texas have similar experiences.

Since some municipalities dropped RLC program in the middle of contract with camera vendors, which led to loss of revenue for the vendor, there are also cases of legal dispute between municipalities and RLC vendors. A new state law in Tennessee took effect in 2011 that prohibited the use of cameras to issue tickets for right-turn-on-red violations (Tracy et al. 2011). As a result, the RLR citations decreased by three quarters which led to loss of revenue to both cities and camera vendors. Two RLC vendors filed lawsuits against the city of Knoxville and the town of Farragut respectively after the new state restriction was issued in 2011 (Brewer and Jacobs 2011). A RLC company also filed a lawsuit against the city of Houston for breach of contract after a 2010 referendum shutting down RLC. Recently the company has reportedly agreed to drop the lawsuit against the city and take down all cameras providing the city pays a settlement of at least $4.8 million (Moran 2012). Similar cases in Manatee, FL; San Bernardino, CA; and Baytown, TX were reported as well (theNewspaper.com).

To generate revenue from RLC, some local agencies are accused of taking measures to increase the probability of RLR through signal timing or other engineering strategies. In 2008, six cities, including Chattanooga and Nashville, TN; Dallas and Lubbock, TX; Springfield, MO; and Union City, CA, were caught shortening the yellow light duration at RLC equipped intersections presumably to increase RLR (blog.motorists.org 2008). The city of Billings in Montana planned to shorten yellow times before the state legislature banned RLC in 2009. Winnipeg was reported to trap drivers with shortened yellow timing. The city of Seattle, WA also shortened yellow lights when it expanded its RLC program in 2008 (theNewspaper.com). As a result, the city of Chattanooga in Tennessee (Lazenby 2008) and the city of San Carlos in California (theNewspaper.com 2009) refunded fines to motorists who received tickets for running red-lights at an improperly timed intersection. The state of Georgia issued new law mandating longer intersection yellows at intersections where RLCs were to be installed. The state of Tennessee proposed legislation to ban cities with RLC from shortening yellow signal duration (theNewspaper.com 2008; 2009). It was also reported that in the state of California speed limits were lowered on purpose so that the yellow duration could be reduced without violating engineering standards but trapping more red-light runners. Attempts were also made to reduce the number of warning signs at photo enforcement equipped intersections (highwayrobbery.net 2011). Whether or not the municipalities' motives were explicitly trying to increase revenue, it is clear that there is a strong perception that engineering strategies are used to *increase* RLR. Table 4.1 summarizes

Table 4.1 Summary of policy responses associated with the use of RLC

States of municipalities	Policy responses
Texas, Washington, Missouri, California, and Illinois	Anti-camera referendum
States: Massachusetts, South Dakota, Mississippi, Maine, Nevada, Virginia, Alabama, and Kentucky Cities: Albuquerque, NM and San Jose, CA	Vote to reject the use of RLC
Temple Terrance, FL; Minneapolis, MN; Hazelwood, MO; Lafayette, LA; Miami-Dade County, FL; Santa Ana and South San Francisco, CA; and Clive, IA	Lawsuit against the illegal installation of RLC
States: Georgia, California, Colorado, Washington, Missouri, North Carolina, and Texas Cities: Atlanta, GA; Los Angeles, CA; and Houston, TX	RLC system shut-down
Knoxville and Farragut, TN; Houston and Baytown, TX; Manatee, FL; and San Bernardino, CA	Legal dispute between RLC vendors and municipalities
Chattanooga and Nashville, TN; Dallas and Lubbock, TX; Springfield, MO; Union City, CA; Billings, MT; Winnipeg, Canada; and Seattle, WA	Accused of cheating with the yellow duration for trapping red-light runners
Chattanooga, TN and San Carlos, CA	Refund ticketing fines due to improper signal timing
Georgia	Law against shortening the yellow duration at RLC intersections
California	Accused of cheating with the speed limit for trapping red-light runners

cases of policy response associated with the use of RLC in different states or municipalities.

There are abundant examples of municipalities and vendors adjusting signal cycles to presumably increase revenue. The length of the yellow duration is the primary parameter being adjusted to boost revenue. This paper studies different possible engineering measures, mainly related to the signal timing, which may influence the frequency RLR and thus could be potentially used by RLC providers and municipalities for increasing revenue from the camera systems. These strategies are also revealed so to policy makers and citizens to increase transparency of the divergent motivations of RLC vendors, municipalities, and safety advocates.

4.3 Measures

Based on previous studies, this section introduces engineering measures, mainly related to signal timing, which can potentially increase the frequency of RLR at signalized intersections. The effectiveness of these measures is analyzed either quantitatively or qualitatively. Potential issues related to safety and efficiency of the traffic system and public image of relevant agencies due to implementing such measures are also discussed.

4.3.1 Shorten Yellow Duration and/or Lengthen All-Red

The ITE recommended yellow duration is calculated with Eq. (4-1) (ITE 1989).

$$Y = t_r + \frac{V}{2a + 2gG} \tag{4-1}$$

where Y is recommended yellow duration, sec.; t_r is driver perception-reaction time, use 1.0 s; V is vehicle approaching speed which is typically the 85th percentile design speed, ft/s; a is deceleration rate, use 10 ft/s^2; g is gravitational acceleration, use 32.2 ft/s^2; G is approach grade, ft/ft.

The yellow duration equation is based on the rationale that drivers at the design speed can cross the stop line within yellow if the distance to the stop line at the onset of yellow is shorter than a comfortable stopping distance. This ensures that drivers at the design speed have one reasonable choice from either comfortably stopping or crossing without running a red-light depending upon their distances to the stop line. In other words, if the yellow duration is shorter than the recommended value, some drivers at a certain distance from the stop line face a dilemma situation and have to run red-light because they are unable to comfortably stop before the stop line. The form of such a dilemma zone, which affects the RLR behavior, is discussed in detail in a later section.

Bonneson and Zimmerman (2004b) conducted a before-and-after study on the effects of increasing the yellow interval on the frequency of RLR, and found that an increase of the yellow duration by 0.5 to 1.5s decreased the red-light violations by at least 50%.

Increasing a yellow interval that is shorter than the ITE recommended value yielded the greatest return. Van der Horst and Wilmink (1986) also reported that a 1s increase in yellow (i.e. from 3 to 4s in urban areas and 4 to 5s in rural areas) decreased red-light violations by 50%. Retting et al. (2008) conducted a similar study of increasing yellow intervals from 3 to 4.1s and from 4 to 4.9s at two intersections respectively, and observed a 36% reduction in red-light violations. Bonneson et al. (2002) performed a study to the opposite direction. They studied the effects of decreasing the yellow interval by 1s on RLR and reported a 110% increase in the violation frequency. At the same time, they observed a 53% reduction in the violation frequency when the yellow interval was increased by 1s. These studies imply that shortening the yellow interval duration can potentially increase the occurrence of red-light violations and the revenue generated from RLC, especially when the yellow interval is shortened to a value below the ITE recommended value.

One of the criticisms of strategies to increase or decrease the yellow time is that drivers may adapt to the change in yellow duration after the measure is implemented. The long-term effect is smaller than immediately after the measure is implemented. Such an effect is called "habituation". However, studies showed that the habituation effect did not undo the effect of changing the yellow duration (Bonneson and Zimmerman, 2004b; Retting and Greene, 1997; Gårder, 2004). Figure 1 shows the habituation effect observed by Bonneson and Zimmerman (2004b).

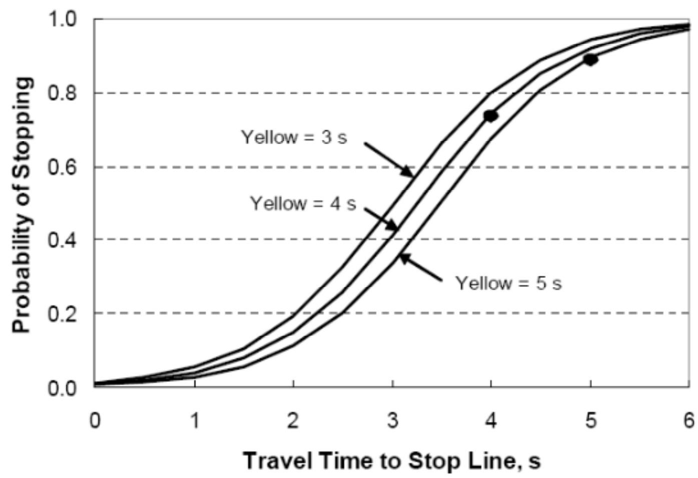


Figure 4.1 Probability of stopping as a function of travel time and yellow duration (Bonneson and Zimmerman, 2004b)

Although ITE recommended yellow duration values are not mandatory (i.e. code or standard), decreasing the yellow duration to an unreasonable low value for the purpose of trapping more red-light runners may impose risks, especially rear-end and right-angle collisions, on drivers. As a result, related traffic departments have to take the responsibility for improper settings of traffic signal timing. Retting et al. (2002) estimated the potential crash effects of modifying the duration of traffic signal change intervals to ITE recommended values. They found an 8% reduction in reportable crashes, a 12% reduction in injury crashes, and a 37% reduction in pedestrian and bicycle crashes at experimental sites relative to controls for a 3-year period following implementation of signal timing changes. Based on a meta-analysis of all worldwide studies in 1997, Gårder (2004) found that longer evacuation times (all-red and/or yellow times) on average reduced crashes by 55%. Bonneson and Zimmerman (2004a) also reported that crash frequency decreased with increasing yellow duration.

Potentially, adding an adequate clearance interval may alleviate the safety concern due to the decrease of the yellow interval. To ensure the safety of intersection operation, the total length of change interval (i.e. yellow plus all-red) should be guaranteed. Therefore, lengthening the clearance interval will not undo the effects of trapping more red-light runners after shortening the yellow interval, and also not cause burden on related agencies for increasing crash rates. Zador et al. (1985) conducted a study at 91 signalized intersections and found that intersections with more adequate clearance intervals had substantially fewer rear-end and right-angle crashes, but yellow running occurrences were unaffected by the length of clearance intervals. Seyfried (2004) also reported that a clearance interval and its length did not influence the drivers' decisions in running the red light. Awadallah (2009) proposed a theoretical approach for reducing RLR and found that clearance intervals were not necessarily a cure for RLR, especially when drivers came to expect an additional safety increment and try to misuse it. Stein (1986) reported that intersections that had inadequate clearance intervals had higher crash rates. McGee (2003) summarized several studies on the effect of adding a clearance interval on intersection crashes and concluded that all these studies showed positive safety benefits in terms of reducing either crash rates or injuries after implementing clearance intervals.

However, some studies had different results regarding the safety effects of installing or increasing the length of clearance intervals. Bonneson and Zimmerman (2004a) stated that increasing the clearance interval was likely to reduce right-angle crashes at the initial onset of the red phase (i.e. the first few seconds of red). However, these initial onset of red crashes are relatively infrequent so increasing the clearance interval might not significantly reduce the total number of right-angle crashes. Souleyrette et al. (2004) evaluated the long-term safety effects of increasing clearance intervals employing a before-and-after analysis of 11 years of data and found short-term reductions in crash rates (one year after the implementation), but long-term reductions were not observed. A study in Indiana also reached the same conclusion (Roper et al., 1990). It is likely that drivers may get used to the change of clearance interval after a certain period of time that could diminish the benefits of the clearance interval installation or an increase of the duration.

4.3.2 Vary Yellow Duration

Many previous studies have indicated that drivers are subjected to the "habituation" effect to the change of signal timing. Drivers who are familiar with the intersection (e.g. the duration of the yellow interval) are more likely to best use the yellow interval and avoid running a red-light. When the yellow interval is changed, drivers need a certain period of time to learn the new setting of the yellow interval and adjust their behavior. Therefore, if the length of the yellow interval is set to change frequently, such as every month, the violation to drivers' expectation immediately after each time the new setting is implemented may lead to drivers' indecision at the intersection and cause more RLR. The effect is expected to be especially significant when the yellow interval is decreased. The advantage of such a strategy is that the length of yellow intervals does not need to go below the ITE recommended value, and thus may not necessarily impose legal burdens on related agencies.

There is no report found on any study or previous practice of varying yellow duration strategy. However, it is expected that the indecision caused by the varying yellow duration may lead to aggressive crossing and abrupt stopping behaviors at the intersection, which are associated with occurrences of rear-end and right-angle crashes. Therefore, safety is still a concern when implementing such a strategy to profit RLC. So long as the all-red phase compensates for the reduced yellow phase (e.g. the yellow phase reduces by 1s and the all-red phase increases by 1s), there would presumably be little safety impact from the increased RLR.

