COMPARING DRIVER AND CAPACITY CHARACTERISTICS AT INTERSECTIONS WITH AND WITHOUT RED LIGHT CAMERAS

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Abstract

The primary purpose of installing Red Light Cameras (RLCs) is to improve intersection safety by discouraging motorists to cross the intersection when the signal for approaching vehicles turns red. Due to the fear of being fined when crossing an RLC equipped intersection at the onset of the red signal, many approaching vehicles may have a tendency of stopping during the yellow phase. This tendency may impact intersection capacity, which can be significant in congested transportation networks during rush hours, especially when several intersections are equipped with RLCs along a sequence of traffic signals, resulting in a disruption of traffic progression. In order to examine the driver and capacity characteristics at intersections with RLCs and compare them with those without RLCs we develop a binary probit choice model to understand driver's stop and go behavior at the onset of yellow intervals, also known as dilemma zone. Further, in order to capture the impact to intersection capacity at intersections with RLCs we develop a probabilistic computational procedure using data from ten intersection pairs (with and without RLCs) in the Baltimore area. The results indicate that, in general, RLCs reduce the intersection capacity since driver's travel behavior is influenced by the presence of the cameras. Other contributory factors for the so-called capacity reduction, such as driver population (e.g., familiar vs. unfamiliar drivers) and traffic-mix (e.g., trucks vs. passenger cars) characteristics have been left for future works.

Keywords: Red Light Camera, Red Light Running, Dilemma Zone, Intersection Capacity, Driver Characteristics, Capacity Characteristics

INTRODUCTION

For over a decade, several cities in the United States have installed Red Light Cameras (RLCs) at signalized intersections. The main reason behind installing a Red Light Camera (RLC) is to reduce Red Light Running (RLR) accidents. Although the use of RLCs is intended to improve intersection safety, their influence on drivers' stop and go decisions during the yellow interval has not been studied thoroughly. Driver behavior at RLC equipped intersections is a major factor contributing to the safety and operation of the intersections. A drivers' decision whether to cross or stop at the intersection during the yellow signal interval generally depends on a number of factors, such as the length of yellow signal interval, approach speed, road and intersection geometrics, and his/her attitude. Depending on the time the driver arrives at the intersection and other unexpected conditions present at that time s/he can either stop if there is sufficient stopping distance or clear the intersection if there is enough clearing time before the signal turns red. Thus, the driver's decision at RLC intersections during the yellow interval can be seen as a binary process in which the two main decisions are either to come to a stop or cross the intersection.

Each of the two decisions have their own consequences, which can impact the traffic operation at the intersection. The stopping decision may result in a rear-end collision and the crossing decision may result in a side collision. Moreover, the travel behavior at non RLC (NRLC) and RLC intersections may not be the same for all drivers. One scenario is that fearing RLR violation ticket, some drivers who are aware of the presence of RLCs may decide to stop during yellow regardless of the availability of safe clearing distance before the onset of the red signal. The cumulative impact of such stopping may result in significant delay in a congested transportation network, especially during rush hours. Such stopping may also impede the smooth progression of traffic along arterial roads during rush hours.

The objective of this study is to compare driver and capacity characteristics at RLC and NRLC intersections. In order to investigate the driver characteristics, driver behavior is examined using a binary probit model. In order to investigate the capacity characteristics, a RLC reduction factor is proposed to calculate the saturation flow rate.

LITERATURE REVIEW

A summary of literature review relevant to driver behavior and intersection performance at RLC intersections is presented in Table 1. The focus of literature review shown in Table 1 is to explore the past studies on RLC and RLR with specific research objectives (Column 1), measures considered (Column 2), test-bed locations (Column 3), and analysis type (Column 4). A review of literature (Porter et al. (1999) (*1*); Porter and England (2000) (*2*); Retting et al. 1999 (*3*), (2002) (*4*); Tarawneh et al. (1999) (*5*); Shattler and Datta (2003) (*6*), IIHS 2011 (*7*)) suggests that several research have been undertaken in connection with RLCs and RLR incidents. Several studies mainly discuss the advantage of using RLCs qualitatively, such as reducing accidents or documenting installation guidelines for RLCs.

Some studies (Sharma et al. (2011) (*8*); Elmitiny et al. (2010) (*9*); Sharma et al. (2006) (*10*); Mahalel and Prashker, (1987) (*11*); Zimmerman and Bonneson, 2004 (*12*); Sunkari et al., (2005) (*13*)) reported that an option zone upstream of intersections at the onset of the yellow signal is associated with larger variability in the drivers' stop/go decisions. When the driver is going at a speed lower than the speed limit an option zone is created, i.e., an area where the driver can stop or cross successfully. When an approaching driver is traveling at a speed significantly higher than the posted limit then s/he can neither stop without slamming on the brakes or cross safely without running the red light. None of the above studies focused upon a possible capacity reduction due to defensive stopping of vehicles at RLC intersections.

Other group of studies focused on the effect of heterogeneous traffic and the time needed to cross the intersection, and their effect on the RLCs. Zimmerman (2007) recommended additional protection time for trucks by allowing for the additional time and distance that trucks require to stop and thereby, reducing the number of trucks in the dilemma zone and red light violations (*14*). Gates and Noyce (2010) (*15*) investigated the influence of vehicle type on various aspects of extended yellow on driver behavior, including brake response time, deceleration rate, and red light running occurrence at urban or suburban signalized intersections. Numerous other studies (Chang et al., 1985; Newton et al., 1997; Köll et al., 2003; Yan et al., 2007; and Papaioannou, 2007) were focused upon driver behavior associated with the signal change, in those studies, the probability of drivers' stop/go decisions was modeled as a function of the space or potential time from the stop line using multiple regression or other logit regressions *(16) (17) (18) (19) (20) (21) (22) (23) (24*). Based on that function, most drivers will either cross the intersection when they have a shorter distance or stop at the intersection when they have a longer distance from the intersection.

