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Real-time estimation of queue length at signalized intersections

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REAL-TIME ESTIMATION OF QUEUE LENGTH AT SIGNALIZED INTERSECTIONS

by

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ABSTRACT

This study develops the method of estimating queue length at a signalized intersection. The method simplifies the past queue length estimation method that was developed using shock wave theory. This simplified method avoids complexity with calculations of shock wave speeds and accounts for the variations in vehicle effective length. The numbers of cars and trucks in each lane were observed upstream of the stop line at a signalized intersection in Windsor, Ontario. Maximum queue length among lanes was estimated in each cycle using second-by-second vehicle count and occupancy data collected from 7 locations of detectors. As a result, the method generally estimated the queue length more accurately than the shock wave method and the estimation errors were relatively consistent regardless of detector locations. The findings provide insights into the development of simpler queue length estimation method and the selection of the optimal location of detectors for accurate queue length estimation.

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CHAPTER I

INTRODUCTION

1.1 Background

Traffic congestion has been recognized as a major and growing issue in many urban and suburban areas. It has significant effects on the economy, air pollution, travel behavior, accident risk, land use and road users. Transport Canada (2006) reported that recurrent congestion in Canada's nine largest urban areas cause a loss of \$2.3- \$3.7 billion due to delay, fuel consumption and emissions in 2002.The Texas Transportation Institute (2005) estimated that traffic congestion in the 85 metropolitan areas in the US caused 3.7 billion vehicle-hours of delay, resulting in 2.3 billion gallons in wasted fuel and a congestion cost of \$63 billion in 2003.

The problems of queuing occur in many non-transportation fields such as the design and operation of industrial plants, retail stores, grocery check-out point counters, bank teller windows, restaurants, serviced oriented industries, computer and telecommunications networks etc. Queuing processes also occasionally occur in many surface roads such as the signalized and stopped-controlled intersections, freeway bottlenecks, incidents sites, toll plazas, parking facilities and merges areas near freeway on-ramps.

In particular, the analysis of queuing at signalized intersections is complex due to interaction of traffic movements in different approaches controlled by signals. The signal permits certain movement and prohibits other movements in each specified time interval. For example, a queue forms during the red interval, and then dissipates within the subsequent green interval. Thus, signal timing plan, capacity of the road section and the traffic flow demand are the major factors affecting the queue at a signalized intersection.

1.2 Importance of queue length estimation

A long queue increases delay at an intersection. It also reduces the capacity of an intersection through spillback and storage blocking between lanes groups. Therefore queue length has been recognized as an important measure for evaluating the operational performance of signalized intersections. At signalized intersections, queue length is a necessary input data for optimizing signal timing (Bang & Nilsson, 1976; Lin & Vijayakumar, 1988).

Queue length can be used to predict intersection delay, travel times and level of service at intersections. This information can potentially be provided to drivers so that they can avoid delay by choosing alternative route. Queue length is also critical to determine the required length of turn bays to prevent queued turning vehicles from overflowing in the bay and blocking vehicle flow in the adjacent through lanes. Queue length is also used to determine the spacing between successive intersections so that a queue does not frequently spill over the upstream intersection.

1.3 Limitation of existing research work

Over the years, numerous studies were conducted by many dedicated researchers (Newell, 1965; Robertson, 1969; Gazis, 1974; May, 1975; Catling, 1977; Akcelik, 1999; Strong et al., 2006; Sharma et al., 2007; Vigos et al., 2008).The past queue length estimation methods for signalized intersections can be classified into (1) Input-Output method and (2) Shockwave method. The Input-Output method estimates queue length using cumulative vehicle arrival counts (input) and departure counts (output). Vehicle counts are mostly collected from detectors at fixed locations upstream of the stop line. However, the Input-Output method has an inherent drawback (Liu et al. 2009). Queue length cannot be estimated when a queue spill over the location of detector.

On the other hand the other is the shockwave model which was developed based on the shockwave theory (Lighthill and Whitham, 1955; Richards, 1956) can estimate queue length even when a queue spills over the location of detector(s) (Stephanopolos and Michalopoulos, 1979, 1981; Muck, 2002; Skabardonis and Geroliminis, 2008; and Liu et al., 2009). However, the past studies on the shockwave method did not consider the following:

(1) The existing queue length estimation methods did not consider the variation in vehicle length although the length of heavy vehicle is longer than the length of passenger cars and queue length is affected by vehicle length.

(2) The existing methods assumed that the vehicle does not change lanes in the vicinity of stop line and did not consider the lane changing and differences in queue length across lanes.

(3) The existing methods did not investigate the effect of detector location on queue length estimation although traffic counts in each lane are significantly varied by the location of detector. If detector is closer to the stop line, more vehicles behind the detector location will not be counted. If detector is further away from the stop line, the number of lane change is not less to be captured.

1.4 Research objectives

The first objective of this research is to develop a methodology for estimating queue length at a signalized intersection on a cycle-by-cycle and lane-by-lane basis considering the variation in vehicle length.