4.3.3 Shorten Cycle Length

It is easy to understand that shortening the cycle length can increase the red-light violation frequency since shorter cycle lengths increase the hourly frequency of signal changes and thus the exposure of drivers to potential RLR situations. Previous research indicated that the frequency of RLR is largely affected by the frequency with which the yellow is presented (Van der Horst and Wilmink, 1986). Many other researchers also recognized the correlation between the frequency of signal changes and RLR (Porter and England, 2000; Baguley, 1988). Based on field studies, Bonneson et al. (2002) found a 29% increase in red-light violations when the cycle length was decreased from 90 to 70s, and an 18% reduction in red-light violations when the cycle length was increased from 90 to 110s.

In addition to the increase of the frequency of signal transition, a shorter cycle length affects RLR because it increases the probability of a transition during the end of a platoon.

A very long cycle can fully discharge the vehicle platoon before the next transition and thus reduce the probability of catching vehicles during the transition. Vehicles arriving in a platoon (or in a close car-following condition) are more likely to run a red-light, which will be further discussed in a later section.

The efficiency of signalized intersection operation is associated with the setting of cycle length. Relatively short cycle lengths may be desirable if the traffic demand is low. However, if the traffic arrival rate is higher than the discharging capacity of the short signal cycle, some drivers need to wait in queue for more than one cycle. The excessive delay may also encourage drivers to run a red-light. Therefore, although shortening the cycle length increases the occurrence of RLR, it may not always be desirable from the efficient operation point of view.

4.3.4 Lengthen Cycle Length or Increase Red/Green Ratio

Many studies have identified excessive delay as one of the major contributing factors to red-light violations (Bonneson et al., 2001; Elnashar, 2008; Gårder, 2004). A survey conducted by FHWA indicated that 66% of Texas drivers believe RLR is due to drivers who are in a hurry (Slemmons, 1998). The excessive delay due to stopping at signalized intersections may cause aggressive crossing behavior and red-light violations. Therefore, any measure that increases driver's actual or perception of delay is likely to encourage RLR behaviors.

Excessively long cycle length (or more specifically red-time) will increase delay to drivers. Drivers approaching a signalized intersection with a long signal cycle, especially those who are familiar with the intersection signal setting, are more likely to cross the intersection to avoid the delay. Seyfried (2004) reported the concept of "optimum" cycle length (C_o) by which delay is minimized. Cycle lengths outside of the optimum range (between ¾ and 1-½ of C_o) result in excessive delay and may induce drivers to respond aggressively during signal transitions. Although a long signal cycle may increase the probability of red-light violations in one cycle, it decreases drivers' exposure to signal transitions in total. Therefore, there is a balance between long and short signal cycles. Future studies are recommended to learn the relationship between the cycle length and the total number of RLR occurrences. In addition, the long cycle length is mostly undesirable from the operation efficiency point of view.

An alternative to increase drivers' waiting time in queue without increasing the cycle length is increasing the red-to-green (R/G) ratio in a cycle. A higher R/G ratio means drivers have to wait a longer time for the green phase to enter the intersection. Also, the capacity of the intersection approach is lowered with the increase of R/G ratio. Some drivers may need to wait for more than one cycle before they can enter if the capacity is exceeded. Gårder (2004) found that the intersection with short green times and long red times had a high RLR frequency. Washbum and Courage (2004) also identified green-to-cycle ratio as one of the factors that were correlated to the level of RLR.

4.3.5 Increase Speed Limit

Drivers at high speed are more likely to be caught in the dilemma zone when approaching a signalized intersection and have a higher probability of running a red-light. When drivers within a distance from the intersection that is shorter than the distance to comfortably stop and longer than the distance to cross the stop line before the onset of red at a constant speed, they are assumed to be caught in the dilemma zone. Assuming a level ground, Eq. (4-2) and (4-3) show the calculation of the necessary stopping distance and the travel distance at a constant speed during yellow.

$$S_1 = t_r V + \frac{V^2}{2a} \tag{4-2}$$

$$S_2 = V * Y \tag{4-3}$$

where S_1 is the stopping distance; S_2 is the travel distance during yellow; Y is the yellow length. Assuming a 4s yellow interval and using other parameter values in Eq. (4-1) as recommended by ITE, the dilemma zone in different driving speeds is shown in Figure 4.2.

It is seen that when the driver's speed is above a threshold value (about 41 mph in the Figure 4.1 case), there will be a dilemma zone. With the increase of speed, the range of dilemma zone increases. Therefore, increasing drivers' speed will increase the probability of catching them in a dilemma zone and thus the probability of red-light running. Many studies have indicated that controlling for travel time to the intersection, high-speed drivers are less likely to stop than low-speed drivers (Allsop et al., 1991; Panagiotis, 2007; Verghese and Alex, 2010). Figure 4.3 shows the relationship between the probability of stopping and the time to stop line in different speeds based on the study of Bonneson et al.

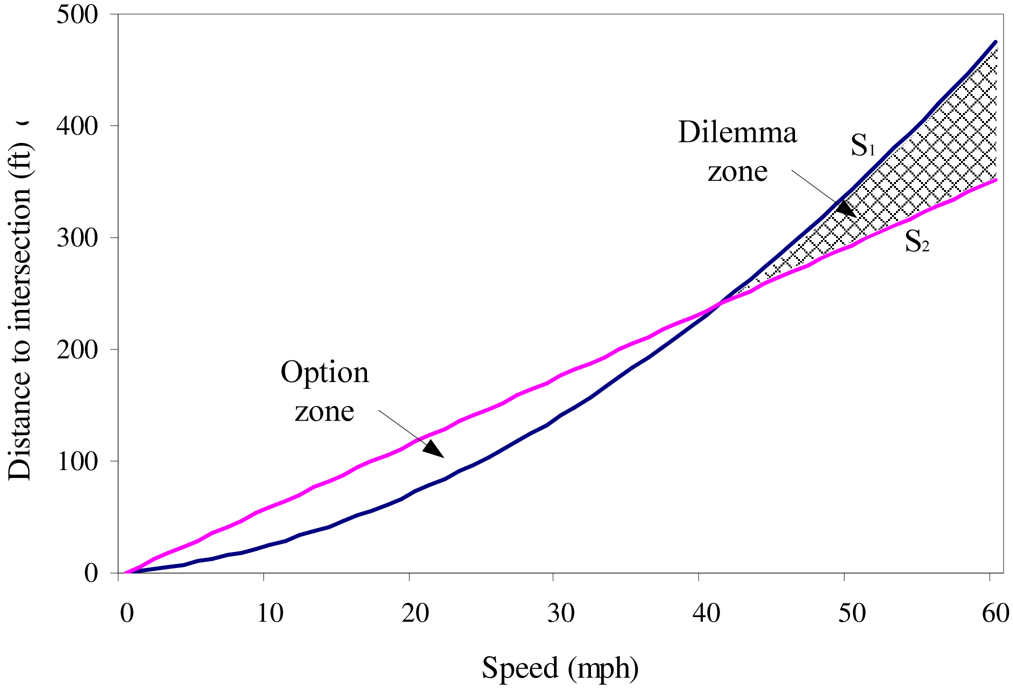


Figure 4.2 Intersection dilemma zone at different speeds

(1994). Bonneson et al. (2002) reported a 45% increase in the red-light violation frequency when the running speed was increased by 10 mph and a 33% decrease in the violation frequency when the running speed was decreased by 10 mph.

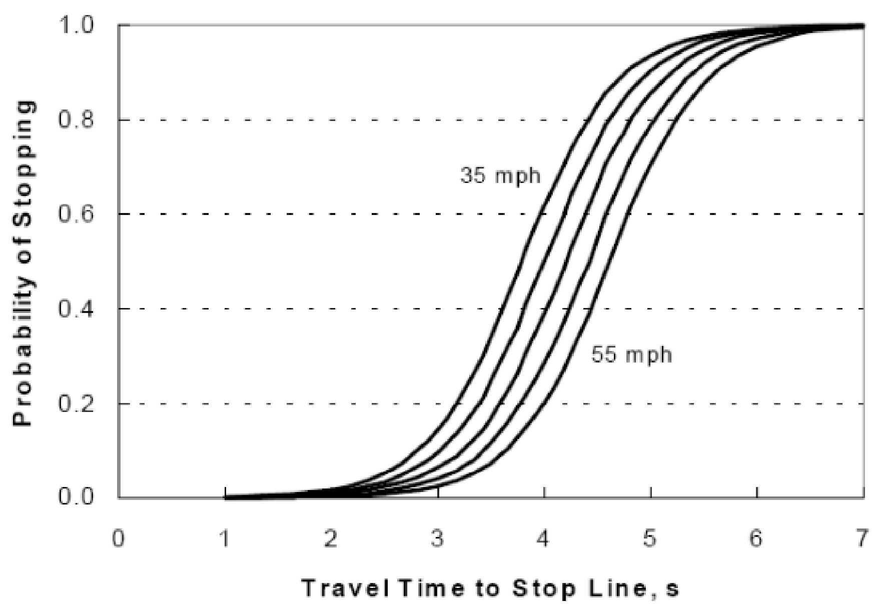


Figure 4.3 Probability of stopping as a function of travel time to stop line in different speeds (Bonneson et al., 2001)

To increase revenue from RLC, raising the speed limit may be the most straightforward and effective way of increasing the driving speed. NCHRP (2003) indicated that operating speed (85th percentile speed) typically exceeds posted speed limit. Even the 50th percentile operating speed either was found to either be near or exceed the posted speed limit. The difference between the operating speed and the posted speed limit varies in different driving environments. As posted speed limit increases, the operating speed increases.

Another important factor contributing to drivers' speeding behavior is roadway geometric conditions, e.g. number of lanes, curvature, etc. Garber and Gadiraju (1989) indicated that drivers tended to operate at increasing speeds as roadway geometric characteristics improve regardless of the posted speed limit. Therefore, intersections with good roadway geometric conditions may have a higher RLR frequency. Installing RLC at these intersections may increase the revenue generation. In addition, the posted speed limit and roadway geometric are interrelated in affecting the operating speed and its variation. Garber and Gadiraju (1989) found that when the posted speed limit is approximately consistent with the design speed (or roadway geometric conditions), e.g. the posted speed limit is between 6 and 12 mph lower than the design speed, speed variance is minimized. Speed variance increases with increasing difference between the two. Therefore, raising the speed limit may also increase the speed variance and the number of vehicles operating in extreme speed. It increases the number of vehicles in the more extreme area of the dilemma zone and thus the probability of RLR.

However, drivers' indecision in dilemma zone due to the increase of the posted speed limit may lead to aggressive crossing or abrupt stopping behavior. The increased speed limit may also lead to drivers' speeding behavior which is incompatible with the actual driving conditions, like road geometry, ambient environments, etc. This will potentially increase crash frequency. Bonneson and Zimmerman (2004a) reported a significant increase in crash frequency with the increase of the operating speed. In addition, higher driving speed is, in general, likely to result in more severe injuries in crash. Therefore, increasing speed may generate more revenue from RLC, but is also likely diminish safety.

4.3.6 Signal Coordination Offset

In a coordinated traffic signal system, ideally most vehicles can pass every traffic signal within the green interval and do not need to face or stop at a red light. In such a case, drivers' exposure to RLR is very low. Shinar et al. (2004) compared red-light violations at synchronized and nonsynchronized intersections by observing 3600 cycles of traffic signals at 12 intersections. They found that synchronized intersections had less RLR by (a) providing fewer RLR opportunities, and (b) having a lower probability of RLR relative to the number of opportunities. The odds of RLR in the synchronized system were nearly 1/7 of that in the nonsynchronized system. Synchronized intersections smooth the traffic flow and thus reduce drivers' frustration from having to stop repeatedly. However, as congestion increased, the synchronization became less effective.

If an offset is placed into the coordinated traffic signal system intentionally, as shown in Figure 4.4, more red-light runners are expected. The basic idea of the offset coordination signal system is forcing the leading vehicle in the platoon to arrive at each intersection at the onset of yellow (or forcing the platoon to arrive at the camera-enforcement intersection only at onset of yellow). This significantly increases drivers' exposure to RLR situations. In addition, most vehicles will experience repeated stopping at intersections, which will increase drivers' frustration and encourage aggressive behavior. In the platooning and congested conditions, if the leading vehicle crossed the stop line after the onset of yellow, following vehicles are likely to follow to avoid excessive delay. Drivers' behavior in close car-following conditions is discussed in detail later.

When the signal system is designed with an offset coordination system, drivers may not perceive any coordination in the system. An alternative to trap red-light runners is changing the setting of one signal in a well coordinated system. While drivers expect every signal in the system to stay green, one of them violates their expectation. For instance, one signal (the signal with camera enforcement) from the system is forced to change to yellow when the vehicle platoon arrives. As a result, some drivers may run the red-light due to the violation to their expectation.

Obviously, the offset coordination is undesirable from both efficiency and safety perspectives. The offset coordination increases the delay to drivers. The capacity of such a system will be much lower than a coordinated system, which is very likely to cause unnecessary congestion in the system. From the safety point of view, many studies have indicated that coordinated system was able to reduce crashes and injuries (Berg et al., 1986; Hulfine and Adams, 1995; Khalaf et al., 1995; Schlabbach, 1988; Shinar et al., 2004; Gårder, 2004).