Adequacy of required yellow time also gained significant attention for urban and rural signalized intersections *(25) (26)*. Zheng et al. (2006) studied cycle failures because of improper signal timing with RLCs (*27*). Though this study is useful to extract number of cycle failures, it does not focus upon capacity loss or any other traffic flow performance measure. The highway capacity manual outlines the formulae for saturation flow, but depending upon various driver and capacity characteristics, there exist a potential opportunity for improvement (*28*).

The literature review presented here is by no means a comprehensive one; rather, it is designed to capture a cross-section of studies conducted on this subject during the last fifteen years. A comparison of traffic flow performance for RLCs in terms of capacity is missing in the literature. In addition, there is a need for a methodology which addresses the type of probability distribution function that best fits for vehicles arriving at the intersections during yellow.

TABLE 1 Summary of Literature Review on (i) Stop/Go Decisions during Yellow, (ii) Capacity Reduction, (iii) Red Light Running, and (iv) Red Light Camera

Research Objective (1)	**Measures Considered** (2)	**Study Area** (3)	**Analysis Type** (4)	**Authors** (5)
Crash reduction, safety issues, before-after studies	Total crashes, crash types,	Urban and rural signalized intersections	OLS, Poisson, Negative Binomial regression, empirical bayes method, etc.	Porter et al. (1999) (*1*); Porter and England (2000) (*2*); Retting et al. 1999 (*3*), (2002) (*4*);

				Tarawneh et al. (1999) (*5*); Shattler and Datta (2003) (*6*)
Driver Stop/Go Decision	Red-light running violation, lane position, Traffic Flow, Vehicle Type	High Speed Signalized Intersection	Tree Based Model; HCM control delay equation modification	Sharma et al. (2011) (*8*); Elmitiny et al. (2010) (*9*); Sharma et al. (2006) (*10*); Mahalel and Prashker, (1987) (*11*); Zimmerman and Bonneson, 2004 (*12*); Sunkari et al., (2005) (*13*)
Heterogeneous Traffic at RLC	Truck percentage, traffic flow, and highway geometry	Rural Signalized Intersections	Delay Reduction	Zimmerman K. (2007) (*14*); Gates and Noyce (2010) (*15*)
Performance of Red Light Running	Violations, stop and go during yellow	Urban Intersections	Quasi-experimental design	Chang et al., (1985) (*16*); Retting et al. (1999) (*29*); Retting et al. (2002) (*4*); Newton et al., (1997) (*17*); Koll et al. (2003) (*18*); Papaioannou (2007) (*20*)
Yellow Time for Left Turning Traffic	Geometric condition, speed	Urban Area signalized intersection	Geometric Design Formulation	Kim et al (2005) (*25*) ; Li and Abbas (2010) (*26*)
Studies on Cycle Failures, and Capacity Reduction	Capacity reduction because of signal failures	Signalized intersections	Experimental design	Zheng et al. (2006) (*27*)

METHODOLOGY

Intersection Analysis

In order to realistically investigate how a typical driver would respond when crossing through a RLC intersection as opposed to a NRLC intersection, it was necessary to collect data along arterial streets that had a series of such intersections in a progression, preferably with an alternating sequence of RLCs and NRLCs since this would ensure that same drivers crossed both RLC and NRLC intersections pairs. Therefore, we carefully designed our test bed and collected relevant data that affect driver and intersection capacity characteristics. We found ten such intersection pairs in the Baltimore area shown in Table 2 and Figure 1. The chosen RLC and NRLC intersection pairs have similar geometric characteristics. The posted speed limits at the study intersections are in the range of 25-40 mph; the total number of lanes vary from 2 to 6, signal cycle lengths vary from 100 to 120 seconds, and the yellow interval at all the intersections is 4 seconds. In Figure 1 each star represent a pair of RLC and NRLC intersections. Eight of the intersection pairs are located in Baltimore City and two in Baltimore County.

