Travel Time Estimation on Arterial Streets

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Abstract

Micro-simulation models are widely and successfully used to estimate travel time on arterial streets. But micro-simulation of a large-scale traffic network is difficult to conduct because of the following three reasons: 1) limited computer power, 2) complicated network input data and 3) long model building, debugging, and validating time. In contrast, link performance functions such as the BPR function needs less detail input data. Input data of BPR functions like volume and link capacity are easy to obtain. Therefore, BPR functions are still widely used to estimate travel time on arterial streets for planning purposes. However, it is also well known that BPR functions generate erroneous estimation of travel times on arterial streets especially under congested traffic conditions. This paper introduces a new algorithm for travel time estimation on arterial streets. Vehicles are grouped into packets/vehicle observed group in the algorithm. Link travel time and intersection control delay of each packet/ vehicle observed group are estimated by the developed algorithm. The developed algorithm is a modified HCM2000 method which requires less network input data and computer power compared with micro-simulation. It provides better travel time estimation during short time interval updates on signalized arterial streets compared with BPR functions. This algorithm has been developed and has been validated by using actual data collected in the field as well as the developed algorithms in HCM2000. In other words, this algorithm can overcome the weaknesses of Micro-simulation and BPR functions. It can be an alternative for large-scale traffic network application.

1. Introduction

The estimation of travel time on arterial streets provides important applications to Advanced Travel Information Systems (ATMS) and Mesoscopic-simulation in transportation planning process. Link travel time and Intersection Control Delay are important parameters of travel time that are frequently used to evaluate performance of intersection and network. Reliable travel time estimation can also provide reliable dynamic traffic assignment results in transportation planning process. In the study, detectors exist about 500feet away from the stop line which provides a reliable detected link travel speed. Once the intersection control delay is estimated, travel time on an arterial street is derived from the link travel time plus the intersection control delay in each short time interval update.

2. Literature Review

Statistical methods, dynamic flow theory and AVI technology have been successfully used to estimate travel time on freeways. These methods are also applied to estimate travel time and intersection control delay on arterials.

Statistical methods have traditionally utilized linear or non-linear models to predict travel time on freeways or arterial streets based on historic traffic data. H. M. Zhang(1998)[1] used the historic critical v/c ratio, and the occupancy from the loop detector to build up a non-linear model named “the journey Speed Model” to predict travel time on arterial streets. The journey speed is represented as the weighted sum of the historic speed and the current speed from the detector. Neural networks are built by utilizing previous existing data. However, neural networks can perform better than the normal statistical methods in mapping the relationships between travel time and input

data. In the past several years, neural networks have been successfully applied to predict short-term traffic flow and travel time. Abdulhai, Porwal and Recker(1999) [2] used an advanced Time Delay Neural Network (TDNN) model combined with Genetic Algorithm (GA) to predict traffic flow and density which can be used to estimate travel time. Ishak, S., and C. Alecsandru (2003) [3] used multiple topologies of dynamic neural network to optimize short-term travel time estimation and prediction. They also tested and compared four different neural network architectures under different settings and traffic conditions.

Dynamic flow methods are widely used to compute travel time particularly the average delay of vehicles caused by the red time at a signalized intersection. The well-known Webster formula [4] introduces how to calculate average signal delay near the stop line of an intersection. Later, Highway Capacity Manual 2000 [5] introduces intersection control delay caused by a signalized intersection. Intersection control delay includes uniform control delay, incremental delay caused by random arrivals and oversaturation queues and initial queue delay. Tsekeris and Skabardonis (2004) [6] compared the performance of the method in HCM2000 with the spot speed model (SSM) and BPR method on arterial networks. In their study, the stated methods in HCM2000 demonstrated the most promising modeling approaches. Fadhely, Kenneth, and Donald (2002)[7] introduced and compared queue methods from SIDRA, HCM2000,TRANSYT-7F, SOAP, NCHRP 279 Guidelines, SIGNAL 97,NETSIM and Oppenlander's Method. In their study, the SIDRA and HCM2000 method provide better accuracy than other methods since most of other methods will forgive the residual queue built up by the previous cycle length and some of them only report average value of the queue. Their research shows that it is important to consider the queue built up by the pervious time intervals. The HCM2000 approach can estimate the average intersection delay in a relatively long time interval. However, it is not suitable for short term dynamic traffic applications especially under congested traffic conditions. Henry and Wenteng(2006)[8] explored a travel time estimation model for signalized arterial network. A dummy vehicle was traced to calculate the possibility it hits the red light as it approaches every intersection. Alexander and Nikolaos[9] estimated real-time travel time by separating the total delay into the delay of a single vehicle, the delay because of queues and the oversaturated delay. However, these methods are difficult to implement in a large arterial network in real-time, since it traces and computes the delay of each vehicle.