4.3.7 Signal Control Strategy

Many studies have been performed on the influence of signal control strategy (i.e. pre-timed v.s. actuated) on RLR occurrences and crashes. The majority of these studies show that vehicle-actuated traffic signal systems have a higher rate of red-light violations or

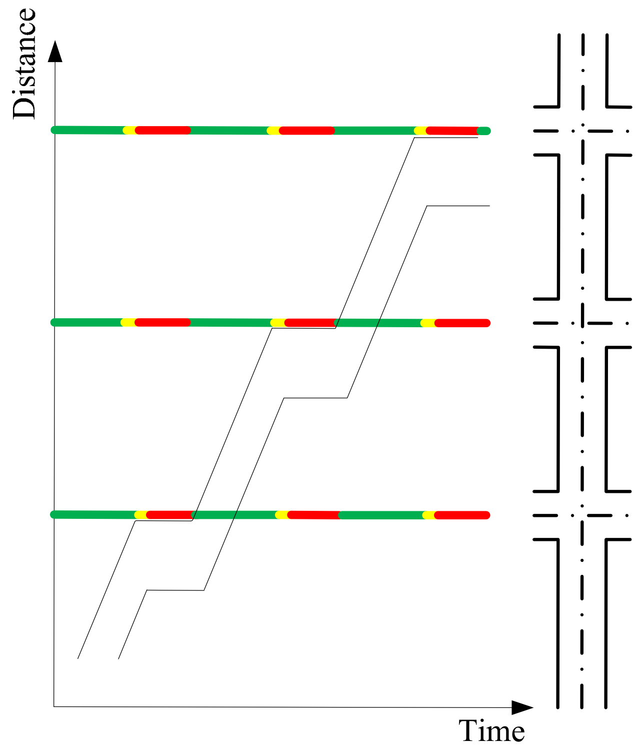


Figure 4.4 Signal coordination offset

crashes. Hasim (2009) reported a 56 % higher violation rate in actuated systems compared to other types of signal system. Mohamedshah et al. (2000) found that fully actuated signals had more crashes than semi-actuated and pre-timed signals controlling for other factors. The number of expected RLR crashes for fully actuated signals was approximately 35-39% higher than pre-timed signals. Based on previous studies, Bonneson et al. (2001) compared the stopping probability curve of actuated and pre-timed intersections, shown in Figure 5.5. At the same travel time to stop line situation, the probability of stopping at actuated intersections is lower than that at pre-timed intersections.

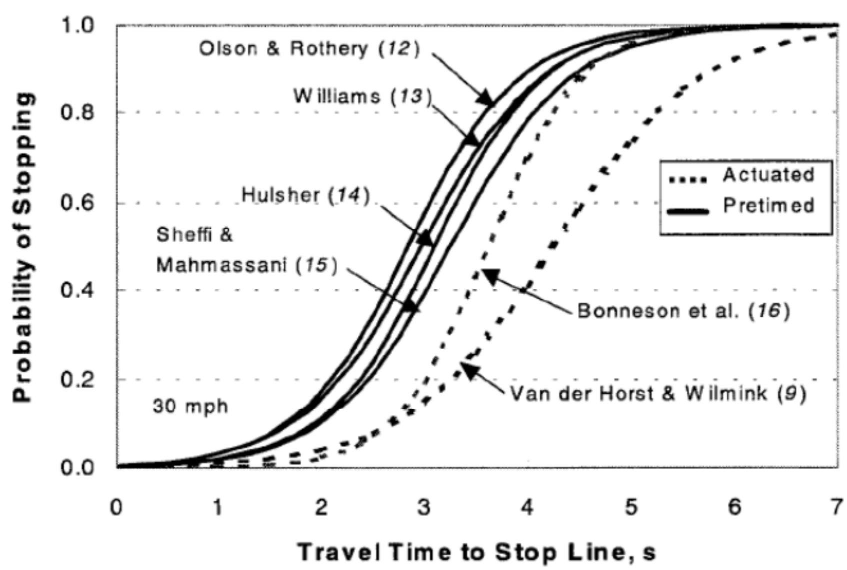


Figure 4.5 Probability of stopping as a function of travel time to stop line in actuated and pre-timed intersections (Bonneson et al., 2001)

Van der Horst el al. (1986) pointed out that the signal actuation contributes to a higher RLR rate because 1) drivers approaching an actuated intersection are less likely to stop since they develop an ad hoc expectancy as they travel without interruption through successive signals. They expect that each signal will remain green until they (and the rest of the platoon) pass through; 2) the actuated system is likely to generate a platoon that determines the termination of green for through traffic. Drivers desire to stay within the platoon through a series of interconnected signals. They are less willing to stop even if the yellow indication is displayed. A few related studies supported this argument. Bonneson et al. (2002) observed a 21% increase in RLR violations when the platoon ratio (defined as the percentage of vehicles arriving during the effective green divided by the percentage of effective green time in a cycle) was increased by 1 and an 18% decrease in the violation frequency when the platoon ratio was decreased by 1. Green (2003) indicated that 52% of vehicles running the red-light signal were immediately preceded by another vehicle. Zhang et al. (2008) reported that over 60% of RLR vehicles were with a headway less than 3s or belonged to a platoon.

However, there are also studies showed that signal actuation reduced RLR occurrences. Hasim (2009) pointed out that a vehicle-actuated traffic signal system had better performed in terms of red-light violation rate. Puan and Ismail (2010) found that the percentage of RLR at actuated intersections was much lower than others. It should be noted that these two studies were both performed in Malaysia. Drivers' reaction to signal actuation could be different in different countries.

Based on these studies, it appears that actuated signals can potentially increase the number of RLR and thus increase revenue from RLC. However, further study is still needed based on local conditions where RLCs are installed.

4.3.8 Increase V/C Ratio through Signal Re-timing

Drivers are more likely to run a red-light in high volume-to-capacity (V/C) conditions because either they experience more frustration in congested flow conditions or they follow the behavior of the leading vehicle and try to stay in a platoon. Bonneson and Zimmerman (2004a) reported that a decrease in V/C ratio was associated with a decrease in red-light violations. The V/C ratio in the range of 0.6 to 0.7 yields the lowest number of violations regardless of speed, path length, yellow duration, heavy-vehicle percentage, cycle length, phase duration, or traffic volume. Several other studies also identified correlations between traffic flow rates and the incidence of RLR events (Baguley, 1988; Hasim, 2009; Porter and England, 2000) and crashes (Mohamedshah et. al., 2000). Also, many studies indicate that drivers are more likely to run red-light in close-following driving situations (Allsop et al., 1991; Green, 2003; Zhang et al., 2008). Figure 4.6 shows the probability of stopping in different car-following situations based on the study of Allsop et al. (1991).

Based on these studies, increasing the V/C ratio by timing the traffic signal could potentially increase the number of red-light violations. The timing of the signal is not likely to change the total traffic volume (V). However, the capacity of intersections (C) is largely impacted by the signal timing. For instance, the aforementioned offset

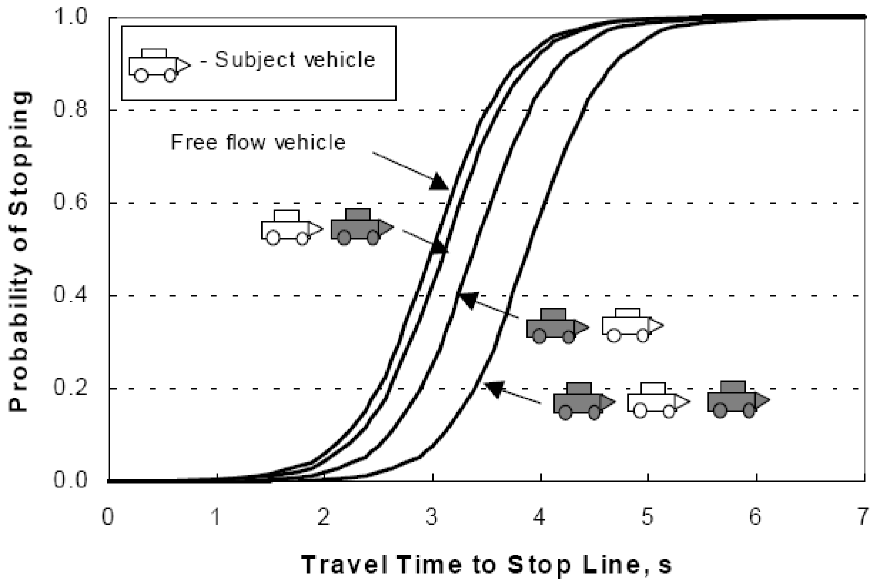


Figure 4.6 Probability of stopping as a function of travel time to stop line in different car-following situations (Bonneson et al., 2001)

coordination system may significantly decrease the capacity. Some other signal timing measures, like increasing the red-to-green ratio, installing an unwarranted dedicated turn signal phase, etc., can be considered. Depending on conditions of specific intersections, different signal timing strategies can be designed.

Clearly, increasing the V/C ratio by intentionally decreasing the capacity lowers the efficiency of intersection operation, although it may be able to generate more revenue from RLC.

4.3.9 Countdown Timer and Flashing Green

Pre-warning signals like countdown timer and flashing green provide drivers advance information regarding the onset of yellow, which may change drivers' stop/go decision at signalized intersections. Motorist countdown timers are not widely used in the US, but pedestrian countdown timers are very popular at signalized intersections. Although they mainly provide assistance to pedestrians on crossing streets, some drivers also use the information to help their stop/go decision making. Consensus has not yet been reached about whether such kind of devices encourages or prohibits RLR behavior. Long et al. (2011) conducted a study in China and found that intersections with countdown timers had higher rate of RLR. Ibrahim et al. (2008) also reported that the rate of RLR at countdown timer equipped intersections was higher than those without countdown timer. Lum et al. (2006) found that the countdown device would only help to encourage stopping but not curbing red-light violations. Drivers are more likely to run a red-light at countdown timer equipped intersections because either they take advantage of the information provided by such devices or the warning device causes drivers' indecision on stop or crossing. Mahalel and Prashker (1987) found that when a 3s yellow was preceded by a 3s flashing green, the indecision zone ranged from 2 to 8s which is larger than that for signals without a flashing green.

However, a few studies indicated that countdown timers reduced the number of red-light violations (Kidwai et al., 2005; Limanond et al., 2010; Ma et al., 2010; Napiah et al., 2007). It appears that pre-warning devices do have impact on drivers' decision making at signalized intersection and thus RLR behavior. However, whether the installation of such devices increases or decreases RLR is not conclusive. Studies based on local conditions are still needed before implementing this kind of measure.

4.4 Discussion

One of the major challenges with implementing RLC policy is the incentive of tangible revenue for industry and the municipality contrasted with external cost savings such as safety and congestion whose value is not easily captured or internalized by the public sector. As such, there is a tremendous temptation to modify the signal systems to increase revenue, particularly if restrictive legislation forces municipalities into contractual challenges with vendors (e.g. revenue guarantee requirements).

This paper is not advocating compromising safety or operations for revenue generation, but rather highlights a few easy-to-implement engineering tricks to increase RLR. This paper is meant to clarify the debate and highlight motivations for signal phase changes. The public sector can view this and reflect on the motivation for changing signal operations. The public stakeholders can use this paper to identify if their municipality is changing signal timing based on solid engineering evidence or based on increasing revenue.

Based on previous discussions, Table 4.2 summarizes these measures and an estimation of their effectiveness on increasing red-light violations and thus revenue along impacts on safety and efficiency.

The effectiveness and impacts of these measures are discussed in the circumstance that only one single measure is implemented. While each measure has its merits and faults, a combination of more than one measure may produce good results in maximizing RLC revenue and low impacts on safety and efficiency. Since drivers behavior vary among different driving conditions and driver populations, the effectiveness of these measures also varies with time and location. Studies based upon local conditions should be conducted before implementing such measures.

Our study focused on some engineering measures, particularly focusing on low cost modification of signal phase. Our study is not exhaustive in terms of clever strategies possible to increase RLR. In the survey of literature, the authors noticed that there are some other measures adopted by some transportation agencies for increasing revenue, such as removing “Signal Ahead” signs. We also do not consider effects of changing

Table 4.2 A list of measures and their effectiveness, safety impacts, and efficiency impacts

Measures	Effectiveness at Increasing RLR	Negative Safety Impact	Negative Efficiency Impact
Shorten yellow	High	High	Low
and/or lengthen all-red		Low	Low
Vary yellow duration	High	Low	Low
Shorten cycle length	High	Low	Demand Dependent
Lengthen cycle length	Low	Low	Demand Dependent
Increase R/G ratio	Moderate	Low	Moderate
Increase speed limit	High	High	Low
Signal coordination offset	High	Low	High
Actuated signal control	Uncertain	Low	Low
Increase V/C ratio	Low	Low	High
Pre-warning device	Uncertain	Uncertain	Low

signal head configuration and intersection geometry, which may potentially change driver's RLR behavior. For example, in areas with abundant heavy vehicles, signal head placement to mitigate occlusion from following vehicles could influence RLR.