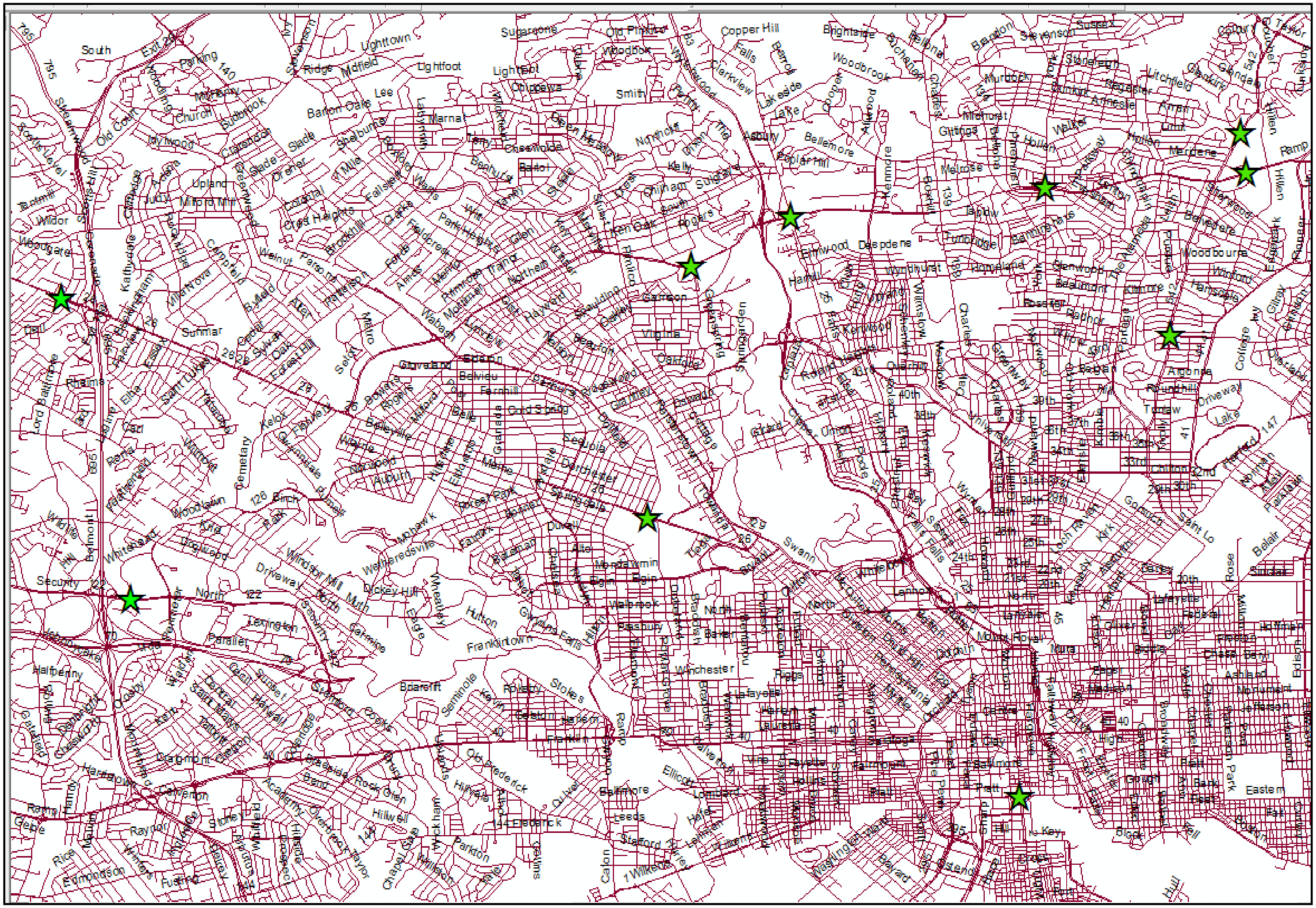

FIGURE 1 Location of RLC and NRLC Intersection Pairs for the Test bed

TABLE 2 Study Intersections and their Characteristics

	RLC and NRLC Intersection Pairs	Speed Limit (mph)	Number of lanes	Green time (sec)	Yellow time (sec)	All-Red Time (sec)	Cycle length (sec)
RLC 1 NRLC 1	Security Blvd at Whitehead Road Security Blvd at Woodlawn Drive	35	4	58	4	2	120
RLC 2 NRLC 2	W. Northern Pkwy at Green Spring Avenue W. Northern Pkwy at Green Spring Avenue	40	4	46	4	2	100
RLC 3 NRLC 3	E. Northern Pkwy at Waverly Way E. Northern Pkwy at Loch Raven Blvd	40	4	50	4	2	100
RLC 4 NRLC 4	Loch Raven Blvd at Loch Hill Road Loch Raven Blvd at Walker Avenue	40	2	44	4	2	100
RLC 5 NRLC 5	W. Northern Pkwy at Falls Road W. Northern Pkwy at Ronald Avenue	35	4	60	4	2	120
RLC 6 NRLC 6	Liberty Road at Washington Avenue Liberty Road at Lord Baltimore Drive	35	4	60	4	2	120
RLC 7 NRLC 7	Liberty Heights Ave at Wabash Avenue Liberty Heights Ave at Druid Park Drive	35	4	48	4	2	100
RLC 8 NRLC 8	Light Street at Pratt Street Light Street at Lombard Street	35	6	60	4	2	120
RLC 9 NRLC 9	Cold Spring Ln at Loch Raven Blvd Cold Spring Ln at The Alameda	25	3	56	4	2	120
RLC 10 NRLC 10	E. Northern Pkwy at York Road E. Northern Pkwy at Bellona Avenue	35	4	60	4	2	120

Distribution Function Analysis

At the ten RLC and NRLC intersection pairs a set of data were collected. Data collected include speed, distance to the stop line during yellow, motorists' stop and go decision, and the presence of RLCs. The observations were made during peak hours and included only motorists going on the through lanes, i.e., left and right turning vehicles were not included. Since the intersections are not frequented by truck drivers the only vehicle type used for this study is the automobile. A total of 600 vehicles which is 30 per intersection were counted. The total number of vehicle count is more than the minimum sample size required for a 95% confidence level significant testing.

From the distance to stop line and speed, the time to get to the stop line during the yellow time interval is computed. In order to find the best function to fit the time to get to stop line we analyzed the travel time at the ten intersection pairs with continuous distribution functions. Table 3 shows the functional form of five continuous distribution functions and the parameters used for the analysis. The parameter values in Table 3 are computed using standard formulas in traffic flow theory. Table 4 shows the computation of the actual, Normal, Lognormal, Exponential, Erlang, and Weibull distribution functions for the study intersections using the parameter values shown in Table 3. The comparison for the actual distribution functions of the RLC and NRLC intersections is shown in Figure 2. Figures 3 and 4 show the graph for the actual and all distribution functions for the RLC and NRLC intersections respectively.

