Determination of Optimum Green Interval for Intersections with Excessive Green Time

Onanong Sangphong[1], Siradol Siridhara[2]

[1]Department of Transportation Engineering, Suranaree University of Technology, E-mail: nongon333@hotmail.com
[2] Lecturer, School of Transportation Engineering, Institute of Engineering, Suranaree University of Technology, E-mail: siradols@sut.ac.th

Abstract – **Design traffic signal is too long green time resulting in inefficient flow of the intersection. Since the capacity of the intersection is not fully and increase a delay to the other direction. Traffic characteristic at intersections have green time too long. Traffic flow will be passed only during when the start green time that direction until there is no queue. There are not traffic flow through intersections although the situation in green time period. And the other direction at a red time is received it will be queuing. As a result the potential to motivate behavior driving through red time and the risk of accidents followed. That such a split would occur with not traffic flow through while the green time / Traffic demand less than the design traffic signal that is green time.**

The capacity loss can be obtained from the flow of traffic during the green period from a saturated approach. The method of integration, were recruited from the area function relationship of the graph. Will find that the capacity loss at the beginning less than the end of green phase, It is too long green time. This study to adjust optimum green time by considers design traffic signal using a balance between capacity loss at the beginning and the end of green phase. Change traffic light to appropriate by reducing the green green intervals. Change new cycle length is 83 second from the original traffic lights is cycle length 94 second ,result the morning to reduce the capacity loss 161 vehicles per hour and the evening will reduce the capacity loss 215 vehicles per hour. Optimum green time interval for intersections will be increase capacity at intersection.

Keywords – Effective Green Intervals / Lost Time / Saturation Flow Rate

1. INTRODUCTION

In Thailand, determination of cycle length and green interval during peak hours usually depends on a judgment of policemen. Most of them tend to use long cycle lengths to accommodate a large number of vehicles [8]. Long cycle lengths are advantageous as they reduce start-up lost time at the beginning of green. This is true especially when the traffic is combined among motorcycles, passenger cars and large trucks. Nonetheless, a main shortcoming of using too long green time will take place after the green interval is shown for a period of time in which gaps between cars gradually increase and traffic flow start to drop from a saturated value. Furthermore, long red intervals risk long queue and easily affect to nearby intersections.

Signalized intersection with the too long green time. The capacity loss due to the traffic less green time that designed. In this study. In this study select the Burapha Intersection in Nakhon Ratchasima province is a case study. Because of traveler complain “This intersection has green and red time too long very much”. Improve optimum time for intersections by decrease cycle length. Result to increase capacity of the intersection and reduce the risk of causing accidents.

Most research topics focus on actuated signal control. In our local case, however, Nakhon Ratchasima Municipality Area consists of grid-system roads on which traffic lights are fixed-time controlled. The design considers mainly peak-hour traffic volumes. Off-peak traffic patterns which are often neglected do not resemble those in peak intervals. This study focuses on off-peak traffic control where green time is excessively provided and intersection capacity is lost. This study finds an optimum green interval to more efficiently accommodate traffic at intersectional.

2. OBJECTIVES

The objectives of the study are envisaged as follows:

1. To analyze characteristics of lost time during signal phase change,
2. To study relationship between flow and gap to determine start-up and clearance lost time,
3. To study effects of green interval adjustment to capacity loss

3. STUDY SCOPE AND DATA COLLECTION

Video cameras were installed to collect data on road driving behavior and reaction to green intervals for all four sides (i.e., East to West – EW, West to East – WE, North to South – NS, and South to North – SN).

Ideal signalized intersections for the study should be located in the municipality area, and have no impedance after the traffic flows through the intersection such as parking and construction work. An Intersection near Burapha Community was selected for analysis as its traffic volume apparently cannot fill the whole green interval.

This study collected drivers' behavior in response to green time interval through video recording on each approach. Traffic data was collected in two intervals; morning and evening, one hour and thirty minutes each for three days (Tuesday, Wednesday and Thursday). The video camera was at a point where traffic lights and references can be seen clearly and all necessary data could be collected for analysis.