CHAPTER 5 THE EFFECTS OF ANGLE-OF-TURN AND TANGENT ON RIGHT-TURN TRAFFIC

(This chapter is a slightly revised version of a paper to be submitted by Qiang Yang, Lee D. Han)

5.1 Introduction

The right-turn vehicles at intersections or driveway locations have effects on the efficiency and safety of traffic operation. Vehicles turning into driveway or joining major road from driveway enforce closely following vehicles to slow down and cause delay to the traffic. At intersections, right-turn vehicles affect the delay and capacity of traffic operation.

Geometric characteristics of turning lanes, such as angle-of-turn, radius, lane width, etc. affect the characteristics of turning vehicle operations. There is a lack of studies to investigate the influence of right-turn lane configurations, especially the angle-of-turn, on the operation of right-turn traffic. For instance, right angles are widely adopted in the design of driveways although such a design enforces drivers to significantly decrease their speed for a comfortable turn. Other angle designs and the associated turning characteristics are not examined. Another example is skewed intersections, which are popular in some cities like Washington D.C. At these skewed intersections, drivers may experience very shape right turns. The influence of the skew angle and other related intersection configuration parameters on right-turn vehicle operations is unclear.

The problem engineers face is a lack of information regarding the impact of poorly designed right turn lanes on right-turn vehicle operations at signalized intersections and driveway access points, as well as potential improvements that could be made. There are loose guidelines available for designers, but no quantitative analysis to help them understand the effects of the design on the delay and capacity for turning vehicles. As a consequence, capacity is sacrificed and delays are introduced to the intersection and driveway operations.

The objective of this study is to investigate the influence of angle-of-turn and tangent distance on the operation of right-turn vehicles. Right-turn traffic characteristics, like headway, capacity, travel time, speed profile, are studied with various right-turn lane configurations. The study results will provide useful references for engineers to deliberate on the appropriate design of right-turn lanes at intersection and driveway locations.

5.2 Literature Review

Highway Capacity Manual (TRB 2010) defines an adjustment factor to reflect the effect of right-turn traffic on the saturation flow rate. The right-turn adjustment factor f_{RT} is computed with Eq. (5-1).

$$f_{RT} = \frac{1}{E_R} \tag{5-1}$$

where E_R is the equivalent number of through vehicles for a right-turn vehicle. For protected right-turn lanes, E_R is 1.18. In Highway Capacity Manual, the adjustment method is generally applicable to right-angle turning lanes. The effect of angle-of-turn and radius is not included into consideration in the calculation of the right-turn capacity. Ibrahim et al. (2007) pointed out the limitation of HCM in their paper and proposed right-turn adjustment factors suitable for signalized intersections in Malaysia which took into consideration turning radius and the proportion of turning vehicles. Coeymans and Herrera (2003) also proposed a formulation to estimate turning vehicle equivalent factor and saturation flow rate in Chile. The formulation considered lane type and width, turning radius, and several other factors. Webster (1964) discussed the right-turn capacity issue at signalized intersections and compared saturation flow rates of right-angle intersections with different turning radii. The effect of the intersection angle was not discussed. Chandra et al. (1994) developed a mathematical model to estimate the right-turn saturation flow rate at signalized intersections. In the model, the right-turn saturation flow rate is a function of turning radius and traffic composition.

Right-turn maneuvers may cause delay to through traffic at driveway or intersection locations. The delay is related to several factors like traffic volume, traffic speed, right-turn lane configuration, etc. Alexander (1970) observed seven intersections on two-lane two-way highways in Indiana to determine the delay to through traffic due to right-turn vehicles. McShane (1995) used traffic simulation model to quantify the effects of right-turn activity on through vehicle travel speed. Stover and Koepke (1988) pointed out that turn speed is related to curb return radius and driveway width in a linear manner. James (1998) proposed a model for predicting the delay to major street through drivers due to right-turn activities from the outside through traffic lane on the major street. Wolfe and

Lane (2000) studied the delay to through traffic due to right-turn vehicles at intersections and found that with the decrease of the turning radius, the speed of right-turn vehicles decreases and the potential delay to following vehicles increases.

The turning lane angle and radius also have effects on safety besides their effects on traffic capacity and delay. Tarawneh and McCoy (1996) investigated the effects of right-turn lane geometries on the performance of drivers with respect to age and gender. Garcia and Libreros (2007) also performed a study to examine the effects of right-turn lane geometries, mainly skew angle, on driver behavior and vehicle trajectories. The drivers' field-of-view and intersection safety were analyzed and related to the intersection skew angle. Harwood et al. (2000) presented a model that quantified the safety effectiveness of realigning intersection approaches to reduce or eliminate intersection skew. An accident modification factor was proposed as a function of intersection skew angle. Dixon et al. (2000) identified several right-turn treatments, like lane configuration and traffic islands, and their effects on safety at intersections. However, both papers did not address the effects of turning radius and angle. Using kinematic measures in turning maneuver, Classen et al. (2007) compared turning vehicle stability and driver confidence between acute-angle and improved (right-angle) intersections. It was found that FHWA guidelines for intersection improvement were effective for driver safety, benefiting both old and young drivers.

Based on the literature survey, it is found that the influences of angle-of-turn on traffic capacity and delay are rarely studied. Although some researched the effects of various turning radius, they are all in the right-angle turning situation. Therefore, this study is to investigate driver behavior and traffic operation characteristics in various angle-of-turn and tangent (it equals to turning radius in right-angle situations) conditions when vehicles are turning right at intersections and driveway access points.

5.3 Methodology

A field experiment was designed to construct various right-turn scenarios with varying angles-of-turn and tangents. Paid subjects were recruited to participate in the study by driving through various turning scenarios. Driving data were recorded with a video camera. Traffic flow characteristics and driver behavior metrics were extracted from the video data and analyzed subsequently.

5.3.1 Experimental Design

The experiment and data collection were performed in the parking lot of a local middle school in Knoxville, TN. Traffic cones were placed in the parking lot to form right-turn entrance and exit lanes. Five cable markers were set along the lane to track the position of turning vehicles during the experiment. The first marker defined the stop bar location on the right-turn lane. The second marker defined the start point of the exit lane, while the rest three were used as 20-ft incremental reference points along the exit lane. The configuration in site is shown in Figure 5.1.

Figure 5.1 a) vehicle at stop bar b) vehicles in run

Figure 5.1 shows the configuration of the right-turn lane with 75 degree angle-of-turn. The first cable marker (i.e. the stop bar) can be moved to form any other angle-of-turn scenarios. Figure 5.2 shows the layout of different experiment scenarios with acute-angle, right-angle, and obtuse-angle. In different angle-of-turn scenarios, the stop line is rotated along the intersecting point of road edges. The rest four markers remain still. Figure 5.2 also shows different tangent scenarios. The tangent is measured along the road edge from the intersecting point. In the right-angle scenario, the tangent is equivalent to the turning radius; in other angle-of-turn scenarios, they are different.

Table 5.1 lists all experiment scenarios with different combinations of angle-of-turn and tangent that were tested in the study. Each scenario was tested for more than one run, as shown in Table 5.1.

Test subjects were students recruited at the University of Tennessee. They were paid for a day of participation in the test. Test subjects were driving their own vehicles (either SUV

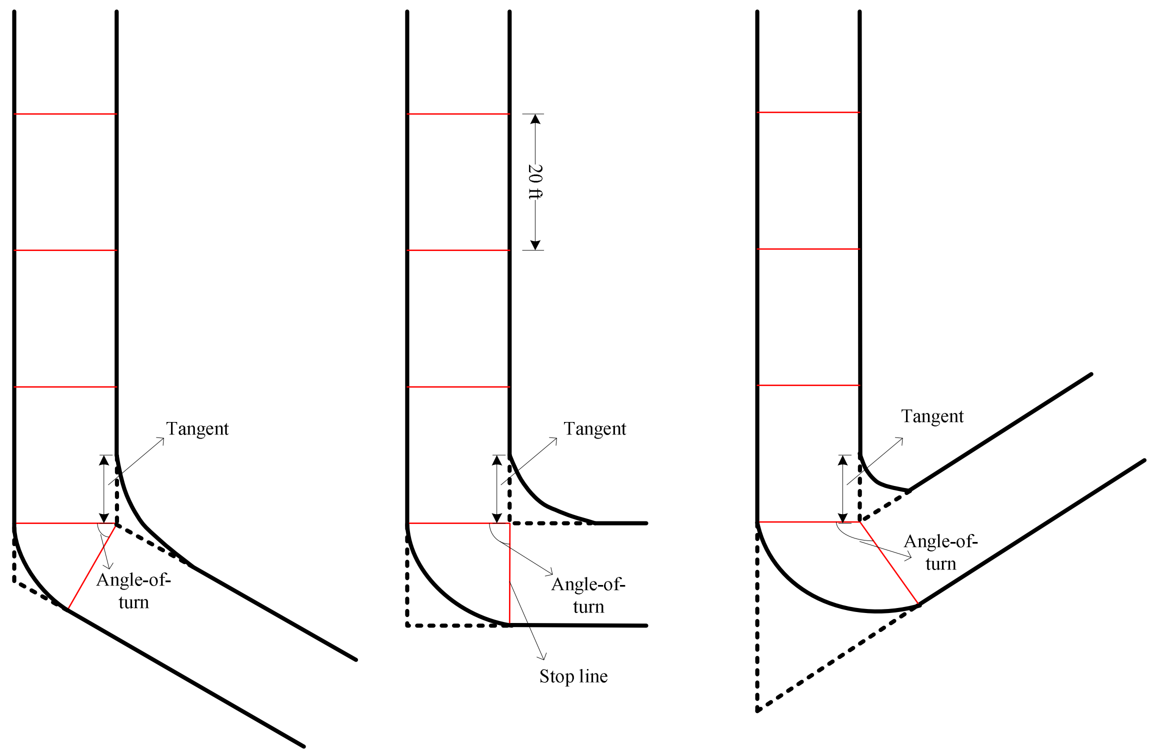


Figure 5.2 Experiment layouts of various angle-of-turn and tangent scenarios

Table 5.1 Number of test runs for experiment scenario

Tangent (ft)	Angle-of-turn (degree)							
	0	30	60	75	90	120	135	150
0	4	3	6	4	7	3	3	6
10			6	3	5	4	3	7
20			7	3	5		3	4

or passenger car) in the test. For each run, vehicles were lined up randomly behind the stop line. The experiment conductor standing at the stop bar location gave a signal to commence a run. Vehicles were discharged after the "signal indication" which simulated the traffic dispersion process on the right-turn lane at a signalized intersection. After all vehicles in line finished the run (i.e. passed the last marker), participants were asked to park their vehicles in the parking lot side by side so the order of vehicles in queue can be rearranged. The vehicle order in line was randomized from run to run to eliminate the order effect which may impact the results of the study.

5.3.2 Data Collection

A video camera was used to collect data in the study. The camera was set at a vantage point from which the entire experiment scene could be covered. All the tests and data collection were finished in one day.

5.3.3 Data Processing

Videos were manually inspected. Videos were recorded to a precision of 30 frames per second. Before the data extraction, videos were registered with special software to add stamps onto each frame in the format of "hh:mm:ss;ff", which denotes the exact hour, minute, second, and frame number. The marker locations were also highlighted in the video. Figure 5.3 shows a snapshot of the video file. Then, the time that the front wheel of each vehicle passed each marker was extracted and recorded into a spreadsheet.