The results from distribution functions analysis (figures 3 and 4) show that for both RLC and NRLC intersections the normal distribution is the closest function to fit the time to get to the stop line. The graphs for Lognormal, Exponential, Erlang and Weibull distribution functions are not the closest fit for the travel time data at the study intersections.

TABLE 3 Functional Forms of Continuous Probability Distribution Functions

Distribution	Probability Distribution Function	Parameters	Parameter Definition	Parameter Values
Normal	$f(x) = \frac{1}{\sqrt{2\pi\sigma^2}} e^{-\frac{(x-\mu)^2}{2\sigma^2}}$	μ and σ	Mean and standard deviation	$\mu_{RLC} = 3.98$ $\mu_{NRLC} = 4.22$ $\sigma_{RLC} = 2.32$ $\sigma_{NRLC} = 2.22$
Exponential	$f(x) = \lambda e^{-\lambda x}, x \geq 0$	λ	Rate Parameter	$\lambda_{RLC} = 0.2513$ $\lambda_{NRLC} = 0.2370$
Lognormal	$f(x) = \frac{1}{x\sigma\sqrt{2\pi}} e^{-\frac{(\ln x-\mu)^2}{2\sigma^2}}, x > 0$	μ and σ	Mean and standard deviation	$\mu_{RLC} = 3.98$ $\mu_{NRLC} = 4.22$ $\sigma_{RLC} = 2.32$ $\sigma_{NRLC} = 2.22$
Weibull	$f(x) = \frac{k}{\lambda}(\frac{x}{\lambda})^{k-1} e^{-(x/\lambda)^k}, x \geq 0$	$k \text{ and } \lambda$	Shape parameter and scale parameter	$k_{RLC} = 2$ $k_{NRLC} = 2$ $\lambda_{RLC} = 4.49$ $\lambda_{NRLC} = 4.76$
Erlang	$f(x) = \frac{x^{k-1} e^{-(x/\lambda)}}{\lambda^k (k-1)!}, x \geq 0$	$k \text{ and } \lambda$	Shape parameter and scale parameter	$k_{RLC} = 2$ $k_{NRLC} = 2$ $\lambda_{RLC} = 0.2513$ $\lambda_{NRLC} = 0.2370$

TABLE 4 Continuous Distribution Functions for the RLC and NRLC Intersections

RLC Intersections									
Time Group (sec)	Mid value (sec)	Observed Frequency	Relative Frequency	Continuous Distribution Functions					
				Normal	Exponential	Lognormal	Weibull	Erlang
0	0	20	0.066667	0.039403	0.251300	-	0.000000	0.000000
0-1	0.5	23	0.076667	0.055107	0.221627	0.045235	0.048989	0.098235
1-2	1.5	23	0.076667	0.094417	0.172379	0.034989	0.133087	0.178283
2-3	2.5	36	0.120000	0.135589	0.134075	0.028767	0.181895	0.179756
3-4	3.5	44	0.146667	0.163203	0.104282	0.024625	0.189105	0.152242
4-5	4.5	45	0.150000	0.164650	0.081109	0.021627	0.163500	0.118414
5-6	5.5	44	0.146667	0.139228	0.063086	0.019333	0.121691	0.087554
6-7	6.5	30	0.100000	0.098678	0.049068	0.017510	0.079308	0.062597
7-8	7.5	18	0.060000	0.058619	0.038164	0.016020	0.045697	0.043694
8-9	8.5	11	0.036667	0.029187	0.029684	0.014775	0.023420	0.029957
9-10	9.5	6	0.020000	0.012181	0.023088	0.013716	0.010719	0.020255
Total		300						
NRLC Intersections								
0	0	10	0.033333	0.035326	0.237000	-	0.000000	0.000000
0-1	0.5	17	0.056667	0.050063	0.210516	0.031011	0.043666	0.088643
1-2	1.5	25	0.083333	0.088077	0.166095	0.027356	0.119939	0.165543
2-3	2.5	38	0.126667	0.129878	0.131048	0.023740	0.167571	0.171753
3-4	3.5	46	0.153333	0.160525	0.103395	0.021007	0.180058	0.149684
4-5	4.5	51	0.170000	0.166294	0.081578	0.018888	0.162681	0.119802
5-6	5.5	44	0.146667	0.144391	0.064364	0.017191	0.127927	0.091151
6-7	6.5	35	0.116667	0.105083	0.050783	0.015797	0.089061	0.067059
7-8	7.5	21	0.070000	0.064100	0.040067	0.014627	0.055424	0.048167
8-9	8.5	9	0.030000	0.032772	0.031613	0.013628	0.031018	0.033982
9-10	9.5	4	0.013333	0.014044	0.024942	0.012764	0.015674	0.023643
Total		300						