The dynamic flow method requests reliable detected traffic information from the traffic detector. Ashish Sen(1996) [10] addressed the shortcomings by analyzing the best locations of loop detectors at a signalized intersection to estimate travel time. Ruey, Qun and Der-Horng(2002)[11] used the upstream link stop line detector and platoon dispersion model to predict the travel time on the observed link. Sungho Oh, Bin Ran and Keechoo Choi (2003)[12] performed a study to find the best location of a detector to estimate travel time on a relatively long urban link. These detector location studies provided us with guidelines to locate and use detectors in order to get accurate traffic information in our study.

Besides the Statistical methods and dynamic flow theory, a new category of arterial travel time estimation called AVI method is based on the idea of tracking the same vehicle traveling along arterial streets by GPS or by the vehicle identification technology[13]. Similarly, vehicle platoon is identified and tracked in order to estimate travel time by Lucas and Verma(2004)[14]. 'ITS Orange Book'[15] (2004) introduced the

application of several technologies to estimate and predict travel times in networks. "ITS Orange Book" shows several cutting-edge AVI technologies for estimation of travel time. Introduced in the "ITS Orange Book", Vehicle Relayed Dynamic Information (VERDI) uses the GSM network to communicate the probe vehicles with the control centers and determines their positions via GPS. It successfully combines the GPS and GSM technology to get real time traffic information. The "ITS Orange Book" discussed the research done at the University of Southampton Transportation Research (United Kingdom) which shows that getting traffic information from a higher proportion of vehicles with GPS tracking system may not lead to an increase in estimated accuracy of travel time. Information from 26 percent of vehicles provides perfect performance. Their research provides a good guideline to select observed proportion of vehicles when using probe to estimate travel time. Choi and Chung(2001)[16] fused travel time detected by GPS and loop detectors to enhance accuracy of the estimation procedure. The fuzzy regression and Bayesian pooling methods are utilized in the abovementioned fusion procedure. Even though these advanced approaches have very high accurate travel time estimation, they are too expensive to apply in an urban-wide system. In addition, these approaches are difficult to be used to estimate real-time travel time due to sample size issues faced by these systems.

Travel time estimation with short time interval update should provide a better accuracy of estimation process. Statistical methods and AVI technology generally require a large sample size for estimation. Therefore, it is not recommended to be used to estimate travel time in real time especially in a short time interval update and congested situation. On the other hand, traffic flow theory including HCM2000 has been the recommended method to estimate intersection control delay and travel time. However, it has a shortcoming in estimating delay in a relatively short-time interval update especially in congested conditions as discussed earlier. This paper plans to modify the algorithms in HCM2000 to develop new algorithms for travel time estimation on arterial streets in real time, with the assumption that a detector exists in the middle of an arterial link.

3 Developed Algorithms

The developed methodology is presented into two sections. The first section introduces the algorithms that would estimate travel time on an isolated arterial street. The second section introduces the algorithms that would estimate travel time on a signalized arterial link that also considers traffic situation on the upstream and downstream links. However, this paper only discusses the first section which has been validated by the real-world data due to the paper size limitation.

Travel Time Estimation on an Isolated Arterial Street

Travel time on an isolated signalized arterial link includes two components: i) link travel time, ii) intersection control delay. Link travel time is computed based on the detected speed from detectors and the length of the street. Since the methodology assumes detectors are placed far from the stop line, the detected link travel speeds are reliable. Therefore, no special methodology is developed to estimate the link travel time. The link travel time is computed as the length of the street divided by the detected speed. The estimation of intersection control delay is more complicated and a dynamic flow algorithm is developed in order to estimate it. In the algorithm, vehicles are defined into

vehicle groups. Then the intersection control delay of the observed vehicle group is estimated to represent the intersection control delay for each time interval. The observed vehicle group is identified as the group of vehicles detected by the detector during an observed time interval. The updated observed time interval is considered to be equal to the signal cycle length of the intersection.

An intersection at N Franklin St/Peppers Ferry RD in Christiansburg, Virginia was selected to initially conduct control delay analyses based on traffic volume and the arrival of vehicles in the observed group. The observations at the intersection show that the average control delay of the observed vehicle group has a strong relationship with the traffic volume and with the arrival time of the first vehicle of the observed vehicle group as shown in Figures 1 and 2 respectively. Therefore, the developed algorithms will consider the delay of the first vehicle as well as the traffic volume in estimating the intersection control delay.