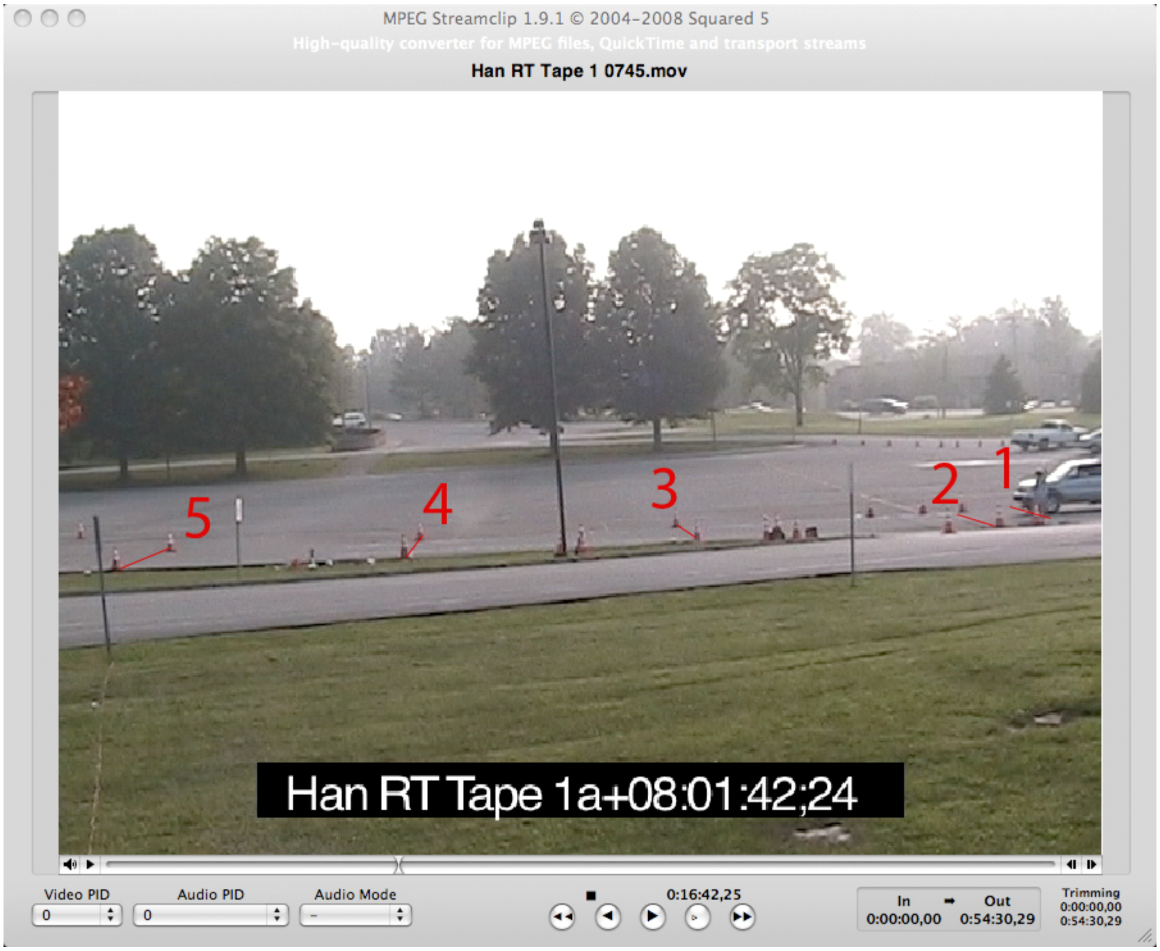


Figure 5.3 A snapshot of the video data

With the time information, vehicles' travel time between markers and headway between vehicles could be easily calculated. Vehicles' speed in each segment between two markers was also calculated with the travel time and distance information. The speed between the first and second marker was not calculated since the distance between them varied with the movement of the stop line in different scenarios.

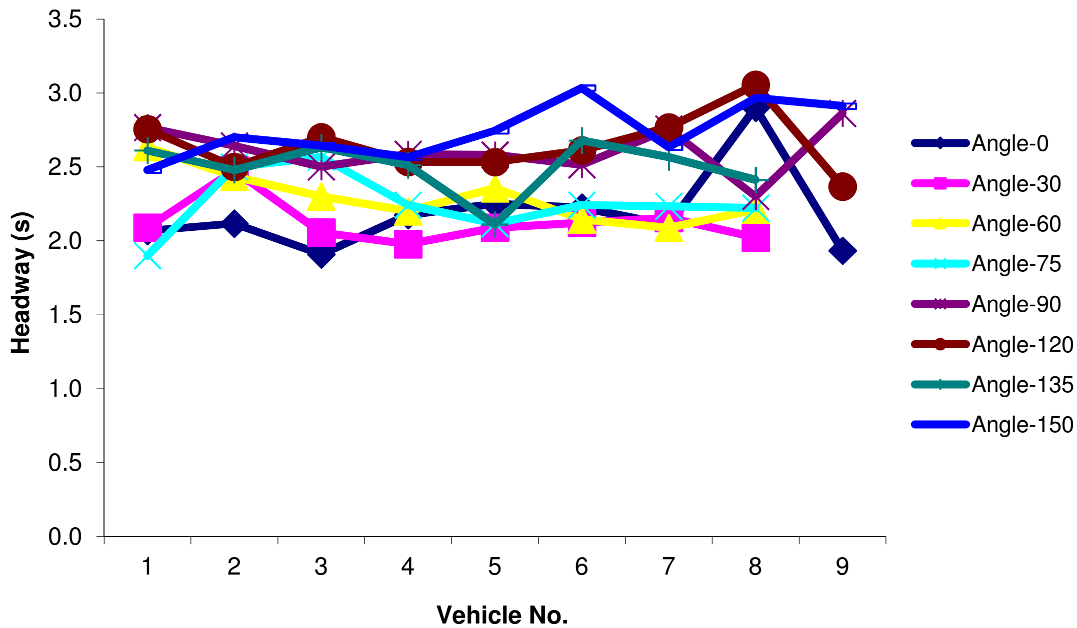
5.4 Results

In this section, the results of the data analysis are presented, which compares capacity, speed, and travel time in different right-turn angle-of-turn and tangent scenarios.

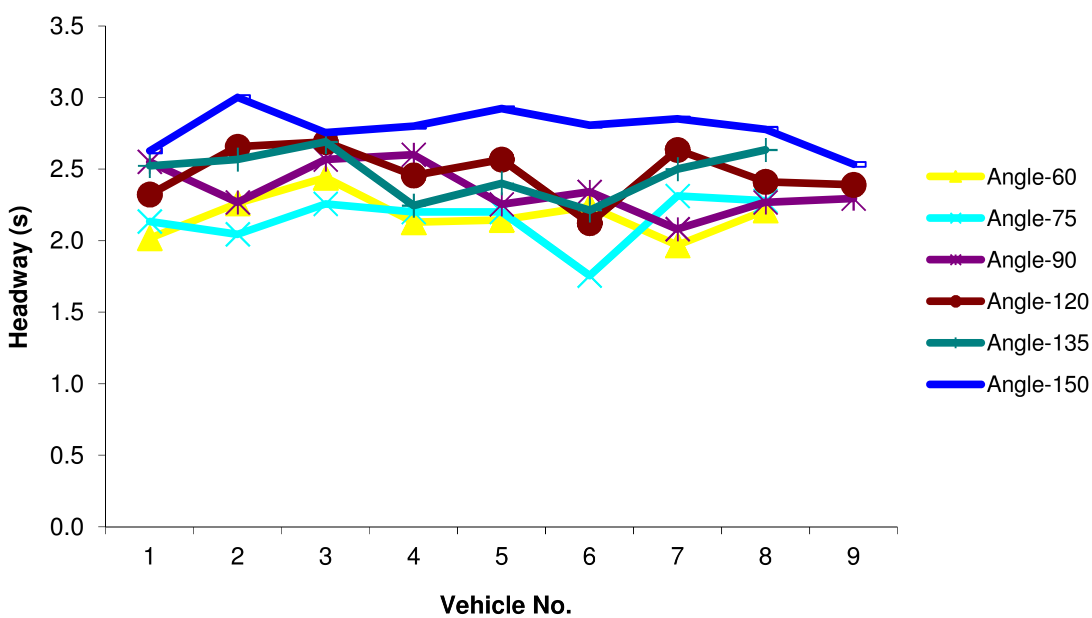
5.4.1 Capacity

Headways were calculated based on the time vehicles crossing the stop line. The first headway in queue is the start-up lost time of the leading vehicle, which is the time difference between the signal indication and the front wheel of the leading vehicle passing the stop line. Figure 5.4 shows the headways of vehicles in queue in different angle-of-turn and tangent situations. Most test runs were performed with nine vehicles in queue, while some had only eight or seven.

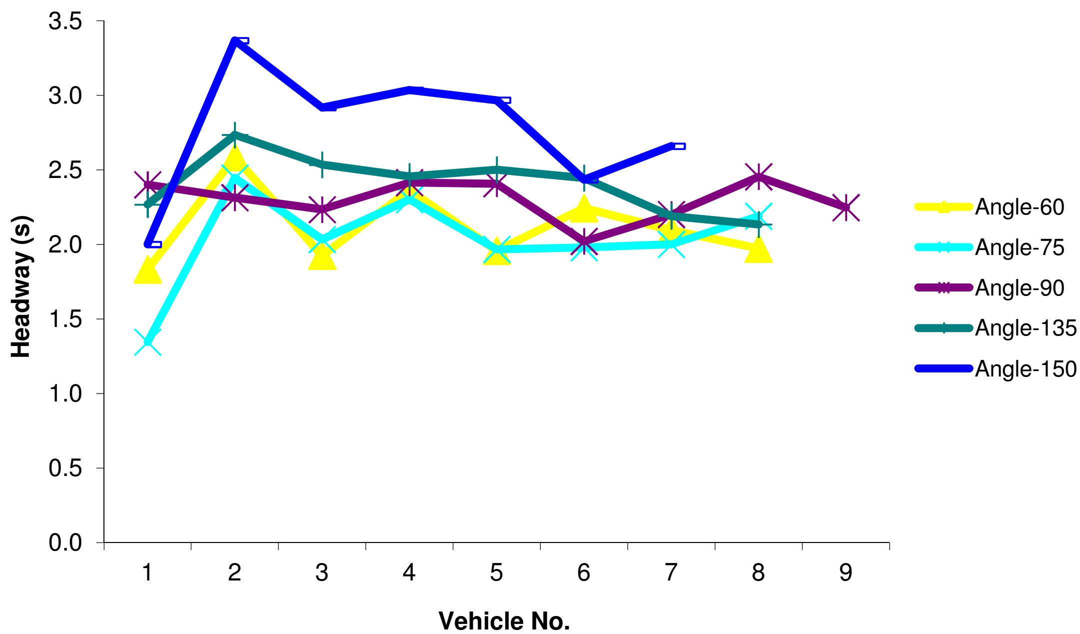
Figure 5.4 indicates that vehicle headways are mostly in the range of 2 to 3 seconds. Although show some fluctuations, headways generally increase with the increase of angle-of-turn in each of the three tangent conditions. An analysis of variance (ANOVA) was performed on the headways. Independent variables included angle-of-turn and



a) Tangent = 0



b) Tangent = 10 ft



c) Tangent = 20 ft

Figure 5.4 Vehicle headways in different angle-of-turn conditions tangent. The ANOVA test results show that angle-of-turn is significant to the headway (F=15.938, p-value<0.01). However, the post-hoc test shows that only the headway of the 150 degree angle-of-turn is significantly different from other angle-of-turn scenarios (p-value<0.01). There is no significant difference among the rest of them. The post-hoc test results are shown in Table 5.2.

The ANOVA test results indicate that tangent is also a significant factor to the headway (F=7.659, p-value<0.01). The post-hoc test shows that the no tangent scenario is significantly different from 10- (p-value<0.05) and 20-ft (p-value<0.01) tangent scenarios. However, there is no significant difference between 10- and 20-ft tangent scenarios.

The headway distribution in queue in the straight-through scenario is shown in Figure 5.5. The study shows a different headway distribution from the one given by HCM, which indicates that the headway decreases until the 4th to 6th vehicle in queue. The headway is relatively steady from the leading vehicle to the last vehicle in queue. Figure 5.5 compares the discharging headway distribution of the straight through scenario with that from a previous study (Greenshields et al., 1947). Both of them show the median value and the range of headways for each vehicle position in queue. The data from the study of Greenshields et al. (1947) indicates a decrease of the headway from the leading vehicle to the 6th vehicle. After the 6th vehicle, the discharging headway becomes saturated and reaches a steady value of about 2 seconds. The trend is consistent with the HCM. However, the discharging headway in this study does not show a decrease from the beginning. The start-up lost time of the leading vehicle is almost equivalent to the

Table 5.2 P-values of post-hoc comparison tests of headway on the variable angle-of-turn

***P*-value**								
Angle-of-turn	**0**	**30**	**60**	**75**	**90**	**120**	**135**	**150**
0	1.000							
30	0.996	1.000						
60	0.999	1.000	1.000					
75	0.993	1.000	1.000	1.000				
90	1.000	0.926	0.729	0.679	1.000			
120	0.993	0.794	0.584	0.522	0.998	1.000		
135	1.000	0.929	0.835	0.768	1.000	1.000	1.000	
150	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.000

saturated headway, which is also around 2 seconds. Apparently, this experiment underestimated the start-up delay of the real condition at signalized intersections, which can be attributed to the difference between the experimental and real start-up conditions. Experiment participants could be more alerted and better prepared since the "green" indication was given shortly after vehicles were lined up. There was no long waiting period as the red phase in a real signalized intersection. Also, the "green" indication was given by the experiment conductor by waving the hand. It took a slightly longer time than the signal transition which occurs immediately in the real intersection. It gave drivers some time to react to the signal and thus shortened the start-up delay.