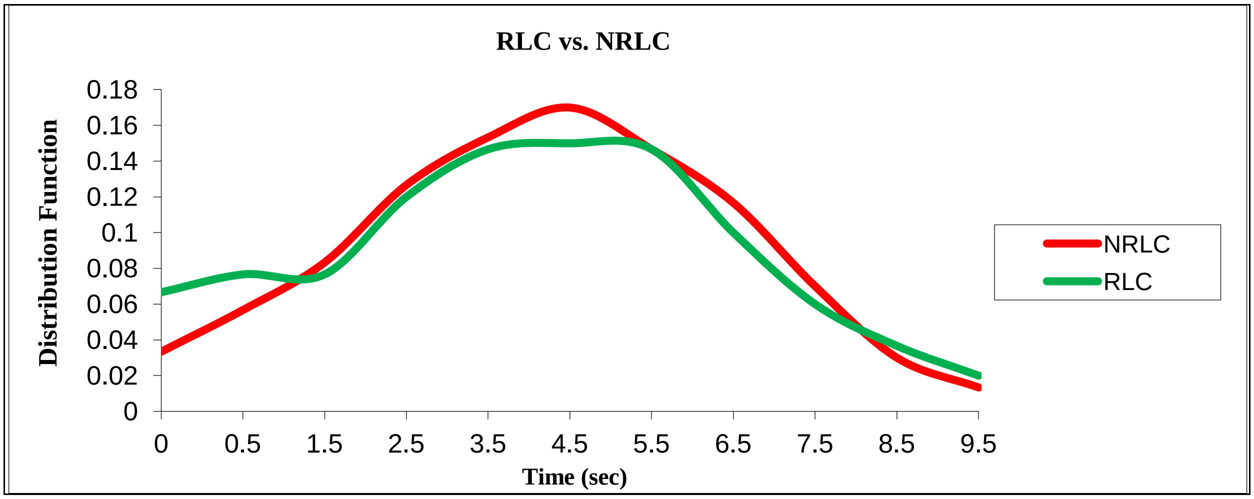

FIGURE 2 Actual Distribution Functions of RLC vs. NRLC Intersections

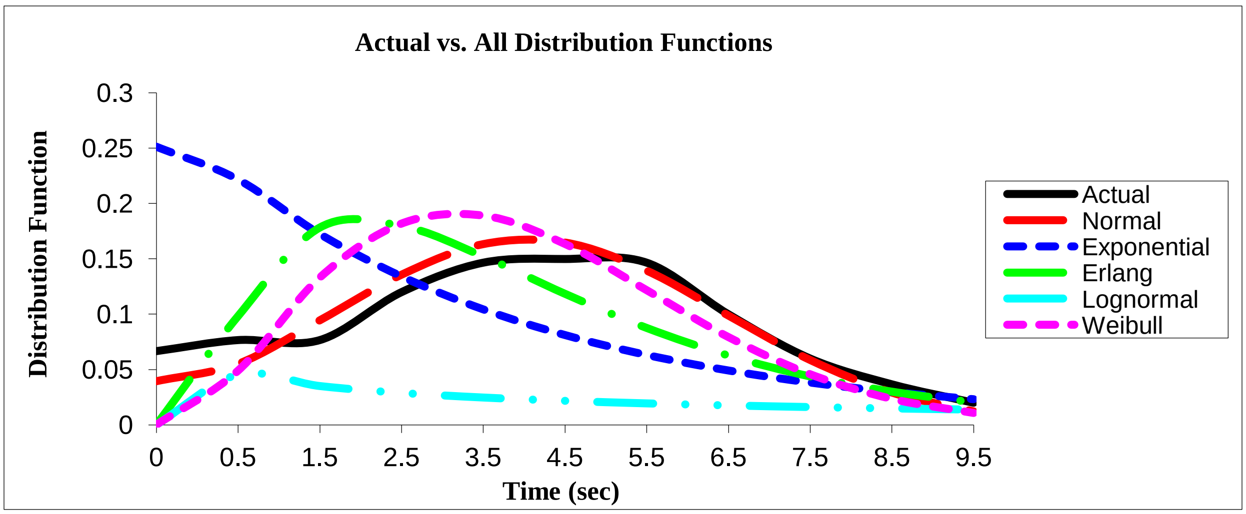

FIGURE 3 Actual vs. All Distribution Functions for RLC Intersections

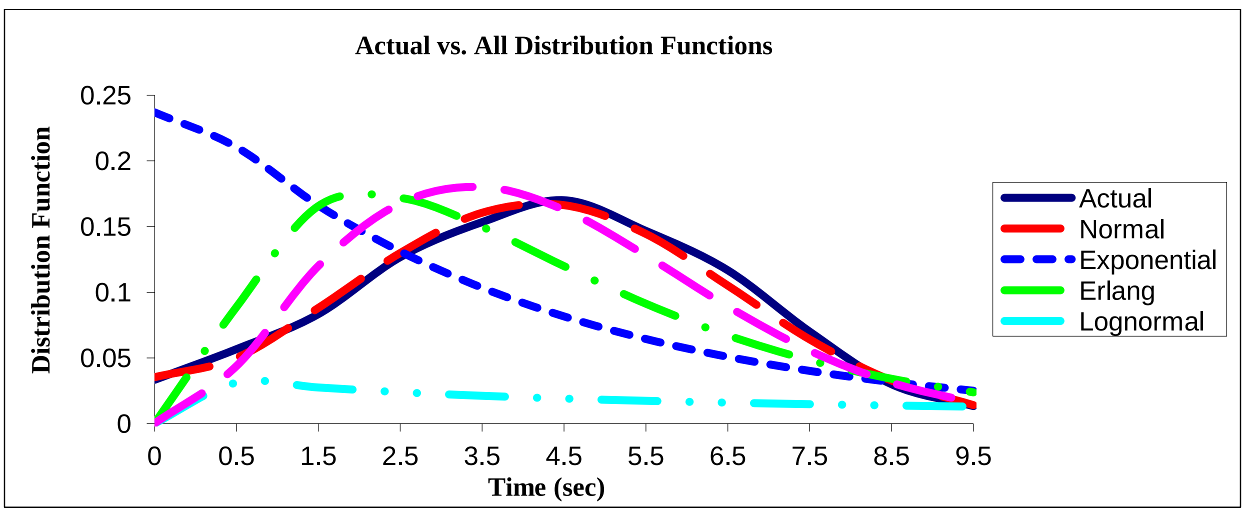

FIGURE 4 Actual vs. all Distribution Functions for NRLC Intersections

Probability of Stopping and Going

In the previous section we found the normal distribution to be the closest function to fit the travel time at the study intersections. Thus using the normal distribution function and a similar approach used by Sheffi and Mahmassani (*21*) at a high-speed signalized intersection the probability of stopping during yellow at the RLC and NRLC intersection pairs can be represented as a binary choice model given as:

$$P_{stop}(t) = \Phi\left(\frac{t - \mu_{(RLC,NRLC)}}{\sigma_{(RLC,NRLC)}}\right) \tag{1}$$

Similarly, the probability of going during yellow is given by:

$$P_{go}(t) = 1 - \Phi\left(\frac{t - \mu_{(RLC,NRLC)}}{\sigma_{(RLC,NRLC)}}\right) \tag{2}$$

where:

$P_{stop}(t)$ = probability of stopping during yellow;
$P_{go}(t)$ = probability of going during yellow;
$\Phi(\bullet)$ = standard cumulative normal function;
t = time to get to the stop line during yellow, sec;
$\mu_{(RLC,NRLC)}$ = respective mean time for RLC and NRLC intersections, sec; and
$\sigma_{(RLC,NRLC)}$ = respective standard deviation for RLC and NRLC intersections, sec

Table 5 shows the computation of the probability of stopping and going during the yellow time using the parameters for the normal distribution. To account for vehicles stopping on yellow only due to the presence of RLCs the difference between the probabilities of stopping at RLC and the very next NRLC intersection is shown under the RLC – NRLC column.