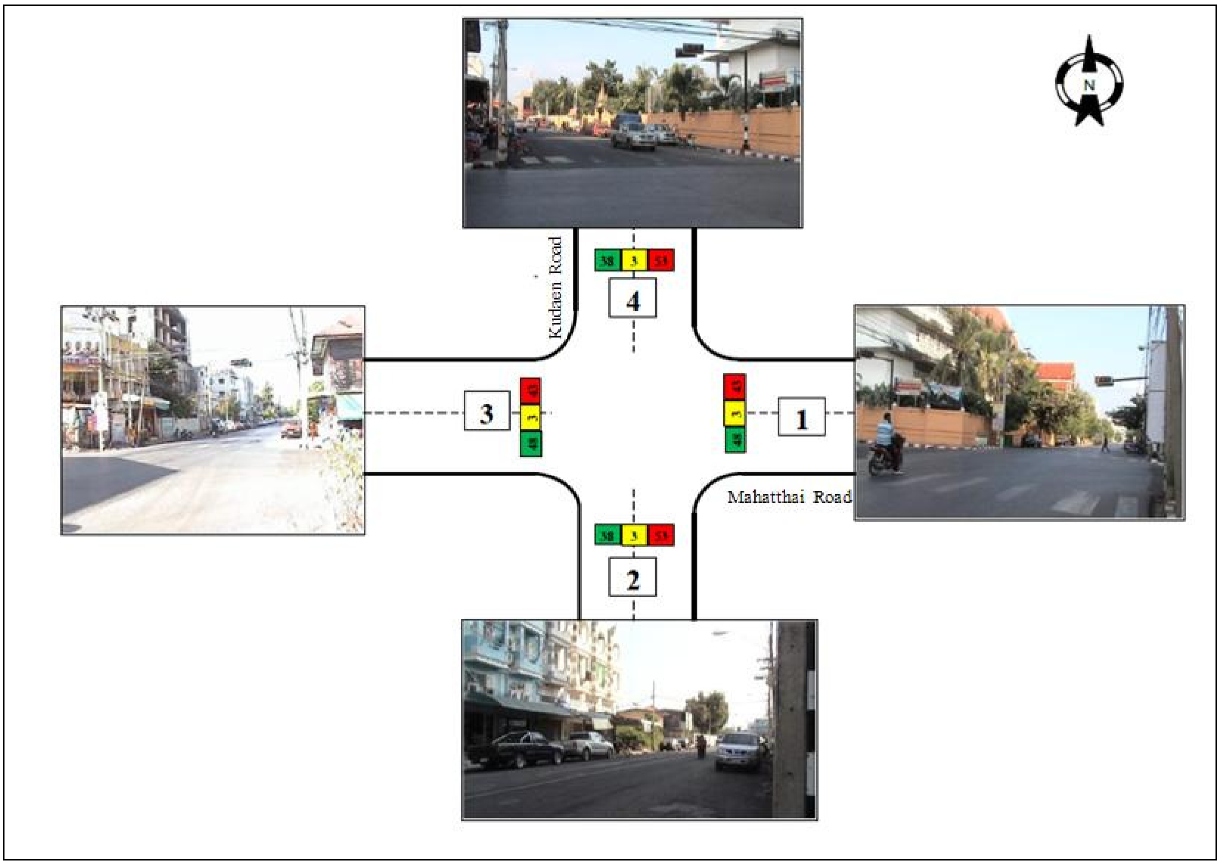

Figure 1 Physical Characteristics of Burapha Community Intersection

4. DATA ANALYSIS AND STUDY RESULT

4.1 Headway

Headways between vehicles in the queue were plotted to analyze drivers' behavior during green intervals.

The chart was constructed from average values of headways between two successive vehicles versus the vehicle order in the queue to illustrate unused green portion.