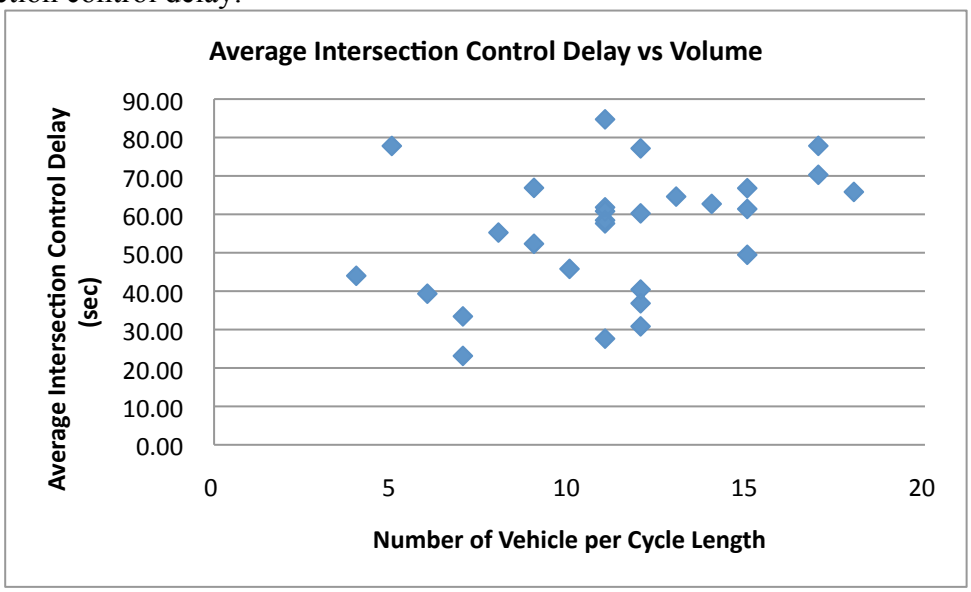

Figure 1 – Average Intersection Control Delay vs Volume

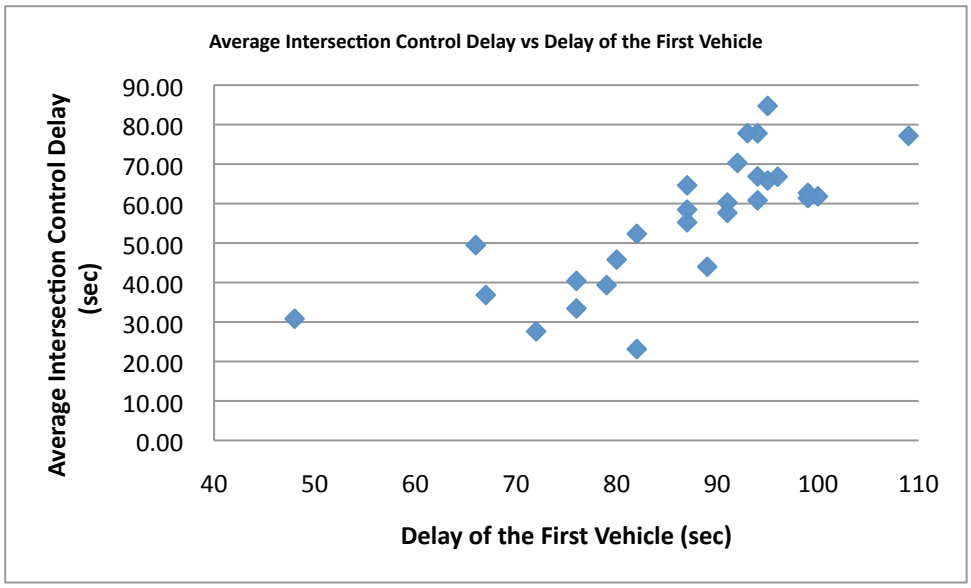

Figure 2 - Average Intersection Control Delay vs Delay of the First Vehicle in the observed group

The stopped delay at a signalized intersection varies with the size of the initial queue, the percentage of vehicles arriving during the red time, the time the first vehicle in the observed group arrives at the red time, and the volume of the observed group. According to HCM2000 algorithms, when the incoming volume is greater than the intersection capacity, uniform delay and over-saturation delay are computed independently to estimate the intersection control delay of the observed group. However, this separation is not needed to estimate the average stopped delay of the observed group behind the intersection, particularly if the 'queue vs time' curve is utilized. Therefore, in this algorithm, the computation of over-saturation and uniform delay is not separated.

The following relationships and assumptions are adopted:

a) The control delay is the average control delay of all the vehicles in the observed group.

b) The first vehicle in the first observed group will arrive at the intersection at the start of the red time phase