The second objective of this research is to determine the optimal location of detectors at a signalized intersection using the proposed queue length estimation method to improve the accuracy of the estimated queue length.

1.5 Thesis organization

Following the first chapter (Introduction), the thesis is composed of subsequent five chapters.

The second chapter reviews different analytical approaches to queue length estimation and optimal detector location at signalized intersections of urban arterials roads.

The third chapter describes the observed traffic and road geometric characteristics of the studied signalized intersection.

The fourth chapter explains the queue estimation method developed using shock wave theory and the simplified method which overcomes the limitations of the shock wave method.

The fifth chapter compares the accuracy of queue length estimation between the shock wave method and the simplified method based on simulation results. The section also determines the optimal location of detectors for accurate estimation of queue length.

The last chapter presents conclusions and recommendations.

CHAPTER II

REVIEW OF LITERATURE

Queue length has long been recognized as a valuable measure for traffic engineers to evaluate performance of a signalized intersection. There are two definitions of queue length. Queue length at signalized intersections is typically defined as the distance between the stop line of the intersection and the rear end of the last queued vehicle (called "horizontal queue"). Queue length is also defined as the number of vehicles in queue (called "vertical queue"). However, vertical queue does not represent a physical length of queue that occupies the space. Vertical queue can be easily converted to horizontal queue if the vehicle length and distance headways are the same. However, if the length and headways of individual vehicles are significantly different, horizontal queue cannot be simply reflected by vertical queue. In this study, queue length is represented by horizontal queue.

This chapter reviews the past studies on the queue length estimation at signalized intersections and optimal location of detectors on urban arterial streets. This chapter at first presents the state-of-the-art methods of queue length estimation and optimal detector location determination at signalized intersections and identifies their assumptions and limitations.

2.1 Queuing analysis method

Queuing models are typically classified into (i) deterministic models and (ii) stochastic models. The distribution of vehicle arrival and service are assumed to follow a uniform pattern in deterministic models. Deterministic queuing analysis can be undertaken at two different levels. First, the analysis can be carried out at the macroscopic level, where the arrival and service patterns of vehicles are considered to be continuous. The analysis can also be carried out at the microscopic level, where both arrival and service patterns are considered to be discrete.

On the other hand, the arrivals and service time are assumed to follow some probability distribution(s) in stochastic models. Due to variation in arrival and service rates, queuing occurs in stochastic process. There are many types of probability distributions that can be used to represent the arrival and discharge processes of vehicles at a transportation facility. These are (i) random; (ii) Erlang and (iii) generalized probability distributions (May, 1990). However, since arrival and service processes do not always follow certain distributions, stochastic models may not reflect actual queuing behavior (Meyer and Miller, 2001). Since the proposed queuing analysis method in this research is deterministic and microscopic in nature, only deterministic models will be reviewed.

2.1.1 Deterministic queuing analysis at signalized intersection

Deterministic queuing length (vertical queue) can be estimated using cumulative vehicle counts (at signalized intersections) under the following assumptions: (i) traffic flow is undersaturated (travel demand is less than capacity) (ii) no vehicle waits for more than one cycle; (iii) overflow from one cycle to the next does not occur and (iv) the queue discipline is "first in, first out" (FIFO) system.

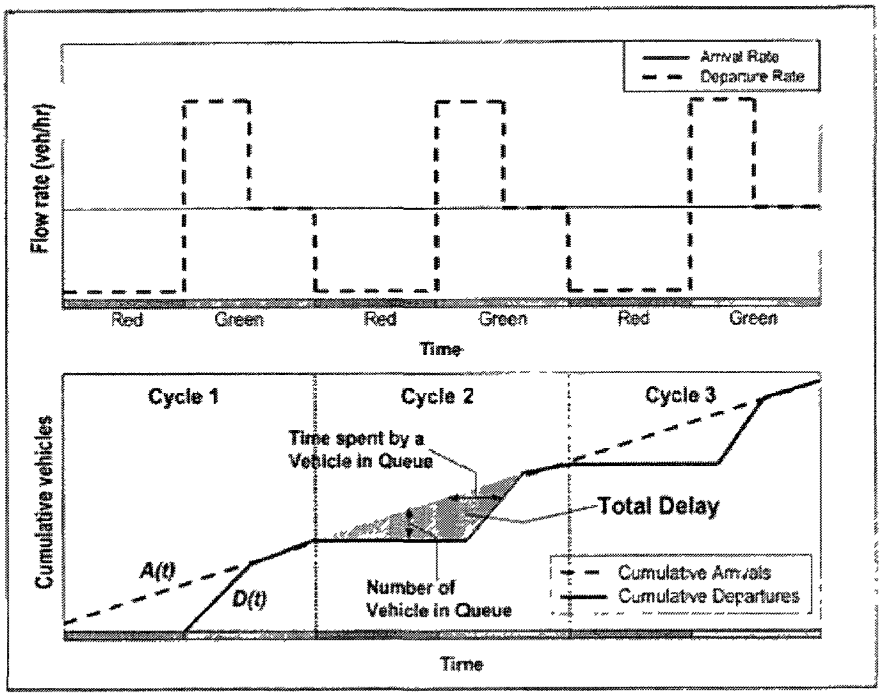


FIGURE 2-1: Deterministic queueing diagram for signalized intersection (Kang, 2000)