To better understand the influence of angle-of-turn and tangent on right-turn traffic capacity, the average headway in queue in different angle-of-turn and tangent scenarios is calculated and shown in Figure 5.6 as flow rate.

Following findings are observed in Figure 5.6.

1) The right-turn traffic capacity generally decreases with the increase of angle-of-turn especially when it is larger than 75 degree. It appears that the right-turn discharging capacity can be significantly improved if the angle-of-turn could be reduced to a level below 75 degree.
2) In small angle-of-turn situations (less or equal to 120 degree), the tangent has remarkable influence on the traffic capacity. The traffic capacity increases with the increase of the tangent. In sharp right-turn situations (angle-of-turn larger or equal to 135), the effect of the tangent is barely discernible.

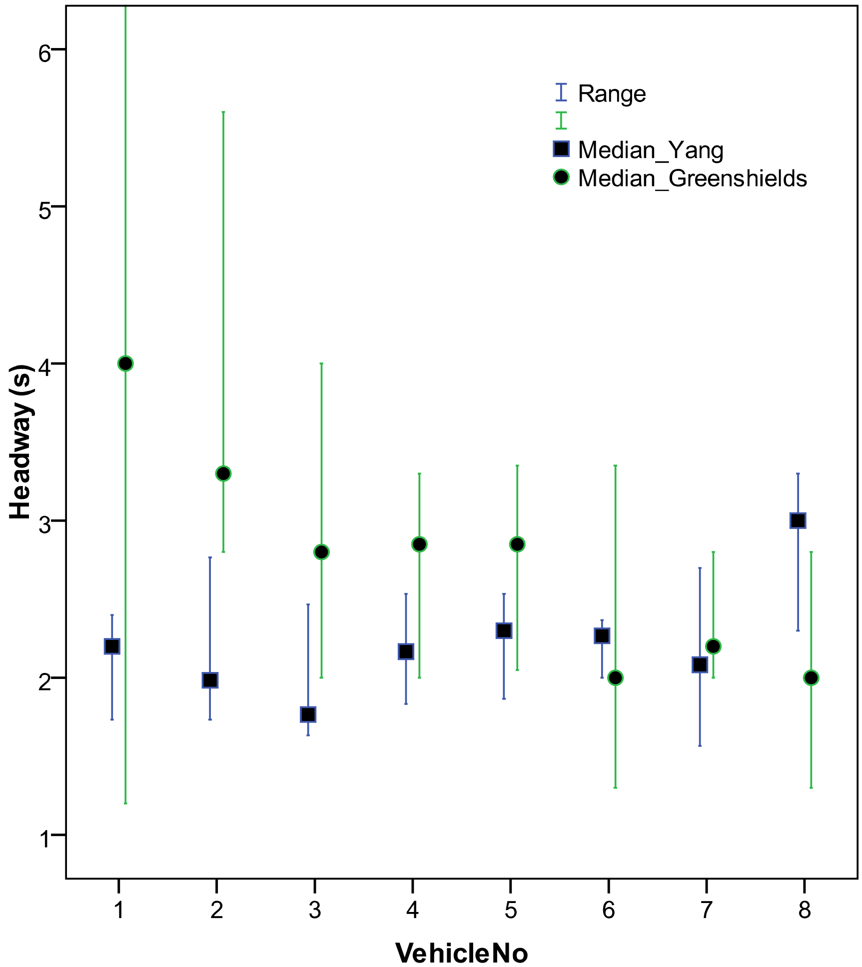


Figure 5.5 Comparison of the discharging headway distributions

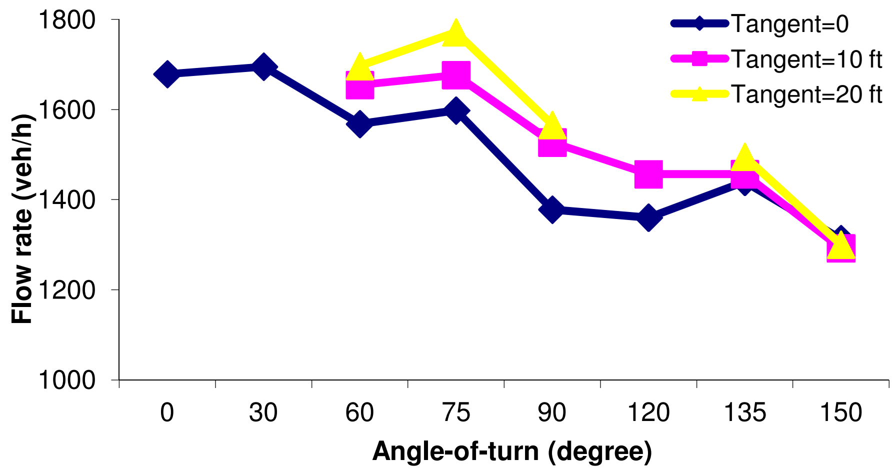


Figure 5.6 Discharging capacities in different angle-of-turn and tangent conditions

3) The capacities in straight through (0 degree angle-of-turn) and 90 degree right-turn situations are 1,679 and 1,378 veh/hr respectively. It results in a right-turn adjustment factor (i.e. the equivalent number of right-turn vehicles for a through vehicle) of 0.82, which is close to the HCM recommended value of 0.85. HCM does not consider or provide adjustment factors for other angle-of-turn conditions.

The current practice of HCM in calculating the right-turn adjustment factor is shown in Eq. (5-1), which does not consider the factors of angle-of-turn and tangent. Based on the discharging capacity data, the values of the right-turn equivalent factor, E_R, in different angle-of-turn and tangent conditions are calculated with Eq.(5-2).

$$E_R(ij) = \frac{C_0}{C_{ij}} \tag{5-2}$$

where $E_R(ij)$ is the right-turn equivalent factor in the angle-of-turn i and tangent j scenario, C_0 is the discharging capacity of the straight-though scenario, C_{ij} is the discharging capacity in the angle-of-turn i and tangent j scenario.

A linear model considering angle-of-turn and tangent is built to calculate the right-turn equivalent factor, E_R, as shown in Eq.(5-3).

$$E_R = A + B * ANGLE + C * TANGENT \tag{5-3}$$

where *ANGLE* is the angle-of-turn in degree, *TANGENT* is the tangent in feet, *A, B, C* are regression coefficients.

Based on the right-turn equivalent factors calculated with Eq.(5-2), a regression analysis is performed. The analysis results show that both variables are statistically significant.

The obtained coefficients are *A*=0.93, *B*=0.002, and *C*=-0.004. The model yields an R square value of 0.79. Then, Eq. (5-3) can be expressed as

$$E_R = 0.93 + 0.002 * ANGLE - 0.004 * TANGENT \tag{5-4}$$

The HCM right-turn adjustment factor can be calculated with Eq. (5-5).

$$f_{RT} = \frac{1}{0.93+0.002*ANGLE-0.004*TANGENT} \tag{5-5}$$

A similar approach could be used to build a right-turn adjustment factor model for the capacity analysis in HCM. Although a model is proposed in this paper, it is based on the discharging flow in the experimental driving condition. Future studies in the field are recommended. Also, the study only considered three tangent scenarios. More tangent conditions should be studied in the future.

A left-turn experiment was not performed in this study. Considering left-turn as an equivalent right-turn with larger turning radius, a left-turn adjustment factor is estimated with Eq. (5). For a right-angle intersection with two-way two-lane roadways, the tangent is about 18 feet assuming a 12-ft lane width. With Eq. (5), the left-turn adjustment factor is 0.96, which is very close to the HCM recommended value of 0.95. However, with other roadway configurations, the value of the left-turn adjustment factor will vary. A future study is recommended to develop the left-turn adjustment factor equation for the analysis of left-turn capacity.

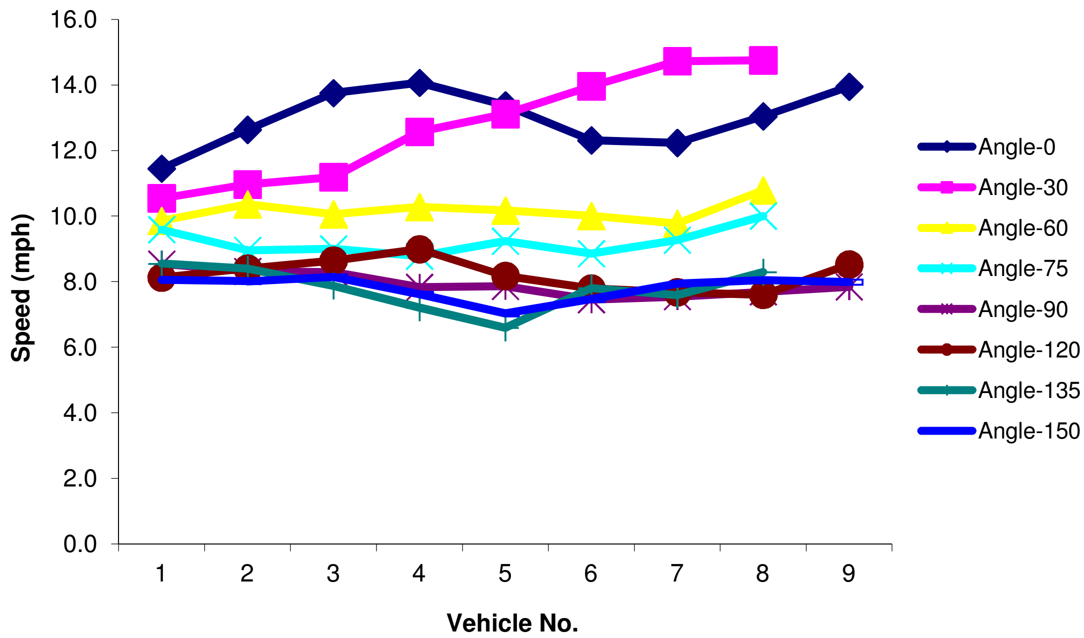
5.4.2 Speed

The speed profile of the traffic flow in the turning process with different right-turn lane geometry conditions is also examined. Figure 5.7 shows the speed of vehicles in queue in different angle-of-turn and tangent conditions. Since the vehicle speed during turning process (i.e. between the first and second marker) cannot be accurately calculated as stated earlier, the speed on the exit lane immediately after finishing the turn (i.e. in the 20-ft segment between the second the third marker) is used in the analysis.

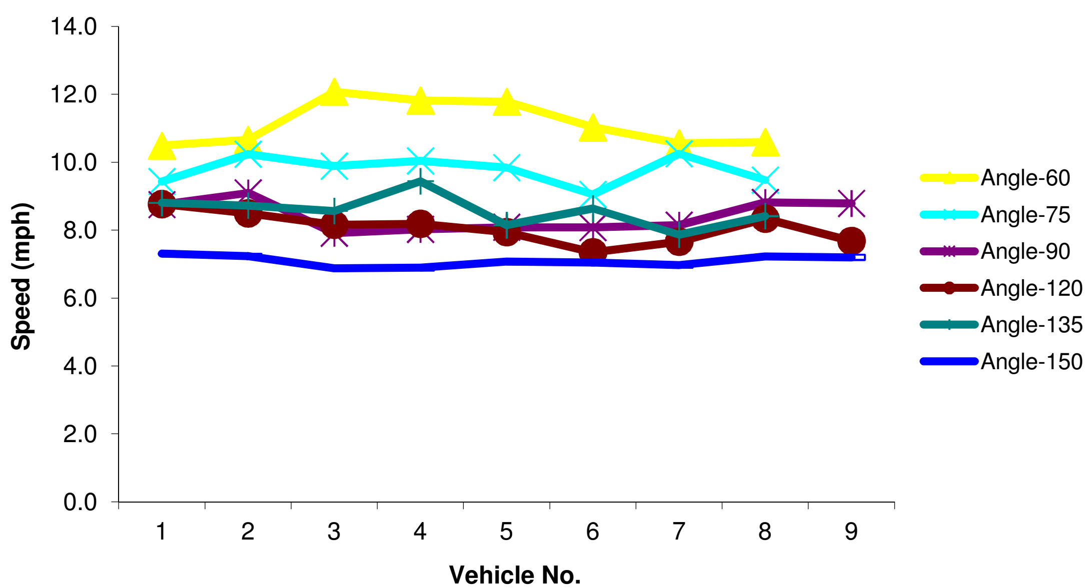
In Figure 5.7, it is seen that the vehicle speed decreases with the increase of angle-of-turn. In all three tangent scenarios, an angle-of-turn less or equal to 75 degree yields vehicle speeds significantly higher than other angle-of-turn conditions. In small turn situations, i.e. less than 60 degree, the observed vehicle speed in queue is generally increasing. In sharp turn situations, the observed speed in queue almost remains constant. It appears that when the angle-of-turn increases, the factor that dominates the impedance to vehicle speed or traffic dispersion changes from interactions between vehicles to right-turn lane geometrical constraints.