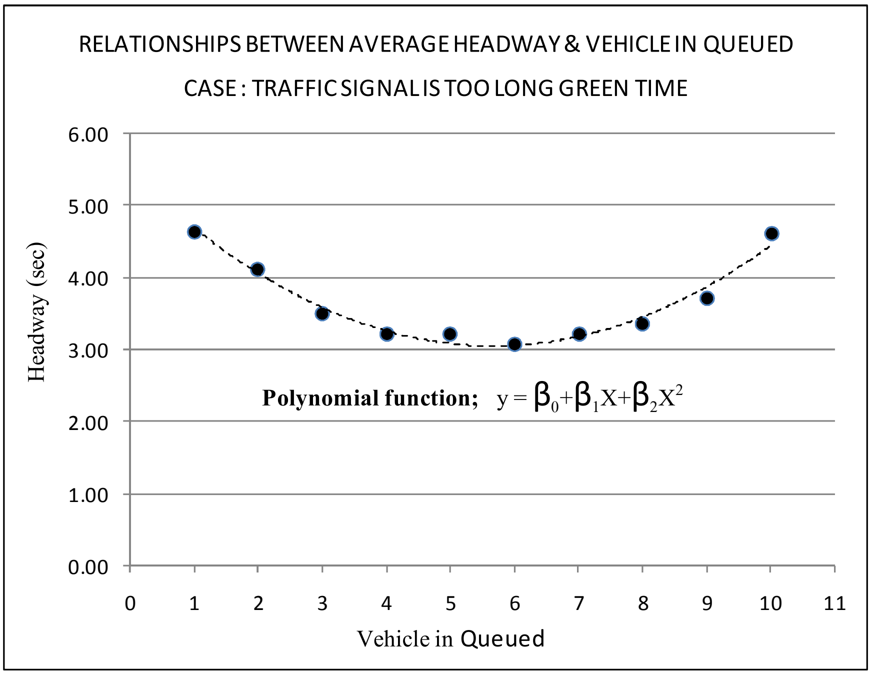

Figure 2 Relationships between Average Headway and Order of Queued Vehicle.

4.2 Saturation Headway

As the headway between vehicles in the field would not perfectly conform the theoretical saturation headway characteristics, it is essential to set criteria to determine whether the flow has reached a saturation point. A t-test, a proven tool for comparing means of two independent sample groups with unknown variances, was applied to confirm that two successive headways were equal and the saturation flow state had started. The test process consists of the following stages:

1) Set a hypothesis for a one-tailed t-test.

$$H_\circ : \mu_1 - \mu_2 = 0$$
$$H_1 : \mu_1 - \mu_2 > 0$$

2) As variances of the two group are not equal ($\sigma_1 \neq \sigma_2$), apply a "separated variance" t-test.

$$t' = \frac{\bar{X}_1 - \bar{X}_2}{\sqrt{\frac{S_1^2}{n_1} + \frac{S_2^2}{n_2}}}$$

$$c = \frac{S_1^2 / n_1}{S_1^2 / n_1 + S_2^2 / n_2}$$

Where $$df = \frac{(n_1 - 1)(n_2 - 1)}{(n_2 - 1)c^2 + (1 - c)^2 (n_1 - 1)}$$

3) Determine critical t where $\alpha = 0.05$.

4) Compare t' with critical t to find the test result neglecting the sign.

If t' > t, reject H_0, and
If t' < t, accept H_0

4.3 Saturation Flow Rates

A saturation flow rate is a maximum hourly flow rate that can be accommodated through an intersection by lane or lane group, assuming 100% green time. A saturation flow rate can be calculated from s = 3600/ h_s where h_s is average headway during a saturated flow state (second)

Table 1 Summary of Relationship between headway, saturation headway, saturation flow rate and lost time at Burapha Community Intersection

Direction	Fitted Equation	R Square	Saturation Headway	Order in Queue	Saturation Flow rate	lost time (s) /hr
E-W (M)	$7.884 - 2.276 * N + 0.251 * N^2$	0.796	3.14	4	1,146	277
S-N (M)	$5.625 - 1.119 * N + 0.122 * N^2$	0.813	3.22	4	1,118	151
W-E (M)	$6.923 - 1.924 * N + 0.225 * N^2$	0.839	3.19	4	1,129	245
N-S (M)	$5.574 - 1.238 * N + 0.143 * N^2$	0.889	3.15	4	1,143	265
E-W (E)	$6.256 - 1.513 * N + 0.168 * N^2$	0.928	3.27	4	1,101	206
S-N (E)	$4.909 - 0.796 * N + 0.083 * N^2$	0.746	3.20	4	1,125	119
W-E (E)	$6.608 - 1.376 * N + 0.126 * N^2$	0.763	3.24	4	1,111	131
N-S (E)	$5.131 - 0.951 * N + 0.103 * N^2$	0.840	3.07	4	1,173	195

4.4 Start-up and Clearance Lost Times

Capacity loss can be determined by integrating flow rate-time function.