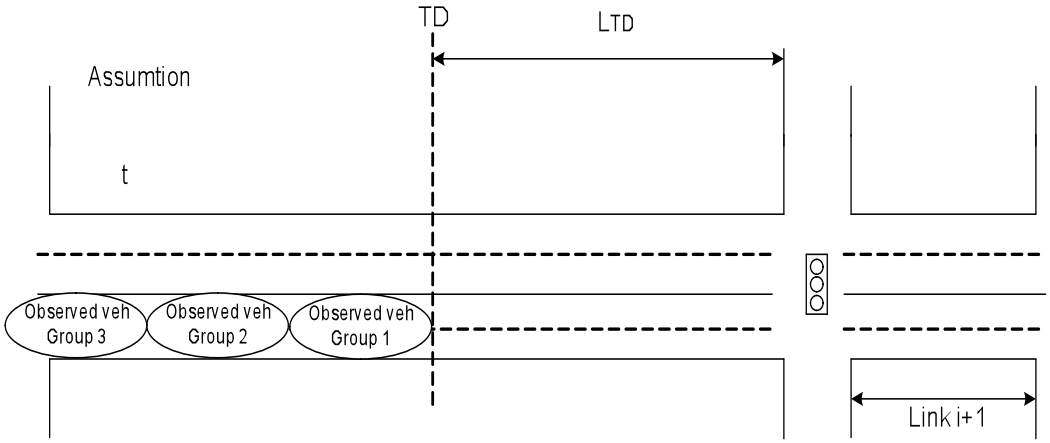

Figure 3 - Observed Vehicle Group

c) The observed group will use a full cycle length to pass the intersection irrespective of what time the first vehicle in the group arrives at the intersection. This is consistent with the approach that we are trying to determine the travel time it takes for this observed group to traverse the link. This assumption is used in calculating the uniform delay for this observed group.

d) If the number of vehicles in the last observed group (for example Group 1 in Figure 3) is greater than the capacity of the intersection, some vehicles are queued at the intersection till the next green time takes place. Based on the aforementioned items c and d, the first vehicle of next observed group (for example Group 2 in Figure 3) will also arrive at the end of the queue at the start of the red time phase.

Before computing the intersection control delay, the initial delay of the observed vehicle group is needed to be determined first. The initial delay of the first vehicle in the observed group (d3) is an important value in the developed algorithm. Different from the one computed in HCM2000,the initial delay of the first vehicle in the observed group in our case, as shown in Figure 4, is the time it takes the first vehicle in the observed group to travel from the time it arrives at the end of the initial queue to the time it arrives at the intersection. Our method computes the clearing time for the initial queue and not the average delay per vehicle in the queue.

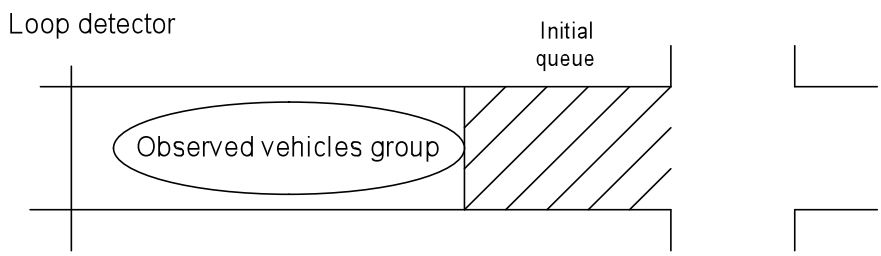

Figure 4 - Initial Delay d3

The adopted approach uses the shockwave method to estimate the initial delay of the first vehicle in the observed vehicle group and the initial queue. In the study, the detector is assumed to be located far from the stop line. If there is no blackout (queue reaches on the detector) taking place on the loop detector which means that the traffic data observed by the loop detector is reliable, the shockwave method is used to estimate the initial queue length and then estimate the initial delay.

There are three cases for estimating the initial delay: a building queue case, a dissipation queue case and no change of queue case.

Step 1:

$W_u = \dfrac{V - C}{k_{td} - k_q}$, for the building queue case where V>C; [1]

$W_d = \dfrac{C - V}{k_q - k_{td}}$, for the dissipation case where V<C; [2]

$W = 0$, for no change in queue case where V=C; [3]

Where
V : Incoming Volume;
C : Intersection Capacity;
k_{td} : is the vehicular density obtained from the detector;
k_q : is the jam density.

The queuing rate QR (veh/h) is computed as:

$$\mathrm{QR} = \frac{\mathrm{dn}}{\mathrm{dt}} = \left(V - C - \mathrm{W_u} \times \mathrm{k}_{\mathrm{d}t}\right); \qquad [4]$$

Where
n is number of vehicle in the queue.

Step 2:

The number of vehicles in queue which is built during the observed cycle length t will be:

$$Q_m = QR \times CL \qquad [5]$$

Where
CL : is cycle length;

The process is repeated for every interval and the total number of vehicles in queue is estimated using the following expression:

$$Q_t = \max(0, \sum_{m=1}^{t} Q_m) \qquad [6]$$

Q_t is the initial queue for next cycle length which is also the queue at the end of cycle t. Thus, if we want to estimate the initial delay for cycle length t, we should consider the queue at the end of last cycle length which is q_{t-1}.