Figure 5.8 shows the average speed of vehicles in different angle-of-turn and tangent conditions.

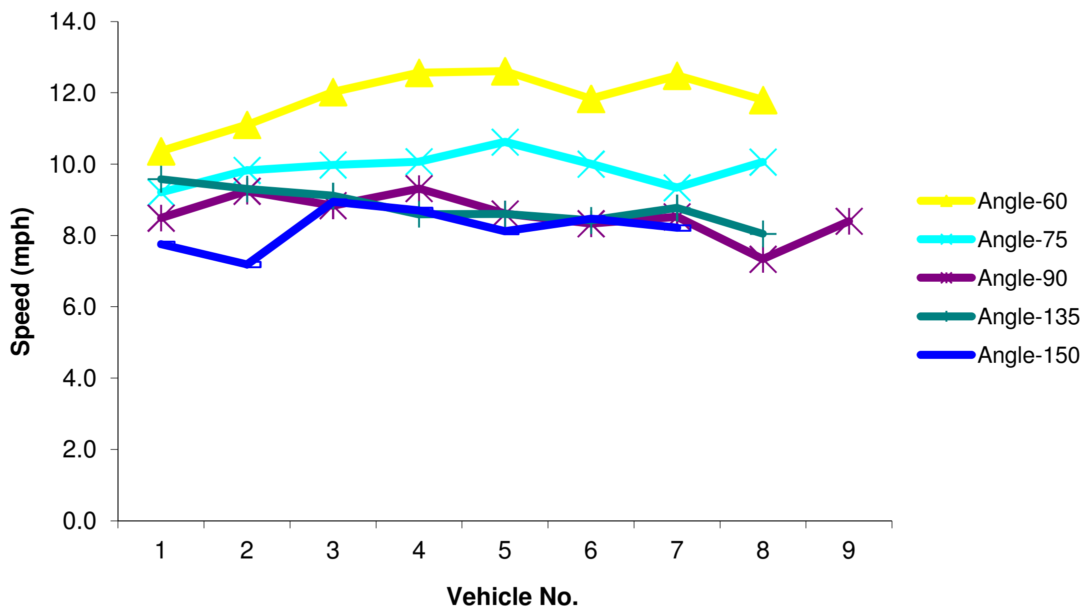
Figure 5.8 indicates that the average speed decreases significantly with the increase of the angle-of-turn when it is lower than 90 degree. The effects of the tangent on the average



a) Tangent = 0



b) Tangent = 10 ft



c) Tangent = 20 ft

Figure 5.7 Vehicle speeds in different angle-of-turn conditions

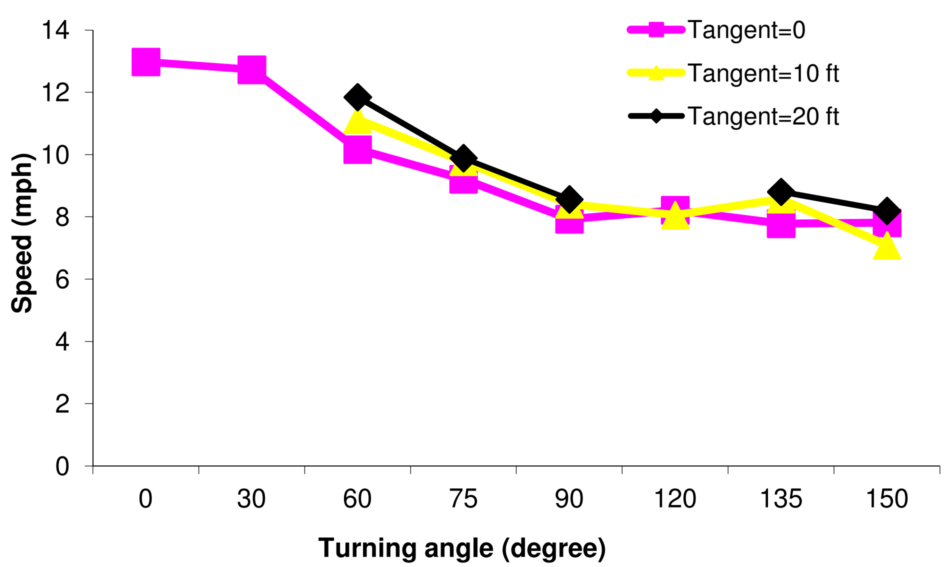


Figure 5.8 Average speeds in different angle-of-turn and tangent conditions

speed are not as remarkable as the angle-of-turn. However, since data are not complete for straight-through and 30 degree turn scenarios, the effects of the tangent in small turn conditions are unknown. An ANOVA was performed on the speed. The test results show that angle-of-turn is a significant factor to the speed (F=228.005, p-value<0.01). The post-hoc test results are shown in Table 5.3. It is seen that the differences of speeds between most pairs of angle-of-turn scenarios are significant.

The ANOVA test results indicate tangent is also a significant factor to the speed (F=13.921, p-value<0.01). The post-hoc test shows that the speed of 10-ft tangent scenario is significantly different from 0- (p-value<0.01) and 20-ft (p-value<0.01) scenarios. However, there is no significant difference between 0- and 20-ft scenarios (p-value=0.073).

5.4.3 Travel time

To study the delay to traffic in right-turn due to the right-turn lane geometry restrictions, the travel times in different right-turn scenarios are compared to the straight-through scenario. The travel time difference (TTD) calculated between right-turn and straight-through scenarios for each 20-ft segment so the delay profile in the entire right-turn process can be examined. Since the travel time in the first segment is dependent on the travel distance which varies among different angle-of-turn scenarios, TTD in the first segment is not analyzed. The analysis is performed for the second, third, and fourth 20-ft

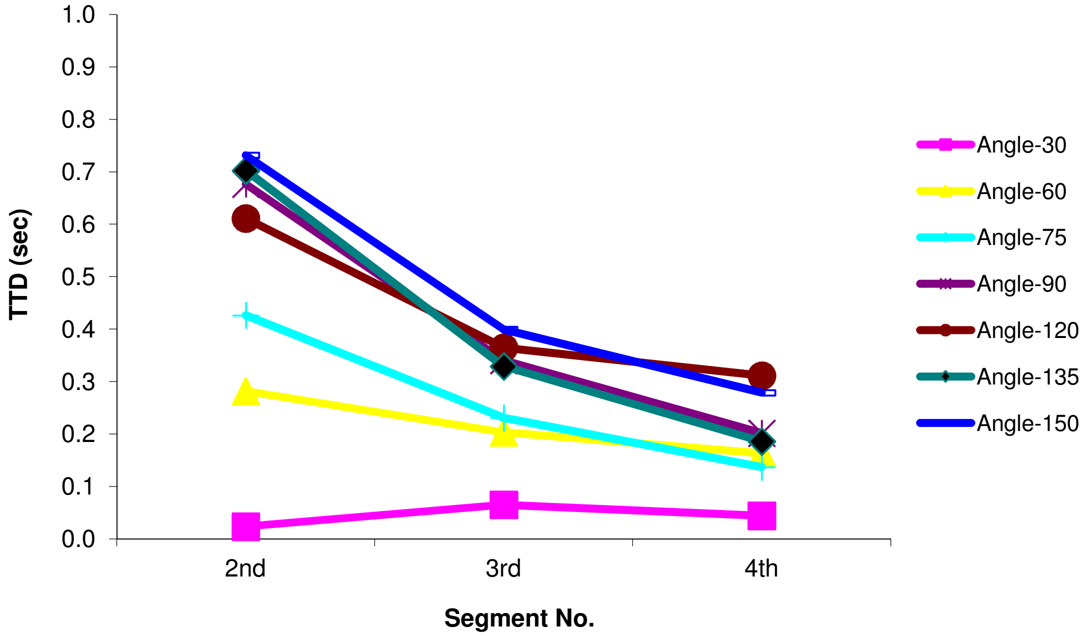
Table 5.3 P-values of post-hoc comparison tests of speed on the variable angle-of-turn

***P*-value**								
Angle-of-turn	**0**	**30**	**60**	**75**	**90**	**120**	**135**	**150**
0	1.000							
30	0.989	1.000						
60	0.000	0.000	1.000					
75	0.000	0.000	0.000	1.000				
90	0.000	0.000	0.000	0.000	1.000			
120	0.000	0.000	0.000	0.000	0.965	1.000		
135	0.000	0.000	0.000	0.000	1.000	0.892	1.000	
150	0.000	0.000	0.000	0.000	0.000	0.048	0.000	1.000

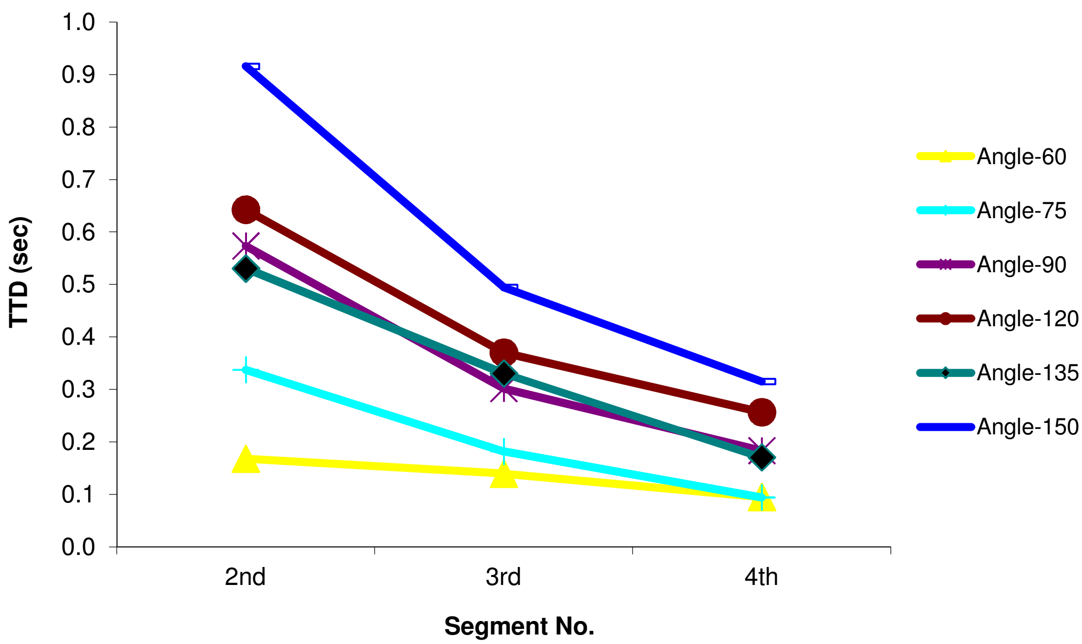
segments. Figure 5.9 shows TTD of each 20-ft segment in different angle-of-turn and tangent conditions.

Figure 5.9 indicates that the vehicle TTD in a 20-ft segment after the turn is between 0 and 1 second. In general, TTD increases with the increase of angle-of-turn. In the second segment which is immediately after finishing the turn, TTD is relatively large and the difference of TTD between different angle-of-turn scenarios is largest. It indicates that the vehicle speed after the turn is largely affected by the angle-of-turn. TTD gradually decreases towards the end of the test track as vehicles speed up and reduce the travel time difference from the straight-through scenario. In the fourth segment, the difference of TTD between different angle-of-turn scenarios is smallest. Even in the fourth segment, TTD is still above zero which means TTD due to the turn cannot be completely recovered toward the end of the 60-ft straight segment in the exit lane. An ANOVA was performed on the TTD in different segments with the independent variable angle-of-turn. The test results indicate that angle-of-turn is a significant factor to TTD in the second (F=144.431, p-value<0.01), third (F=98.823, p-value<0.01), and fourth (F=41.598, p-value<0.01) segment. Most pairs of angle-of-turn scenarios show significant difference in terms of TTD. Detailed results are listed in Table 5.4, 5.5, and 5.6.