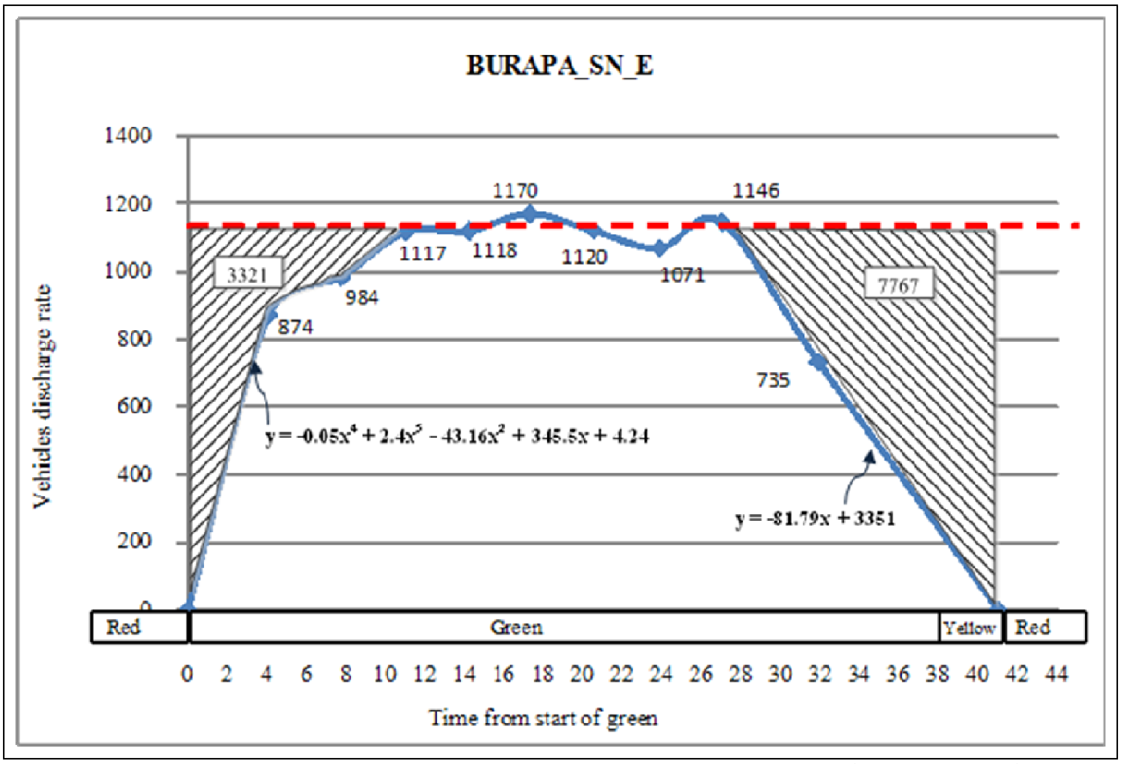

Figure 3 Example Relationship between Traffic Flow Rate and Capacity Loss (Area cast a shadow) by Direction S-N at Burapha Intersection

Table 2 Summary of Relationship between Traffic Flow Rate and Capacity Loss by Direction at Burapha Community Intersection

Direction	Start stage function	End Stage Function	Capacity Loss	
			start-up	Clearance
E-W (M)	$y = 0.011x^4 - 0.51x^3 + 3.78x^2 + 93.47x + 3.26$	y = -44.73x + 2223	7312	16019
S-N (M)	y = 0.002x5 - 0.164x4 + 4.437x3 - 55.93x2 + 353.6x + 0.757	y = -61.46x + 2542	5083	9667
W-E (M)	y = -0.042x4 + 2.020x3 - 34.02x2 + 272.3x + 2.093	y = -38.99x + 1941	5347	17658
N-S (M)	$y = -5E\text{-}05x^6 + 0.006x^5 - 0.33x^4 + 7.25x^3 - 78.71x^2 + 430x + 0.23$	y = -62.44x + 2627	6110	9266
E-W (E)	$y = -0.003x^4 + 0.33x^3 - 12.62x^2 + 201.7x + 1.603$	y = -38.68x + 1960	4393	15971
S-N (E)	$y = -0.05x^4 + 2.4x^3 - 43.16x^2 + 345.5x + 4.24$	y = -81.79x + 3351	3321	7767
W-E (E)	$y = -0.003x^4 + 0.31x^3 - 10.5x^2 + 169.1x + 3.94$	y = -54.29x + 2748	5905	11705
N-S (E)	$y = 0.001x^5 - 0.12x^4 + 3.70x^3 - 54.13x^2 + 378.1x + 2.571$	y = -67.29x + 2794	6833	9620

Explained variable; M = Morning, E = Evening, N = Order of Queued Vehicle, x = Duration of green time and Y = Capacity Loss

In case of excessively long green time, capacity lost due to start-up time would be less than that lost in slowing down and stopping.