If $\frac{q_{t-1}}{D_s} < g$, the initial delay is computed as $\frac{q_{t-1}}{D_s}$ +red_time

Where D_s is the departure saturation flow in green time veh/h/ln, g is the length of the green time(sec),

Else

If $\frac{q_{t-1}}{D_s} < 2g$, the initial delay is computed as $\frac{q_{t-1}}{D_s} + 2 \times red\_time$;Because it should wait for another green time to dissipate the initial queue vehicle for the incoming group of vehicles in time t.b

Therefore, we came out with the following equation to estimate the initial delay.

When $(n-1) \times g \le \frac{q_{t-1}}{D_s} \le n \times g$, where n is an integer number

$$\mathrm{d3} = \frac{q_{t-1}}{D_s} + (n) \times red\_time \qquad [7]$$

After computing the initial delay for the previous time intervals, the 'queue vs time' curve can be determined to compute the intersection control delay. We have three cases that describe variations in queue size and volume in determining the stopped delay which is regarded as intersection control delay. They are:

Case 1- where there is no initial queue for the observed group.
Case 2 – where initial delay (d3) is smaller than a Cycle Length
Case 3- where initial delay (d3) is greater than a Cycle Length

Case 1- There is no initial queue for the observed vehicle group.

Figure 5 shows the development of the vehicles queue size of the observed vehicle group over time at a signalized intersection. The total stopped delay of the observed group is made up of three components: 1) the stopped delay during the red time, 2) the queue delay during the green time, and 3) the over-saturation delay which occurs in the next cycle length.

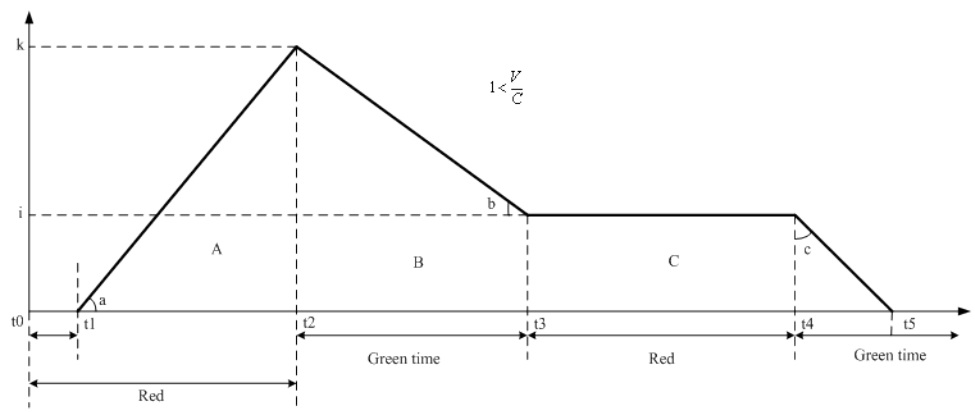

Figure 5 - Case 1- no initial queue for the observed group

As shown in Figure 5, time t0 to t3 is the observed cycle length. Time t0 to t2 is the red phase and the time from t2 to t3 is green phase. The average stopped delay of the vehicles detected within this cycle length represents the intersection control delay of this cycle length. The first vehicle in the observed vehicle will arrive at the intersection at time t1 which can be detected by the detector. After time t1, the queue will grow during the red phase to include the incoming vehicles from the observed group. The arrival rate is computed as the number of vehicle arrived during the red time divided by the remaining red time which is t2-t1. At the end of the red time, the queue is built to k. After time t2, the queue will decrease till the end of the green phase (t3) and now reaches the level of i. If volume is greater than the capacity, the value of i is greater than zero, otherwise, it is equal to zero. The queue of the intersection will start to build up again during the red phase. However, we need to separate the status of the vehicles in the observed group in the current time interval to the status of what is happening at the intersection as a whole. In that regard, we separated the queue development into a) the queue of the observed group and b) the queue at the intersection. In the previous definition, the observed vehicle group is the group of incoming vehicles during an observed cycle length(t0-t3). Therefore, the queue of the observed group, which is our main interest as shown in Figure 5, remains constant during the red phase, and then dissipates again during the next green phase using a time (t4-t5).

The angle "a" in Figure 5 represents the vehicle arrival rate from time t1 to t2. It can be computed as

$$\square\square 2-\square 1 = \square\times\square \quad \square 2-\square 1 = \square\times\square \quad \square-\square 1$$

[8]

Where:

$\square$: the percentage of arrival vehicles in the vehicle observed group during the red time;

$\square$: the total number of the vehicle in the observed vehicle group;

$\square$: number of the vehicle in the observed vehicle group arrive during the red time;

$\square$: red time of the intersection.(secs);

g: green time at the downstream intersection for the traffic movement under consideration (secs).

The arrival rate during green time is computed as $h2=g(1-\square)\times\square$. The dissipating rate during the green time is assumed as h1 (2 sec/veh) according to the field observation. Therefore, the queue dissipating speed (h3) during the green time is equal to $\frac{1}{1/h1-1/h2}$. As a result, the value of i in Figure 5 is computed as $i = k - g/h3$. In addition, the queue of the observed group dissipating speed is equal to h1. Since all the variables in Figure 5 are obtained, we can compute the total stopped delay of the observed vehicle group by computing the area under the curve shown in Figure 5.