TABLE 5 Probability of Stopping and Going during Yellow

Time (sec)	Probability of Stopping			Probability of Going
	RLC	NRLC	RLC - NRLC	(1-(RLC –NRLC))
0	0.043125219	0.028657497	0.014467722	0.985532278
1	0.099486546	0.073466474	0.026020073	0.973979927
2	0.196705380	0.158655254	0.038050126	0.961949874
3	0.336361506	0.291314185	0.045047321	0.954952679
4	0.503439115	0.460529794	0.042909321	0.957090679
5	0.669906563	0.637337613	0.032568950	0.967431050
6	0.808038186	0.788666192	0.019371994	0.980628006
7	0.903494647	0.894761019	0.008733628	0.991266372
8	0.958430710	0.955688141	0.002742569	0.997257431
9	0.984759884	0.984346678	0.000413206	0.999586794
Average			2.3%	97.7%
Average for t= 0 to 4 seconds			**3.3%**	**96.7%**
$P_{stop}(t = 4\sec) = 4.3\%$ and $P_{go}(t = 4\sec) = 95.7\%$				

Figure 5 shows the probability of stopping comparison for the RLC and NRLC intersections. The graph shows that the probability of stopping at RLC intersections is higher than NRLC intersections. Thus, the higher number of vehicles stopping during the yellow interval at RLC intersections will affect the traffic flow at the intersection. The difference between the probabilities of stopping during yellow at RLC and NRLC intersections confirms that some drivers are deciding to stop at RLC intersections due to the presence of the cameras. Thus, in the next section the probability of stopping and going during yellow time will be used to estimate the capacity loss between RLC and NRLC intersections.