4.5 Determination of Optimum Green Time

An intersection with excessive green time would lose capacity as traffic demand was lower than an amount that could be accommodated by programmed green time. At the end of green capacity would be lost due to no demand. Long red time for all other approaches also generated unnecessary delay for waiting vehicles. Signal timing should therefore be adjusted by reducing cycle length which would in turn reduce capacity loss.

This study adjusted cycle length attempting to balance capacity loss due to start-up and capacity loss due to clearance. AS shown in Figure 3 description of variables.

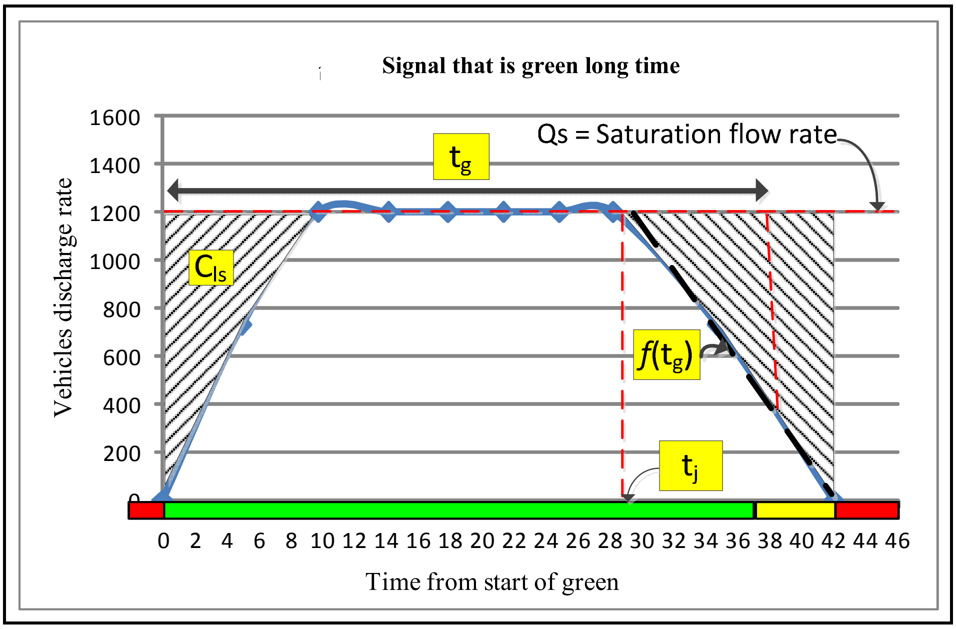

Figure 3 description of variables show on the flow of traffic during the green period from a saturated approach.

Optimum green time for intersections with excessive long green interval can be determined from:

$$C_{ls} = \left(Q_s\left(t_g - t_j\right)\right) - \int_{t_j}^{t_g} f\left(t_g\right)$$

Where C_{ls} = capacity loss due to start-up,
t_j = time where saturation flow ends,
Q_s = saturation flow rate,
q_d = reduction in capacity loss after cutting down green time (veh/hr),
$f(t_g)$ = linear relationship between traffic flow and time during green
t_g = optimum interval of green plus yellow that balances capacity loss during start-up and clearance stages.

Optimum green times at Burapha Community Intersection is shown in Table 3

Table 3 Reduction in Capacity Loss after Using Optimum Green Time

Direction	C_{ls}	t_j	Q_s	$f(t_g)$	t_g	q_d
E-W M)	7312	27	1146	y = -44.73x + 2223	42.4	8707
S-N (M)	5083	25	1118	y = -61.46x + 2542	36.2	4584
W-E(M)	5347	23	1129	y = -38.99x + 1941	37.5	12311
N-S (M)	6110	24.2	1143	y = -62.44x + 2627	37.8	3156
E-W(M)	4393	24	1101	y = -38.68x + 1960	37.4	11578
S-N (M)	3321	27.5	1125	y = -81.79x + 3351	36.2	4446
W-E(M)	5905	32	1111	y = -54.29x + 2748	30.8	5800
N-S (M)	6833	24.3	1173	y = -67.29x + 2794	38.3	2786

Find to optimum interval of green plus yellow (t_g) in each direction. By solving equations using Variables involved data as shown in Table 3. The answer the selected optimum time must to consistent with all directions.

Table 4 Summary of adjust optimum time from solving the equation data.