Area A is computed as follows:

$$a = 0.5\times(k)\times(r-t1) \qquad [9]$$

Areas B and C are computed as follows:

$$b = 0.5\times(i+k)\times g \qquad [10]$$

$$c = 0.5\times(i\times h1)\times i + i\times r \qquad [11]$$

The average stopped delay for the observed group which is regarded as the intersection control delay for this case is $delay = \frac{a+b+c}{V\times CL}$. The part of reducing speed delay when the vehicle is approaching the intersection will be computed in the travel time on the link instead of the intersection control delay in our algorithm.

Case 2- There is an initial queue for the observed vehicle group and it is smaller than a cycle length

In this case, the queue size of the observed group vs. time as well as the intersection queue size is shown in

Figure 6. The intersection queue size includes the queue size of the observed group and the queue generated by the previous time intervals. In the Figure 6, both intersection queue size and observed vehicle queue size are shown before time t4. Before time t4, the bold curve represents the queue size of the observed group vs time. Another thin curve represents intersection queue size vs time. Only the queue of observed group is shown after time t4. As described above, d3 represents the clearance time of the initial queue. Let i' represents the number of vehicle in the initial queue and k represents the number of vehicle of the observed vehicle group arrive at the intersection during the red time. As

computed earlier, k is equal to $P \times N$. Let m represents the number of vehicles in the queue of the observed vehicle group at time t3 when the first vehicle in the observed group arrives at the intersection.

As shown in

Figure 6, the intersection queue will grow during the red phase to include the incoming vehicles from the observed group. At the end of the red time, the queue is now k+i'. It will take a time of g1 from the green phase to clear the initial queue. The intersection queue size is m at this time because some additional vehicles have joined the queue from the observed group. In addition, all vehicles in the queue are from the observed group at this time because the initial queue of observed vehicle group is clear at this time. During the time g1, even though the intersection queue size decreases, the number of vehicle belonging to the observed group in the queue increases. Because the vehicle in the observed vehicle group will continue join the queue until the entire initial queue is clear when is at the end of the g1 time. Therefore, the number of vehicles in the queue of the intersection is equal to the number of vehicles in the queue of the observed vehicle group at time t3.The queue of observed vehicle group will still decrease till the end of the green phase (g1+g2) and now reaches the level of i. The intersection queue size will increase during from t4 to t5 because of the coming vehicles in the vehicle group of in the next time interval. But as described in case 1, the queue of the observed group in the observed time interval, which is our main interest, remains constant during the red phase, and then dissipates again during the next green phase using a time (t6-t5).

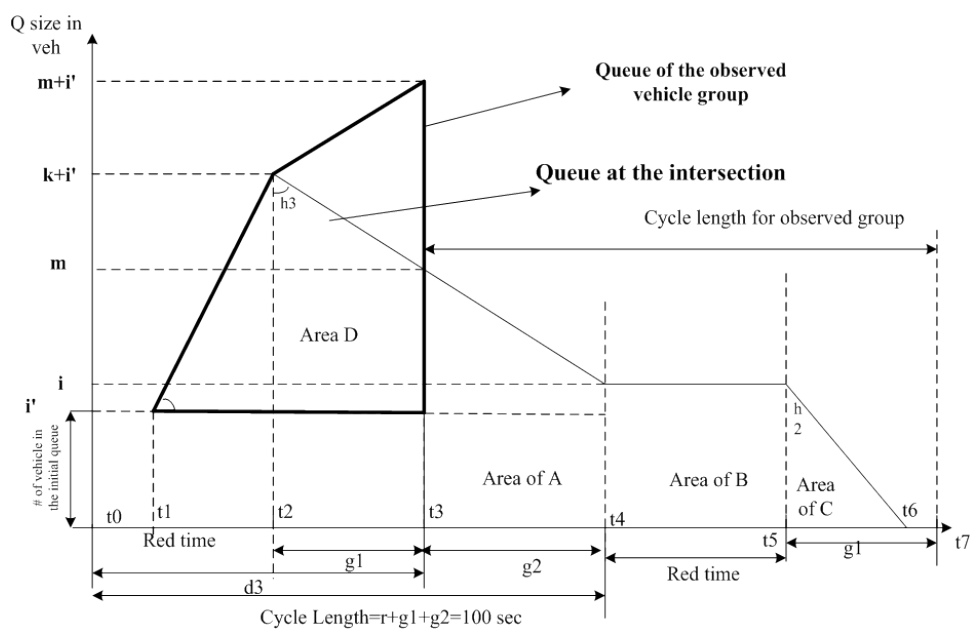

Figure 6 - Case 2- d3 is smaller than a Cycle Length

The total stopped delay encountered by the observed group during time the initial queue dissipated (d3) is shown as area D in Figure 6. In addition, used variables such as h1, h2, h3, k and i are the same ones used in computing the case 1 of stopped delay.