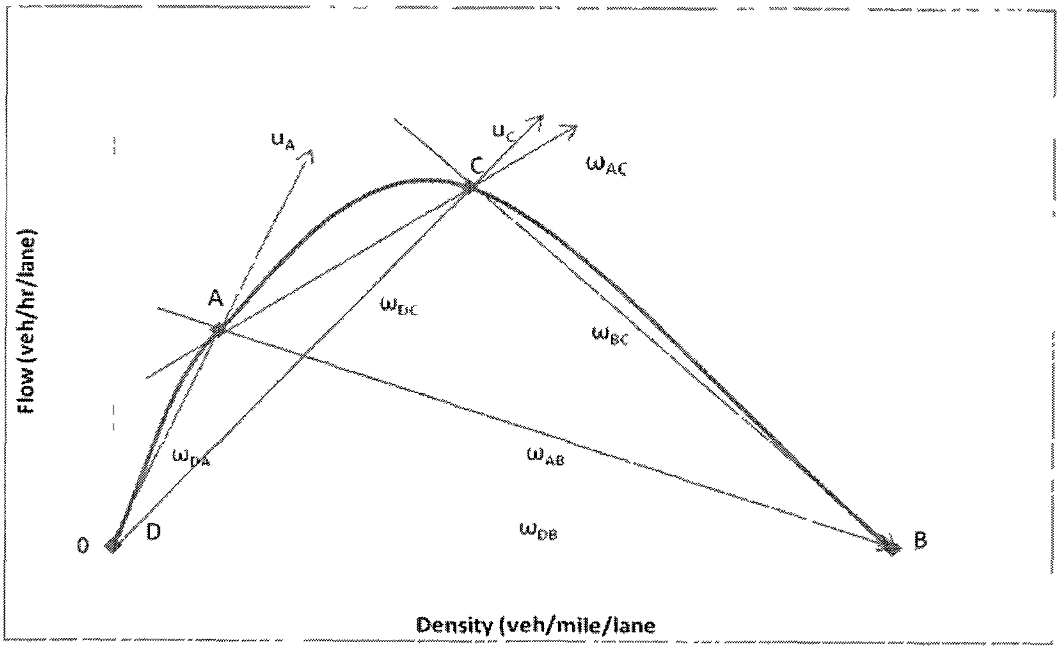
The queuing diagram in Figure 2-1 shows that, during the red interval of the cycle, no traffic can cross the stop line, i.e. the flow rate is zero. As a result, a queue starts to form and the maximum queue length occurs at the end of red interval or the beginning of green interval. Immediately after the signal turns to green, the queued vehicles leave the intersection at the saturation flow rate. It is noted that departure rate can be equivalent to the saturation flow only when the queue is present. When the queue clears some time after the start of the green interval, both the cumulative arrival and departure curves overlap, i.e. the service rate is equal to the arrival rate.

It this case, the queue formed during the red interval is always completely dissipated before the end of the green interval. However, if the intersection is oversaturated (i.e. the queue does not clear before the end of green interval) a residual queue (overflow queue) will occur in the subsequent cycles.

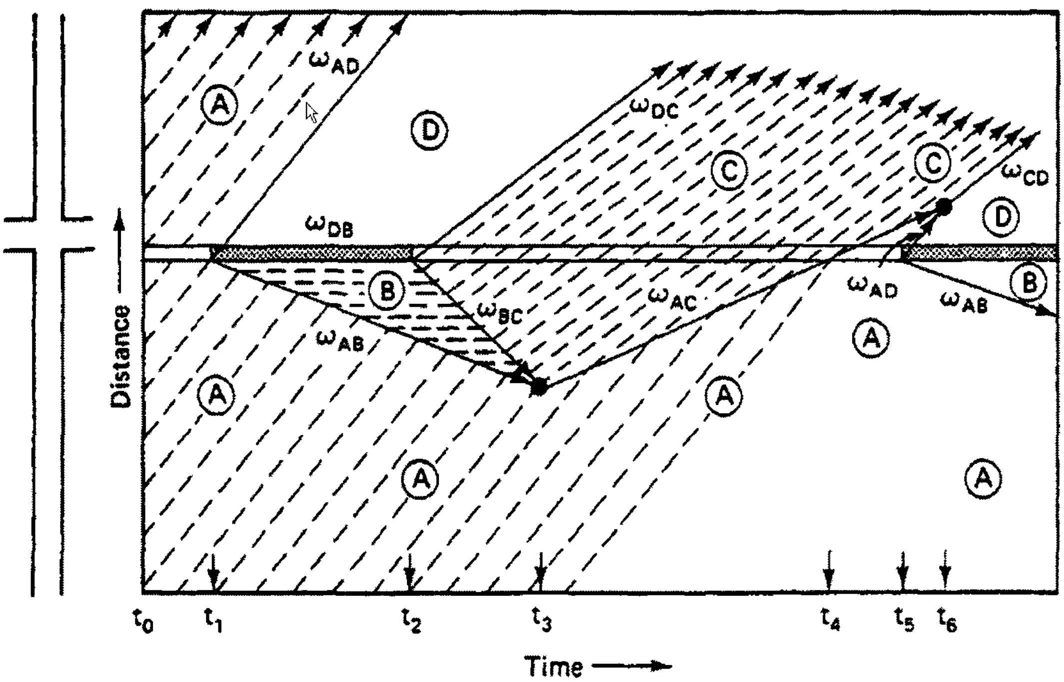
2.1.2 Shockwave analysis at signalized intersections