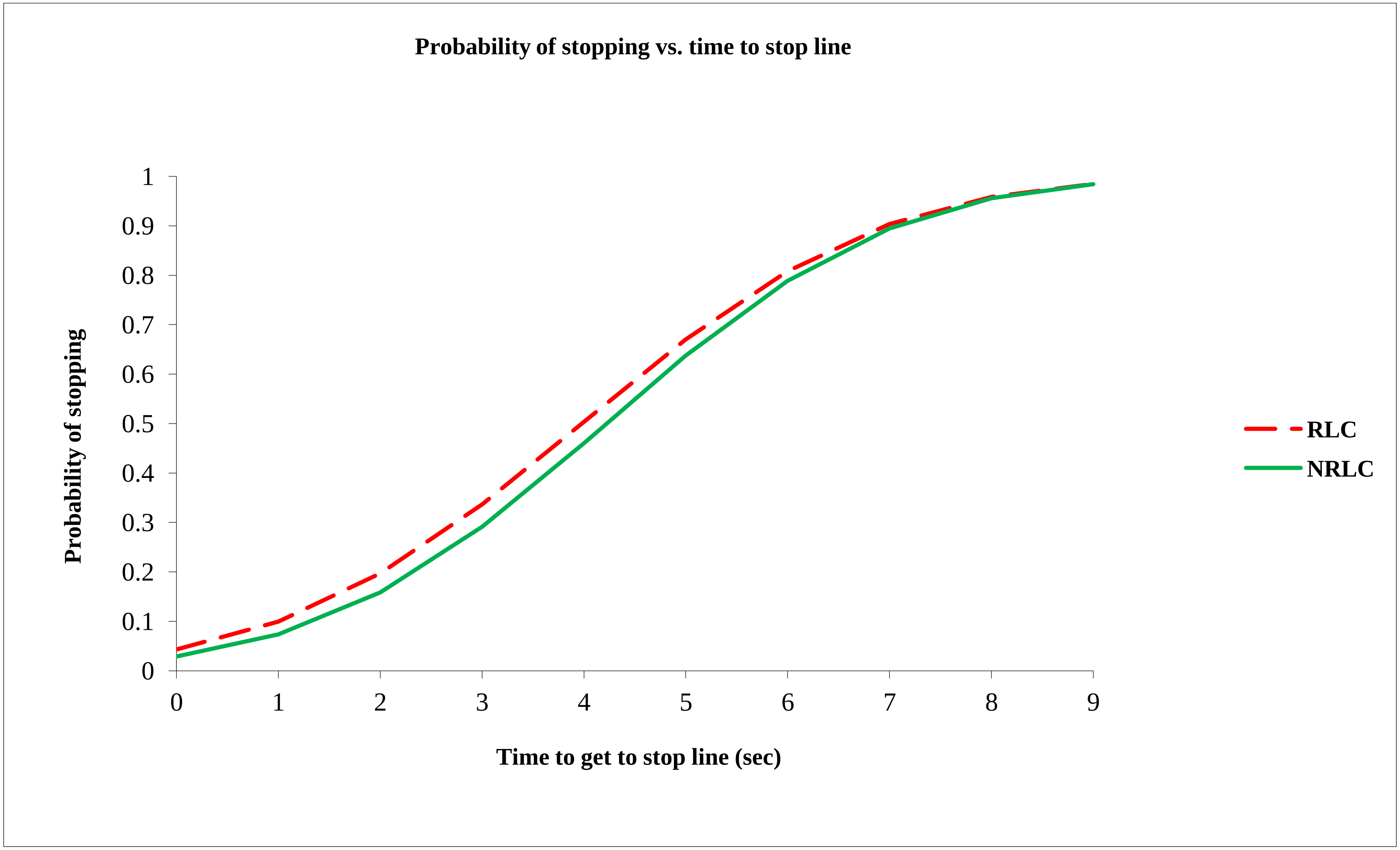

FIGURE 5 Probability of Stopping vs. Time to Get to Stop Line

CAPACITY ANALYSIS

Using the saturation flow rate computation in HCM (*28*), the saturation flow rate for the NRLC intersections is computed according to Equation (3).

$$S_{NRLC} = s_o \times N \times f_w \times f_{HV} \times f_g \times f_p \times f_{bb} \times f_a \times f_{LU} \times f_{LT} \times f_{RT} \times f_{Lpb} \times f_{Rpb} \quad (3)$$

The saturation flow rate at RLC intersections with the RLC reduction factor, f_{RLC} is given by:

$$S_{RLC} = S_{NRLC} \cdot f_{RLC} \quad (4)$$

To find the RLC reduction factor first we calculated the differences of the probabilities of stopping and going between the RLC and NRLC intersections for 0 to 9 second yellow time intervals (Table 5). Since the yellow time length at the study intersections is 4 seconds, the average probability of stopping for t = 0 to t= 4 seconds is used to find the average probability of going at the RLC intersections. This approach is used to account for vehicles stopping during yellow at RLC intersection for the yellow time duration (4 sec). Thus, the average RLC reduction factor (0.967) is the average probability of crossing during the yellow time duration (Table 5). Vehicles coming to the intersection can cross the intersection during yellow if there is enough clearing distance before the onset of the red signal. Thus, the vehicles crossing the intersection during yellow are also considered for the saturation flow computation. Using the average probability of stopping for the 4 seconds the RLC reduction factor can be given by:

$$f_{RLC} = 1 - P_{stop}(t) = P_{go}(t) \quad (5)$$

where:

S_{NRLC} = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group for NRLC intersection (veh/h);
S_{RLC} = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group for RLC intersection (veh/h);
s_o = base saturation flow rate per lane (pc/h/ln);
N = number of lanes in lane group;
f_w = adjustment factor for lane width;
f_{HV} = adjustment factor for heavy vehicles in traffic stream;
f_g = adjustment factor for approach grade;
f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group;
f_{bb} = adjustment factor for blocking effect of local buses that stop within intersection area;
f_a = adjustment factor for area type;
f_{LU} = adjustment factor for lane utilization;
f_{LT} = adjustment factor for left turns in lane group;

f_{RT} = adjustment factor for right turns in lane group;
f_{Lpb} = pedestrian adjustment factor for left-turn movements;
f_{Rpb} = pedestrian-bicycle adjustment factor for right-turn movements; and
f_{RLC} = RLC reduction factor for through lane groups

Estimating the Capacity Reduction

From the average RLC reduction factor (0.967) for the duration of the yellow interval computed above in Table 5 and saturation flow rates at the study intersections, the hourly loss in saturation flow rate, S_{loss} between NRLC and RLC intersection pairs is given by:

$$S_{loss} = S_{NRLC} - S_{RLC} \quad (6)$$

Table 6 shows the computed adjustment factors for the ten intersection pairs using the intersection geometric characteristics. The adjustment factors are then used to compute the saturation flow rates and hourly loss in saturation flow rate between the NRLC and RLC intersection pairs using equations 3 through 6.