Therefore, the area D, which represents the stopped delay by initial queue, can now be calculated as follows:

$$d = 0.5 \times (r - t1) \times m + 0.5 \times (m + k) \times g1 \quad [12]$$

Where

$$m = k + \frac{N \times (1 - P)}{g} \times g1 \quad [13]$$

Area A is computed as follows:

$$a = 0.5 \times (i + m) \times g2; \quad [14]$$

the computation of value i is introduced in Case 1.

Areas B and C are computed as follows:

$$b = i \times r \quad [15]$$

$$c = 0.5 \times (i \times h1) \times i = 0.5 \times h1 \times i^2 \quad [16]$$

The average stopped delay for the observed group for this case is

$$delay = \frac{a + b + c + d}{V \times CL}$$

Case 3- Initial queue clearance time (d3) is Greater Than the Cycle Length

Since initial queue for the observed group is greater than the cycle length, the observed group will experience at least two red times and a green time before it arrives at the intersection. All vehicles in the observed group are assumed to be in the queue before they begin to depart the intersection. The queue of the observed vehicle group vs time is shown in Figure 7. As shown in Figure 7, the first vehicle in the observed vehicle will arrive at the intersection at time t1 which is t1-t0 after the beginning of the red time. After time t1, the queue will grow during the red phase t. At the end of the red time, the queue of the observed vehicle group is built to k. After time t2, the queue of the observed vehicle group will continue to increase till the end of the green phase (t3) and now reaches the level of N which is the number of vehicles in this observed vehicle group since no vehicle in the observed vehicle group can dissipate the intersection. After time t3, the queue will remain the same till the first vehicle arrives at the intersection at time t4. After t4, the queue of the observed vehicle will decrease at the dissipating rate h1.

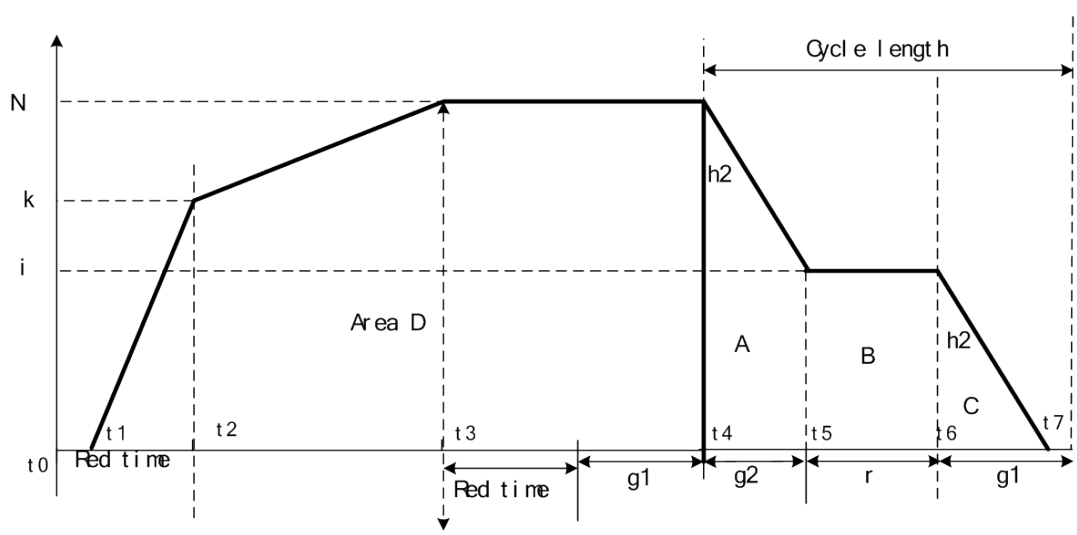

Figure 7 - Case 3 -d3 is Greater Than the Cycle Length

According to this assumption, the number of vehicles in the queue (N) in Figure 7 is equal to the number of vehicle in the observed vehicle group. The dissipation rate of the observed group is h1 (2 sec/veh). From Figure 7, $i = N - \frac{g2}{h1}$

Area A is computed as follows:

$$a = 0.5 \times (i + N) \times g2 \quad [17]$$

Areas B and C are computed as follows:

$$b = r \times i \quad [18]$$

$$c = 0.5 \times (i \times h1) \times i = 0.5 \times h1 \times i^2 \quad [19]$$

Area D is area under the curve from time 1 to time 4 which is computed as follows:

$$d = 0.5 \times (t2 - t1) \times P \times N + 0.5 \times (P \times N + N) \times g + N \times (r + g1) \quad [20]$$

The average stopped delay for the observed group for this case is

$$delay = \frac{a + b + c + d}{V \times CL}$$

Comparison of Results from Actual Delay vs. Algorithm and HCM2000

The real-field intersection control delays as well as the traffic volume are collected in order to compare the travel time estimation result with the developed methodology. In addition, algorithms in HCM2000 are utilized to compare the performance of the developed methodology. The delay of each vehicle was manually recorded at N Franklin St/Peppers Ferry RD intersection in Christiansburg, Virginia in order to evaluate the performance of the developed methods and HCM2000. The real average control delay and the estimation delays by the developed methodology and the methodology in HCM2000 are shown in Figure 8.