Queue length (horizontal queue) at signalized intersections can be estimated using shockwave theory if a flow-density relationship and the traffic states of the queue are specified. A typical flow- density curve is shown in Figure 2-2(a) and a distance - time diagram is shown in Figure 2-2(b). During time t_0 to t_1, the signal is green and traffic proceeds and the traffic state A is represented by flow (q_A) and density (k_A). At time t_1, the traffic signal changes to red and traffic state immediately upstream of the stop line changes to the traffic state B while the traffic state immediately downstream changes to state D. Three shock waves begin at time t_1 at the stop line. These are ω_{AD}, ω_{DB} and ω_{AB}.

When the departure flow at the stop line increases from zero to saturation flow the traffic state C forms. This terminates shockwave ω_{DB} and generates two new shockwaves, ω_{DC} and ω_{BC}. The flow states of A, B, C and D continue until ω_{AB} and ω_{BC} intercept at time t_3. At time t_3 a new shockwave ω_{AC} is formed, and the two shockwaves ω_{AB} and ω_{BC} are terminated.



(a) flow- density relationship



(b) Distance - time diagram

FIGURE 2-2: Shockwave analysis at signalized intersections (May, 1990)

Based on this shock wave theory, a number of real-time estimation of vehicle-count and queue length estimation models were recently developed (Strong et al., 2006; Sharma et al., 2007; Skabardonis and Geroliminis, 2008; Vigos et al., 2008; Liu et al., 2009; Zheng et al., 2009; Ban et al., 2010).

2.2 Real-time queue length estimation model

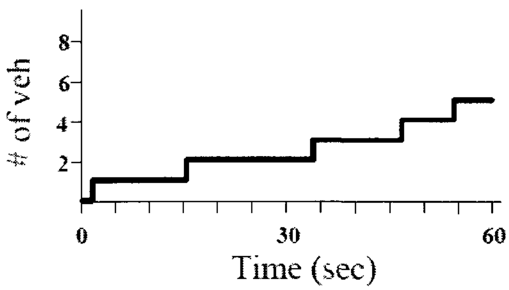
2.2.1 Input-Output and Hybrid queue length estimation technique

Sharma et al. (2007) applied two techniques (Input-Output and Hybrid technique) for the real time prediction of vehicle delay and queue length at signalized intersections based on the cumulative arrival and departure traffic counts collected from loop detectors. The Input-Output technique uses advance detector actuations, signal timing data and parametric data (e.g. saturation headway, start-up lost time, arrival shift, storage capacity etc). In case of Input-Output technique, advance detector is placed upstream of the stop line for recording arrival flow over time (input). On the other hand, the estimated saturation flow rate is used to determine the number of vehicles crossing the stop line over time (output). These two flow profiles are used to calculate the number of vehicles in a queue approach between stop line and the location of the advance detectors (Sharma et al., 2007).

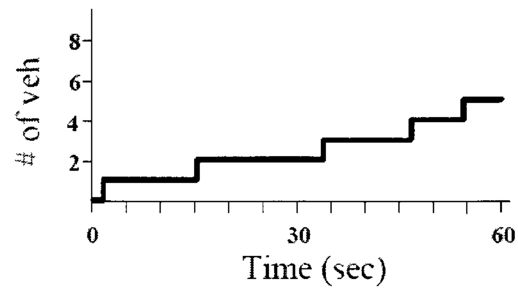
The Hybrid technique uses advance detector actuations, stop bar detector actuations, phase change data and parametric data (e.g. storage capacity etc) as model inputs. The Hybrid technique records number of vehicles passing the loop detectors placed upstream of the stop line for counting arrival flow and at the stop bar for counting departure flow. The number of vehicle in a queue is calculated using arrival and departure flow profiles. Then, the queue length (horizontal queue) can be estimated by multiplying the number of vehicle in a queue by the effective length of vehicles in jam traffic state. However, the evolution report showed that the result for the Hybrid technique was not as good as the Input-Output technique in spite of using more input data. The graphical representation of the Input-Output and Hybrid technique are shown in Figure 2-3.