TABLE 6 Saturation Flow Computations for NRLC and RLC Intersections

Intersection	S_O	N	f_w	f_{HV}	f_g	f_p	f_{bb}	f_a	f_{LU}	f_{LT}	f_{RT}	f_{Lpb}	f_{Rpb}	f_{RLC}	S_{NRLC}	S_{RLC}	S_{loss}
NRLC1	1900	4	0.933	0.998	1.015	1	1	1	0.893	0.95	0.97	1	1		5911		
RLC1	1900	4	0.933	0.998	1.005	1	1	1	0.893	0.95	0.85	1	1	0.967		5716	195
NRLC2	1900	4	0.933	0.985	1.000	1	1	1	0.893	0.95	1.00	1	1		5925		
RLC2	1900	4	0.933	0.985	1.000	1	1	1	0.893	0.95	1.00	1	1	0.967		5730	196
NRLC3	1900	4	0.933	0.985	0.985	1	1	1	0.893	0.95	0.97	1	1		5661		
RLC3	1900	4	0.933	0.985	0.985	1	1	1	0.893	0.95	0.97	1	1	0.967		5474	187
NRLC4	1900	2	0.933	0.985	1.015	1	1	1	0.962	1.00	0.97	1	1		3308		
RLC4	1900	2	0.933	0.985	1.015	1	1	1	0.962	1.00	0.97	1	1	0.967		3198	109
NRLC5	1900	4	0.933	0.985	1.015	1	1	1	0.893	0.95	0.85	1	1		5112		
RLC5	1900	4	0.933	0.985	1.015	1	1	1	0.893	0.95	0.85	1	1	0.967		4943	169
NRLC6	1900	4	0.933	0.985	1.005	1	1	1	0.893	0.95	1.00	1	1		5955		
RLC6	1900	4	0.933	0.985	1.005	1	1	1	0.893	0.95	0.85	1	1	0.967		5758	197
NRLC7	1900	4	0.933	0.985	1.000	1	1	1	0.893	0.95	0.97	1	1		5747		
RLC7	1900	4	0.933	0.985	1.000	1	1	1	0.893	0.95	0.97	1	1	0.967		5558	190
NRLC8	1900	6	0.967	0.985	1.000	1	1	0.9	0.833	0.95	1.00	1	1		7734		
RLC8	1900	6	0.967	0.985	1.000	1	1	0.9	0.833	1.00	0.85	1	1	0.967		7478	255
NRLC9	1900	3	0.933	0.985	1.000	1	1	1	0.893	0.95	0.85	1	1		3777		
RLC9	1900	3	0.933	0.985	1.000	1	1	1	0.893	0.95	0.85	1	1	0.967		3653	125
NRLC10	1900	4	0.933	0.985	1.000	1	1	1	0.893	0.95	0.85	1	1		5036		
RLC10	1900	4	0.933	0.985	1.000	1	1	1	0.893	0.95	0.85	1	1	0.967		4870	166
S_{loss} = Average hourly loss in saturation flow rate for the ten RLC intersections = 179 veh/h																	

CONCLUSIONS AND FUTURE WORK

The main objectives of this research are to investigate driver behavioral changes at RLC equipped intersections and the resulting intersection capacity reduction. Major changes in behavior include drivers not effectively using the yellow signal time, sudden stop during yellow and the reduced capacity at RLC monitored intersections. The research proved the hypothesis that if drivers are aware of the presence of the RLC, either as a frequent user of the intersection or from the posted signs, then some drivers are more likely to stop during yellow even when there is enough clearing distance. Further, the research investigated the effects of RLCs on driver behavior resulting into a possible increase in the probability of stopping during yellow, which can result in a reduction in intersection capacity.

The research findings confirm the hypothesis that the presence of RLCs influence drivers' stop and go decisions and its impact should be carefully examined in calculating intersection capacity. The hypothesis that at RLC intersections the probability of stopping during the yellow signal time is higher than NRLC intersections is proven. The numerical example suggests that there is an average hourly loss in saturation flow rate of 179 vehicles at the RLC intersections. This equates to a lost capacity of 90 vehicles per hour with an average g/c ratio of 0.5.

The capacity loss at RLC monitored intersections occurs when the affected phase is fully saturated. Moreover, a loss in capacity at the RLC intersections may also result in reduced lost time on the crossing streets. A reduction in saturation flow rate and a loss in capacity can be considered as negative utilities for using RLCs at signalized intersections.

The operational effectiveness of the intersections is affected due to the delayed vehicles which would have otherwise crossed the intersection during the yellow interval. At present the computation of the base saturation flow rate in the HCM does not consider the capacity loss caused by driver behavior changes due to the presence of a RLC.

Given the continuous monitoring of intersections by RLCs, the cumulative impact of capacity reduction may be huge. The capacity loss at RLC intersections can be considered significant given the fact that the number of RLCs used to monitor for RLR behavior are increasing nationwide. Moreover, RLCs have been used for more than a decade and in most cases they seem to continue to exist perpetually suggesting that there may not be any respite from lost capacity. Therefore, when deciding future installation of RLCs one must carefully consider the tradeoff between the safety benefits of using RLCs and the capacity reduction resulting due to the presence of RLCs. Additional factors to consider may include looking into the monetary values of accident data along with revenue collected at the RLC intersections to see whether the safety benefits outweigh costs to motorists.

The proposed research is expected to be a valuable tool for more precise calculation of signalized intersection capacity at RLC monitored intersections, to guide cities planning to use RLCs and inclusion of the RLC reduction factor in future versions of HCM. In the future we will perform additional field observation and data analysis to investigate a systemwide capacity reduction in a transportation network due to the presence of RLCs.

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