The Mean Absolute Error (MAE) is used to compare the fitness of two algorithm results with actual intersection control delay.

The Mean Absolute Error is calculated as $MAE = \frac{\sum_{i=1}^{n} Observed - Estimated}{n}$

The Mean Square Error is calculated as $MSE = \frac{\sum_{i=1}^{n} (Observed - Estimated)^2}{n}$

The average MAE is around 10.85 seconds for the developed algorithm but 14.28 seconds for HCM2000. The MSE are 169.72 and 323.8 for the developed algorithm and HCM2000 respectively.

In addition, we use the statistical regression test to analyze the relationship with actual control delay with HCM2000 and developed algorithm. The NOVA statistics are shown in Tables 1 and 2.

As shown in Table 1, the P-value is 0.182 which is greater than critical value of 0.05. Therefore, the result using HCM2000 is not statistically significant relative to the actual delay. However, the statistical test result shown in Table 2 is acceptable. It means the result using developed algorithm is statistically significant relative to the actual delay.

In general, the results show that the algorithms are robust and provide good accurate results when compared with HCM2000.

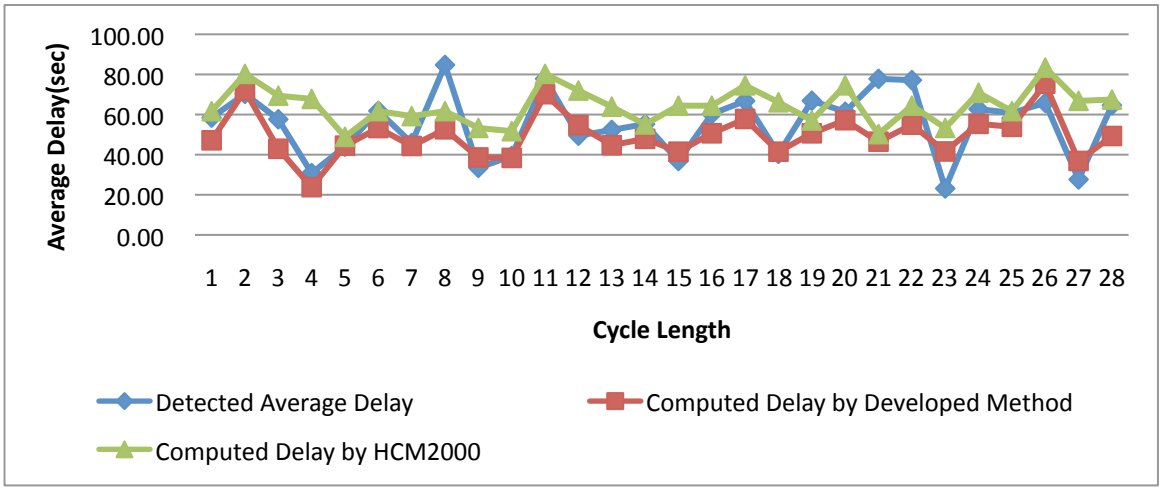

Figure 8- Detected Average Delay and Computed Delay with HCM2000 vs Computed Delay with Developed Method

Table 1 NOVA table for Actual Delay vs HCM2000

Source	DF	SS	MS	F	P
Regression	1	472.2	472.2	1.82	0.182
Residual Error	26	6733.4	259.0		
Total	27	7205.7			

Table 2 NOVA table for Actual Delay vs Developed Algorithm

Source	DF	SS	MS	F	P
Regression	1	4267.4	4267.4	37.76	0.01
Residual Error	26	2938.3	113		
Total	27	7205.7			

4 Conclusion

Estimation of travel time on arterials has been a challenging task because vehicles traveling on arterials are not only subject to queuing delay but also subject to traffic signal delay. Few transportation professionals have conducted research at estimating travel time on arterial street networks, and even fewer of them have utilized the dynamic flow methods to estimate these travel time in a short updated time interval utilizing the detectors in the middle of the link.

This paper presents the first step of how to compute intersection control delay based on the detected data during short time interval updates. As stated earlier, another component of the study was conducted to estimate travel time on a signalized arterial link that also considers traffic conditions on upstream and downstream links. These algorithms in the first step have been developed and have been validated by using actual data collected in the field as well as the developed algorithms in HCM2000. In general,

the statistics show that the developed algorithms are robust and provide good accurate results when compared with HCM2000.

Reference:

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