Direction	By solving the equation		Adjust optimum time			
	t_g (Morning)	t_g (Evening)	Green time	yellow time	Red time	Cycle Length (sec)
E-W	42.4	37.4	40	3	40	83
S-N	36.2	36.2	35	3	45	83
W-E	37.5	30.8	40	3	40	83
N-S	37.8	38.3	35	3	45	83

Determining optimum green time by reducing unused green portion would increase intersection capacity.

However, one must consider existence of traffic demand at other approaches. As for this Burapha Community Intersection, optimum green times were determined as shown in Tables 5 and 6.

Table 5 Optimum Green Times for Burapha Community Intersection, Morning Period

Direction	Existing			Adjusted				
	Green	Cycle Length	Capacity Loss (veh/hr)	Green	Cycle Length	Capacity Loss Start	Capacity Loss End	Capacity Loss (veh/hr)
E-W	48	94	813	40	83	7312	7817	652
S-N	38	94		35	83	5083	6656	
W-E	48	94		40	83	5347	9493	
N-S	38	94		35	83	6110	6319	

Table 6 Optimum Green Times for Burapha Community Intersection, Evening Period

Direction	Existing			Adjusted				
	Green	Cycle Length	Capacity Loss (veh/hr)	Green	Cycle Length	Capacity Loss Start	Capacity Loss End	Capacity Loss (veh/hr)
E-W	48	94		40	83	4393	8299	
S-N	38	94	697	35	83	3321	4753	476
W-E	48	94		40	83	5905	4388	
N-S	38	94		35	83	7173	6509	

After signal timing improvement in which cycle length would be decreased to 83 seconds it is expected that the capacity would increase by 161 and 218 vehicles per hour in the morning and the evening respectively.

5. CONCLUSION AND RECOMMENDATION

The study of Burapha Community Intersection with excessively long green time could be concluded as follows:

Headway between vehicles in the queue became constant from the fourth vehicles on. The saturated headways ranged from 3.07 – 3.27 seconds. Start-up lost time 937 seconds per hour in the morning and 650 seconds per hour in the evening.

In the morning Capacity loss due to start-up is 254 vehicles per hour and due to clearance is 560 vehicles per hour. In the evening Capacity loss due to start-up is 218 vehicles per hour and due to clearance is 479 vehicles per hour.

Green time and cycle length at Barapha Community Intersection was reduced accordingly. The cycle length was reduced from 94 to 83 seconds. North-south green phase is 35 seconds long and east west green phase 40 seconds long, with 3 seconds yellow plus 2 seconds all red time.Capacities gained were 161 vehicles per hour in the morning and 221 vehicles per hour in the evening.

The recommendations were:

1) Green time adjustment, increase or decrease, must consider traffic demand on all approaches so as to achieve best green efficiency.

2) Fixed-time signal in peak and off-peak period should apply different cycle length and timing. The design should be consistent with traffic demand statistics so that best capacity can be achieved.

6. Periodicals

[1] Han, Lee David, P.(2007). Short or Long—Which Is Better? Probabilistic Approach to Cycle Length Optimization, Traffic Signal Systems and Regional Transportation Systems Management.

[2] C. J. Bester and W.L. Meyers, P.(2007). SATURATION FLOW RATES, the 26th Southern African Transport Conference.

[3] Karan Khosla, James C. Williams, P.(2006). Saturation Flow at Signalized Intersections during Longer Green Time, Journal of the Transportation Research Board, Volume 1978 , Transportation Research Board of the National Academies, pp.61-67

[4] Richard W. Denney, Jr., Eddie Curtis, Larry Head, P.(2009). Long Green Times and Cycles at Congested Traffic Signals, Journal of the Transportation Research Board, Volume 2128, Transportation Research Board of the National Academies,pp.1-10

[5] Thirayoot Limanond, Suebpong Chookerd, and Natcha Roubtonglang (2009). Effects of countdown timers on queue discharge characteristics of through movement at a signalized intersection. Transportation Research Part C. 17(6): 662-671.

[6] Kim, J-T, Courage, K G, P.(2003). Evaluation And Design Of Maximum Green Time Settings For Traffic-Actuated Control, Transportation Research Record No. 1852, p. 246-255.

[7] Richard W. Denney Jr, Curtis, Eddie, Head, Larry Long Green Times and Cycles at Congested Traffic Signals, Journal of the Transportation Research Board , Report Numbers: 2009-2273

[8] Sanguanwong. C. Traffic light Traffic flow turbulent, January 5, 2011. STOCK NEWS ONLINE. Article on website http://www.kaohoon.com