However, there were some limitations with this method: (i) "The Input-Output method is insufficient for obtaining the spatial distribution of queue lengths in time" (Michalopoulos and Stephanopolos, 1981); (ii) it cannot estimate queue length when a queue spills over the location of detector (Liu et al., 2009); (iii) both the Input-Output and Hybrid techniques were developed based on the assumption that vehicles do not change lanes after they cross the advance detectors might degrade the performance of the technique and (iv) the Input-Output method used many assumed values of input parameters (e.g. saturation flow rate, storage capacity, etc).

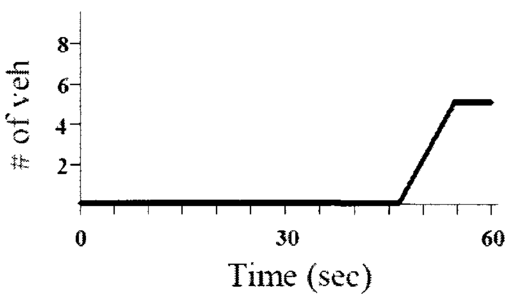
Vigos et al. (2008) developed a methodology for real-time estimation of vehicle-count within signalized links. The number of vehicles in signalized links is valuable information for urban signal control. A Kalman-Filter was used to estimate vehicle counts in real time in signalized links based on measurements at (at least) three loop detectors stations located at both end points and the middle of the link. The vehicle-count estimation problem is illustrated in Figure 2-4. Figure 2-4(a) depicts the relevant signal and detector configuration on the signalized link. The upstream signal (if it exists) determines the traffic demands approaching the link while the downstream signal controls the vehicle flow exiting the link.



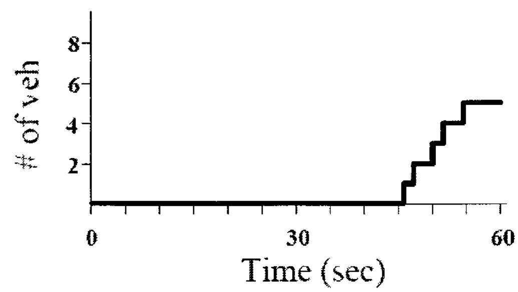
a) Input-Output arrival profile



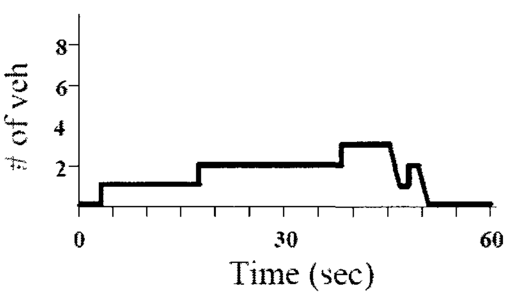
b) Hybrid arrival profile



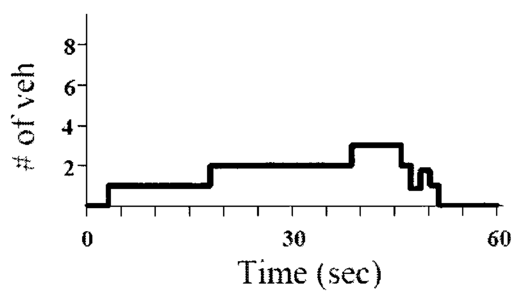
c) Input-Output departure profile



d) Hybrid departure profile



e) Input-Output queue profile for computing delay



f) Hybrid queue profile for computing delay

FIGURE 2-3: Comparison of queue length measurement techniques

(Sharma et al., 2007)

Obviously, whenever the link demand is larger than the link outflow, a queue is built. It is shown in Figure 2-4(a) that three detectors are installed: at the upstream end of the link, at the downstream end of the link and in the middle of the

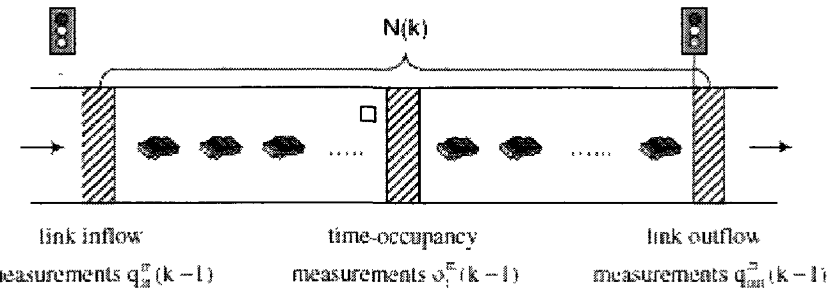
Both boundary detectors provide flow measurements, while the middle detector provides time-occupancy measurements. The basic structure of the queue estimation is shown in Figure 2-4(b). The estimator is imported in real-time (every k) with the flow and time-occupancy measurements from the link detectors. The estimators produce the estimated number of vehicles in the link (between the two boundary detectors) for every k. The number of vehicles in a link obeys the following conservation equation:

$$N(k) = N(k-1) + [q_{in}(k-1) - q_{out}(k-1)] \tag{2-1}$$

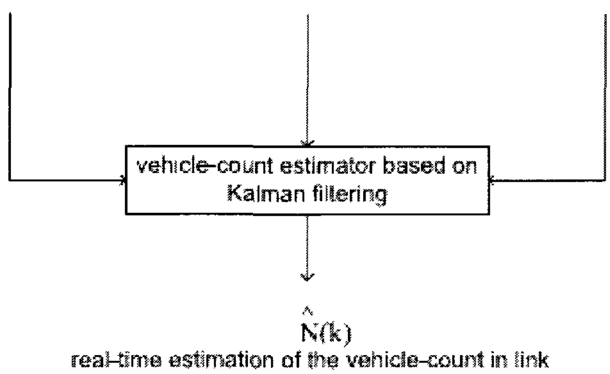
where N(k) denotes the number of vehicles in the link at time k and k is the sampling time (e.g. 20 sec) ; $q_{in}(k-1)$ and $q_{out}(k-1)$ are the flows entering and exiting, respectively, the link during the period [(k-1), k].

2.2.2 Shockwave maximum queue length estimation techniques

Liu et al. (2009) developed a method to estimate the real-time queue length for the congested signalized intersection using the shockwave theory. They used the SMART-SIGNAL (Systematic Monitoring of Arterial Road Traffic and Signals) data collected from the advance detector placed upstream of the stop line. This method enables the estimation of queue length even when a long queue spills over the advance detector.



(a) The signal and detector configuration on the link



(b) The link vehicle-count estimation

FIGURE 2-4: Vehicle-count estimation (Vigos et al., 2007)

The high-resolution event-based data collected by the SMART-SIGNAL identify "Break Points" (A, B, and C) in the occupancy profile and time gap profiles which indicate the times when the traffic condition changes within a cycle (Figure 2-5 & 2-6).

These break points are used to calculate the speed of different shockwaves generated at the signalized intersection due to signal changes. The shockwave propagation at the signalized intersection is shown in Figure 2-7.

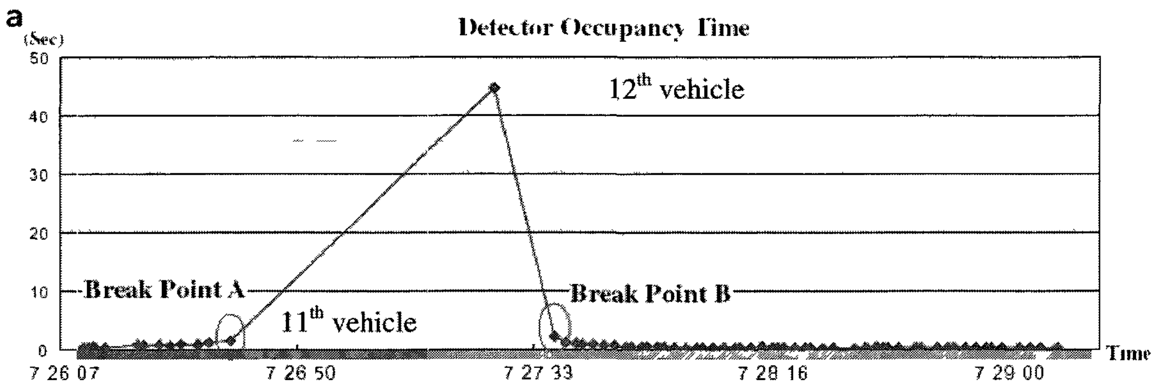


FIGURE 2-5: Detector occupancy profile in a cycle (Liu et al., 2009)

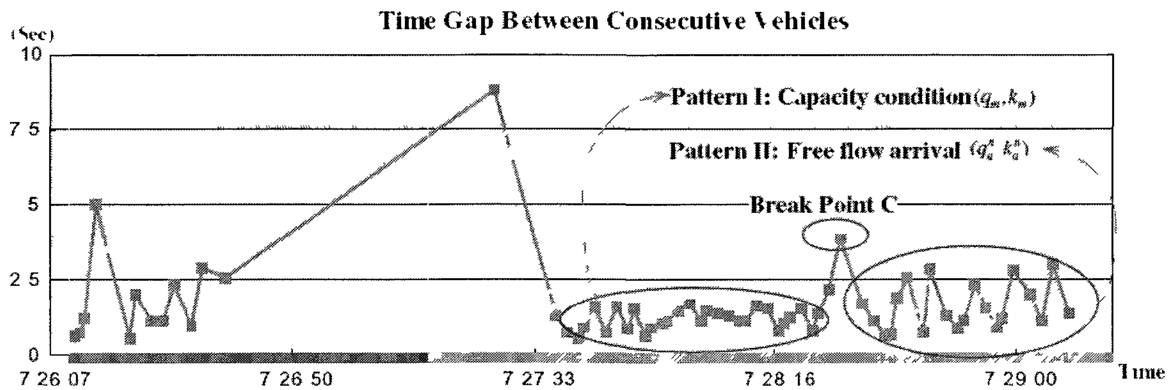


FIGURE 2-6: Time gap between consecutive vehicles in a cycle (Liu et al., 2009)