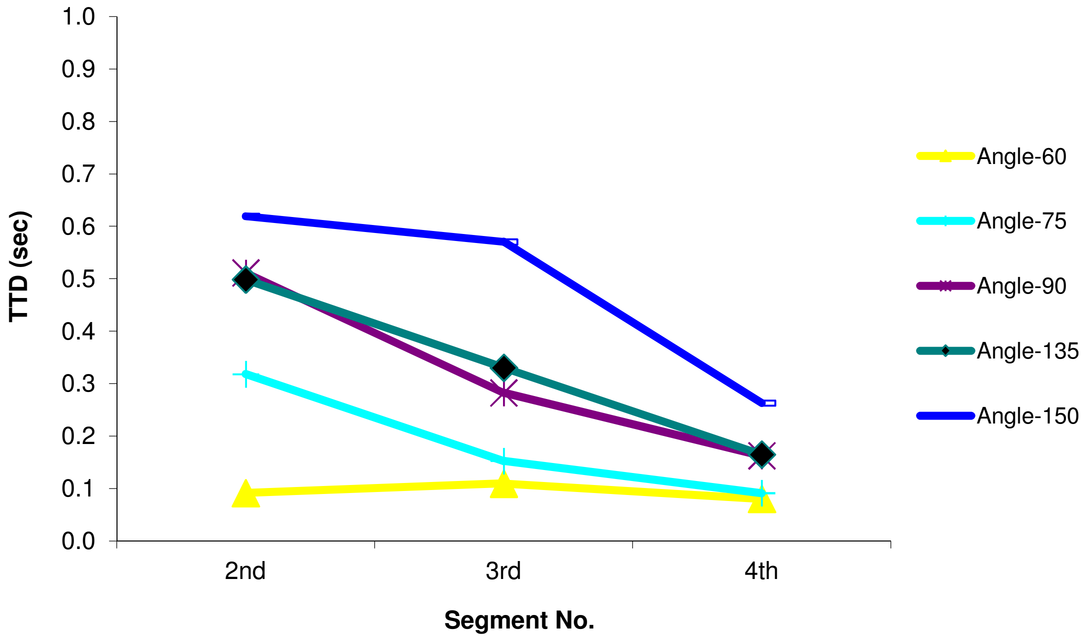
Comparing different tangent scenarios in Figure 5.9, it is found that the setting of tangent helps to reduce the effect of the turn on TTD, especially in the second segment immediately after finishing the turn. TTD in the third and fourth segments is not largely affected by the tangent. It indicates that the influence of the tangent, either on vehicle



a) Tangent = 0



b) Tangent = 10 ft



c) Tangent = 20 ft

Figure 5.9 TTD in different angle-of-turn conditions

Table 5.4 P-values of post-hoc comparison tests of 2nd segment TTD on the variable angle-of-turn

***P*-value**							
Angle-of-turn	**30**	**60**	**75**	**90**	**120**	**135**	**150**
30	1.000						
60	0.011	1.000					
75	0.000	0.000	1.000				
90	0.000	0.000	0.000	1.000			
120	0.000	0.000	0.000	0.968	1.000		
135	0.000	0.000	0.000	0.992	0.815	1.000	
150	0.000	0.000	0.000	0.000	0.000	0.000	1.000

Table 5.5 P-values of post-hoc comparison tests of 3rd segment TTD on the variable angle-of-turn

***P*-value**							
Angle-of-turn	**30**	**60**	**75**	**90**	**120**	**135**	**150**
30	1.000						
60	0.049	1.000					
75	0.001	0.200	1.000				
90	0.000	0.000	0.000	1.000			
120	0.000	0.000	0.000	0.105	1.000		
135	0.000	0.000	0.000	0.958	0.687	1.000	
150	0.000	0.000	0.000	0.000	0.000	0.000	1.000

Table 5.6 P-values of post-hoc comparison tests of 4th segment TTD on the variable angle-of-turn

***P*-value**							
Angle-of-turn	**30**	**60**	**75**	**90**	**120**	**135**	**150**
30	1.000						
60	0.149	1.000					
75	0.238	1.000	1.000				
90	0.000	0.000	0.000	1.000			
120	0.000	0.000	0.000	0.000	1.000		
135	0.000	0.005	0.016	0.996	0.000	1.000	
150	0.000	0.000	0.000	0.000	1.000	0.000	1.000

speed or TTD, is mainly effective during the turning process and within a short distance from the turn. The effect quickly fades away. An ANOVA was also performed on TTD in different segments with the independent variable tangent. The test results indicate that tangent is a significant factor to TTD in the second (F=19.889, p-value<0.01), third (F=3.715, p-value<0.05), and fourth (F=8.603, p-value<0.01) segment. For TTD in the second segment, the difference between 0- and 10-ft tangent scenarios is not significant while between others are significant; for TTD in the third segment, the difference between 10- and 20-ft tangent scenarios is significant while between others are not; for TTD in the fourth segment, the difference between 0- and 10-ft tangent scenarios is not significant while between others are significant.

The average TTD of the three 20-ft segments is calculated and shown in Figure 5.10. The average TTD is in the range of 0 to 0.6 second. As observed in Figure 5.10, TTD increases with the increase of angle-of-turn and the decrease of the tangent. Also, it is seen that the effect of the tangent on the average TTD is much less remarkable than that of the angle-of-turn. The reason is because the effect of the tangent diminishes quickly from the turn location while the effect of the angle-of-turn needs a longer distance to be recovered.

5.5 Discussion

The objective of the study is to provide some insights for potential improvement on skewed intersections and right-angle driveways, where the operation efficiency and

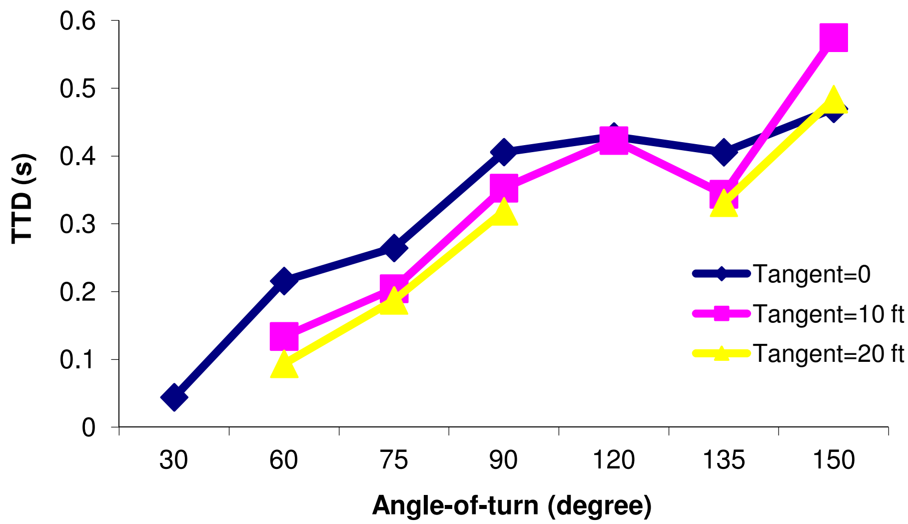


Figure 5.10 Average TTD in different angle-of-turn and tangent conditions

safety are of concern. Based on the study, it is found that intersections and driveways may have quite different right-turn issues and should be treated differently. For skewed intersections, assuming the traffic capacity for right-turn vehicles from road A onto road B (sharp turn) is reduced, the capacity for those from road B onto road A is actually improved. Based on the capacity analysis, the total capacity of right-turn traffic for a skewed intersection (i.e. the total capacity of two-way right-turn traffic) is even larger than a right-angle intersection. For instance, in the zero tangent condition, the total capacity of right-turn traffic for a right-angle intersection is 2,756 veh/hr (=1378×2), while that for a 30 degree skewed intersection is 3,008 veh/hr (=1,695+1,313). The efficiency of the entire intersection for the right-turn traffic is actually improved. The same results can be obtained for other skew angles. The reason for this is because the capacity can be significantly improved when the angle-of-turn is reduced to be lower than 90 degree but larger than 90 degree right-turns do not harm the capacity remarkably. For the same reason, it is recommended to design driveways for a smaller angle-of-turn to improve the efficiency and mitigate the influence to main street traffic.

Although the simple calculation shows that skewed intersections may actually benefit the right-turn operation efficiency, it is by no means to conclude that skewed intersections should be recommended in practice. Firstly, the traffic operation at skewed intersections in the field is more complicated than the experiment in the study. There may be some engineering measures in the field like right-turn channelization, tangent, etc., which could affect the actual capacity of the right-turn traffic. Secondly, only right-turn traffic is considered in the analysis. The skewed intersections may affect the capacity of straight-through and left-turn traffic as well. Thirdly, safety issues should also be considered in the design of intersections and driveways. Skewed intersections and sharp-turn driveways may lead to safety concerns like encroachment onto opposing traffic lane, very low turn speed and impedance to following vehicles on main road, abrupt deceleration and potential rear-end collisions, etc. Finally, this study only examined the start-up situations for an exclusive right-turn lane. Other more complicated situations for shared lanes and continuous traffic flows in higher speeds should be studied in the future. The speed difference between right-turn vehicles and straight-through vehicles is critical to the operation efficiency and safety at right-turn locations. A few previous studies (Alexander, 1970; James, 1998; McShane, 1995; Stover and Koepke, 1988; Wolfe and Lane 2000) examined this issue but none of them took into consideration the angle-of-turn. An experiment should be conducted in the future with right-turn vehicles approaching the intersection in different angle-of-turn and tangent conditions, instead of only starting up from the intersection location as performed in this study, so that the deceleration behavior and speed profile of right-turn vehicles could be studied.

Previous studies (Chandra, 1994; Herrera, 2003; Ibrahim, 2007; Webster 1964) showed that the capacity is influenced by the turning radius, which is related to the tangent in this study. The results of this study indicated that the angle-of-turn is even more significant in affecting the capacity. However, HCM does not consider either one of these factors in calculating the capacity of right-turn traffic. It is recommended to incorporate these two right-turn lane geometric parameters into the intersection capacity analysis in HCM.

The angle-of-turn and tangent are the focus of this study. Basically, the angle-of-turn is found to be significantly influential to right-turn capacity, speed, and travel time. In comparison, the influence of tangent is not as remarkable as the angle-of-turn. Since these two factors are interrelated in affecting right-turn driver behavior and traffic flow characteristics, both of them should be taken into consideration in practice. Other geometric characteristics, like lane width, and engineering measures, like right-turn channelization, also should be studied in the future.

5.6 Conclusions

This paper studied the influence of two right-turn lane geometric parameters, angle-of-turn and tangent, on right-turn driver behavior and traffic flow characteristics based on a right-turn vehicle start-up experiment. The study results indicate that both parameters, especially the angle-of-turn, are influential to vehicle discharging capacity, speed, and travel time. Although study results show benefits of discharging capacity for the right-turn traffic at skewed intersections, it is not recommended in practice for skewed intersections due to several limitations in the scope of this study. The results also show that small angle-of-turn driveways could improve the operational efficiency. A model is proposed in the paper to take the angle-of-turn and tangent into consideration in the calculation of the right-turn traffic discharging capacity. A similar approach is recommended for the capacity analysis in HCM with further studies.

Future studies are recommended from the following perspectives.

1) In addition to efficiency issues, safety performances like speed impedance, collision potential, interference to opposing traffic, etc., should be evaluated in the future for intersections and driveways with different geometric characteristics.

2) Besides the start-up scenario, continuous traffic flows approaching the turn and their behavior like deceleration should be examined in the future. Also, the study should be extended to left-turn situations.

3) Some other right-turn lane geometric parameters like lane width and some engineering measures like right-turn channelization may be interrelated with the angle-of-turn and tangent in affecting right-turn driver behavior and traffic flow characteristics. All these factors should be incorporated in future studies and a more comprehensive traffic capacity prediction model should be built.

CONCLUSIONS

This dissertation complied a few studies on driver behavior and traffic characteristics at intersections and driveway access points. The safety and operation efficiency at these locations in a road network are the major concern of this research. A series of field experiments or observation studies in different places were performed to support this research.

First, a countdown timers study was performed in China about their influences on driver behavior. It was found that the presence of countdown timers may encourage yellow running behavior and late entry into intersection in China. Driver behavior and traffic characteristics are heavily associated with the local conditions like the culture. A future study is recommended to compare the driver behavior in different countries.

Second, a phase gradient method was proposed for the general application to the studies of driver behavior and traffic characteristics at signalized intersections. A case study on red-light cameras was performed at Knoxville, TN. This study proposed the concept of the phase gradient method. A future study is recommended to build a mathematical model to capture the phase gradient based on more data.

Third, a study was performed to learn the legal issues and arguments about the usage of red-light cameras for the purpose of generating profits. A variety of engineering measures, mainly dealing with the setting of the traffic signal, are discussed which could be potentially used by municipalities or camera vendors to trap red-light runners and thus generating more revenues from the camera system.

Finally, an experiment was conducted to simulate the right-turn issues, which impact the safety and operation efficiency at intersections or driveway access points. Two turn lane geometric parameters, angle-of-turn and tangent, and their influences on driver behavior and traffic flow characteristics were studied. The study results indicate that both parameters, especially the angle-of-turn, are influential to vehicle discharging capacity, speed, and travel time. Some suggestions are given regarding the improvements to skewed intersections and driveway designs.

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VITA

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During his almost ten years of study and research in the field of transportation engineering, Mr. Yang explored a wide spectrum of topics mainly including traffic modeling and analysis, traffic safety, human factors, transportation infrastructure management, sensing techniques, spatial analysis, and pavement engineering.

Mr. Yang is active in ITE and TRB activities and served as a reviewer for several academic journals. He holds an Engineer-in-Training certification. During his PhD study, he was awarded several scholarships, paper awards, and his research was featured in media a few times.