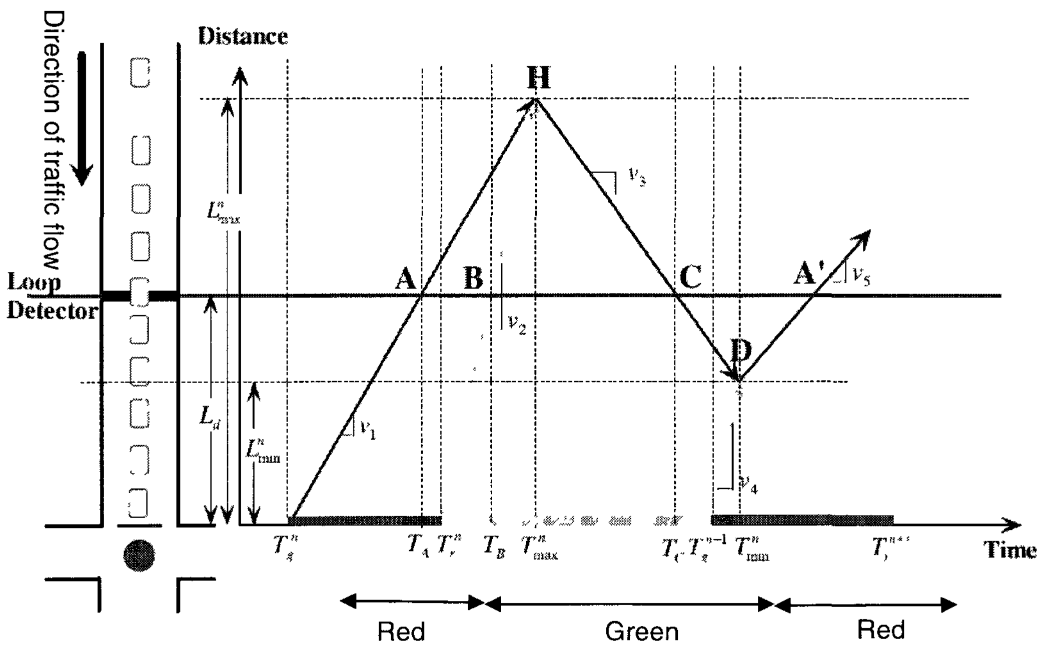


FIGURE 2-7: Shockwave technique (Liu et al., 2009)

When the signal turns to red (T_g^n), two shock waves are generated. First, a queuing shockwave propagates toward the upstream of the traffic flow at a speed v_1. At the start of the green time (T_r^n), the queue begins to discharge at saturation flow state and a discharge shockwave propagates towards the upstream of the flow at a speed v_2. When v_1 and v_2 meet a $T^n{}_{max}$, a departure shockwave propagates toward the stop line at a speed v_3. For over-saturation condition, the queue cannot be fully discharged within one cycle and the fourth shockwave is formed to generate a residual queue moving upstream at a speed v_4.

Once Break Points (A, B, and C) have been identified, the flow (q) and density (k) of each traffic state (i.e. the arrival traffic state (q_a, k_a) and saturation traffic state (q_m, k_m)) can be calculated based on detector occupancy times and time gaps between vehicles. The wave speed of v_3 can be estimated using following equation (Liu et al., 2009):

$$v_3 = \frac{q_a - q_m}{k_a - k_m} \tag{2-2}$$

The discharge wave speed v_2 can be calculated based on the distance from the stop bar (L_d), the difference between the green start time (T_r) and the time when the discharge wave reaches advance detector (T_B) as shown in the following equation (Liu et al., 2009):

$$v_2 = \frac{L_d}{T_B - T_r} \tag{2-3}$$

where, L_d is the distance from the stop line to the loop detector and T_B is the moment when the discharge shockwave (v_2) passes the detector at point B.

Liu et al. (2009) also developed the following equation to calculate the maximum queue (L_{max}) using the estimated values of v_2 and v_3 (Figure 2-6):

$$L_{max} = L_d + \frac{T_C - T_B}{\left(\frac{1}{v_2} + \frac{1}{v_3}\right)} \tag{2-4}$$

where T_A is the moment when the queuing shockwave (v_1) passes the detector at point A and T_C is the moment when the departure shockwave (v_3) passes the detector at point C.

However, the method cannot estimate the queue length when oversaturation occurs. Also, the Break Point C cannot always be determined because two traffic states (queue discharge flow and new arrival flow) are not easily distinguished. Only one type of vehicle (car) is considered in this method. Since this method uses a pre-determined constant effective vehicle length of 7.62 m (25 ft) to estimate individual vehicle speed, it may lead to errors in queue estimation.

Skabardonis and Geroliminis (2008) proposed an analytical kinematic wave model to construct a link travel time and queue length estimate using aggregated 30-second flow and occupancy data from a loop detector upstream of the stop line and the exact times of the red and green phases from the signal controller, the researchers used the aggregated 20-30 second average data.

The arrivals at the upstream detector at distance L_d from the stop line can be predicted as follows: If the flow is near zero, the queue exists. If the sufficiently low flow follows the saturation flow, the queue clears and new flow arrives. As vehicles are moving at free flow speed when departing from the queue, the traffic state changes from the jam state to the saturation state. The maximum length of queue can be identified using data from a detector placed at distance (L_d) upstream of the stop line and the geometry of trapezium (ABCD) in Figure 2- 8 (Skabardonis and Geroliminis, 2008)

$$L_m = [u_f \cdot w / (u_f + w)] \cdot g + L_d = (c / k_j) \cdot g + L_d \tag{2-5}$$

where, L_m = maximum length of queue; c = cycle length; g = green phase; k_j = jam density; u_f = free flow speed in the absence of queues; and w = congested shock wave.

However, free flow speed (u_f) is not always constant in different traffic conditions, and it leads to error in estimation. Since 30-s aggregation dampens variations in vehicle gaps, it is difficult to identify the end of the queue unless the arrival traffic flow is significantly lower than the queue discharge flow. Since queue length estimation was only part of their travel time estimation model, the accuracy of the queue length estimation method was not evaluated.

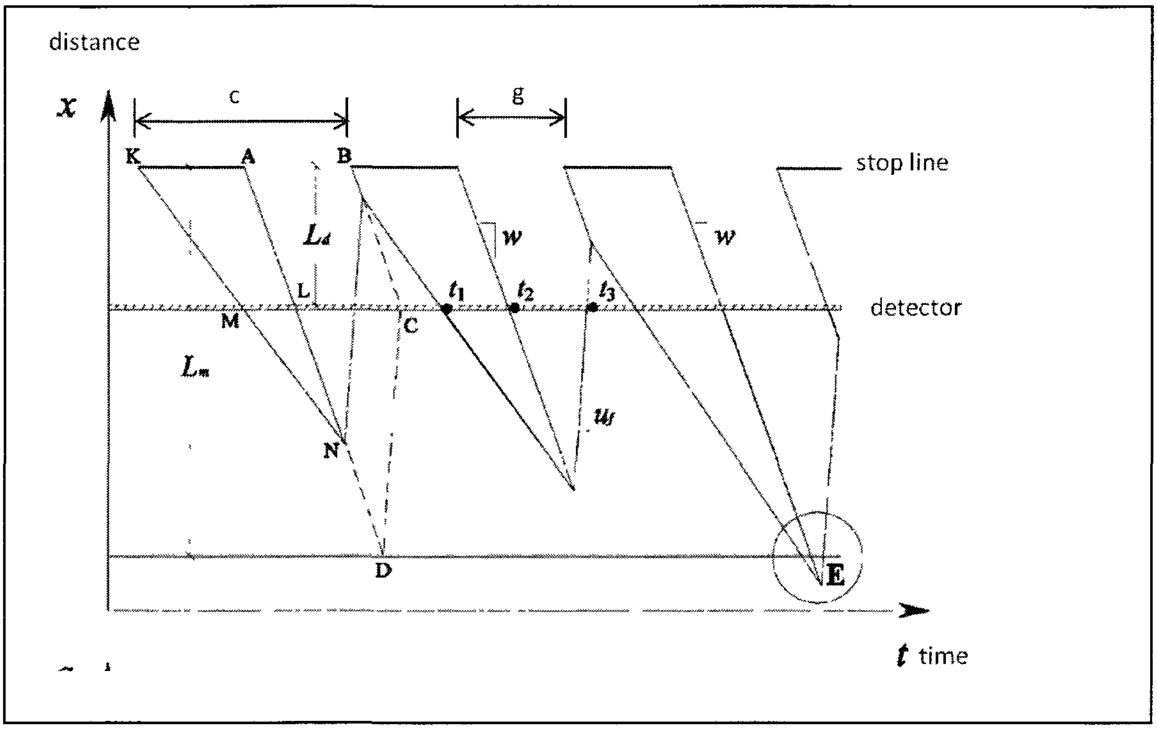


FIGURE 2-8: Space-time diagram (Skabardonis and Geroliminis, 2008)

2.2.3 Recent queue length estimation method

Zheng et al. (2009) developed algorithms designed for measuring average control delay and queue length (vertical queue) at signalized intersections using traffic count data collected from Video Image Processors (VIPs). These algorithms are implemented in a computerized system called In-PerforM (Intersection Performance Measurement) for measuring the performance of signalized intersections in real time. In this study, all the lanes of the approach are equipped with two traffic sensors, placed upstream of the stop line and at the stop line as shown in Figure 2-9. The area between the entry loop and exit loop is called the measuring zone. The difference in the vehicle counts between the entry loop and the exit loop (i.e. vehicles inside the measuring zone) in each lane is taken as the queue length during the red signal indication.

This study estimated lane-by-lane queue length (vertical queue) considering the lane flow, and the left-turn and right-turn flow ratios. This study estimated lane-by-lane queue length (vertical queue) using vehicle counts in each lane obtained from detectors upstream of the stop line and at the stop line.

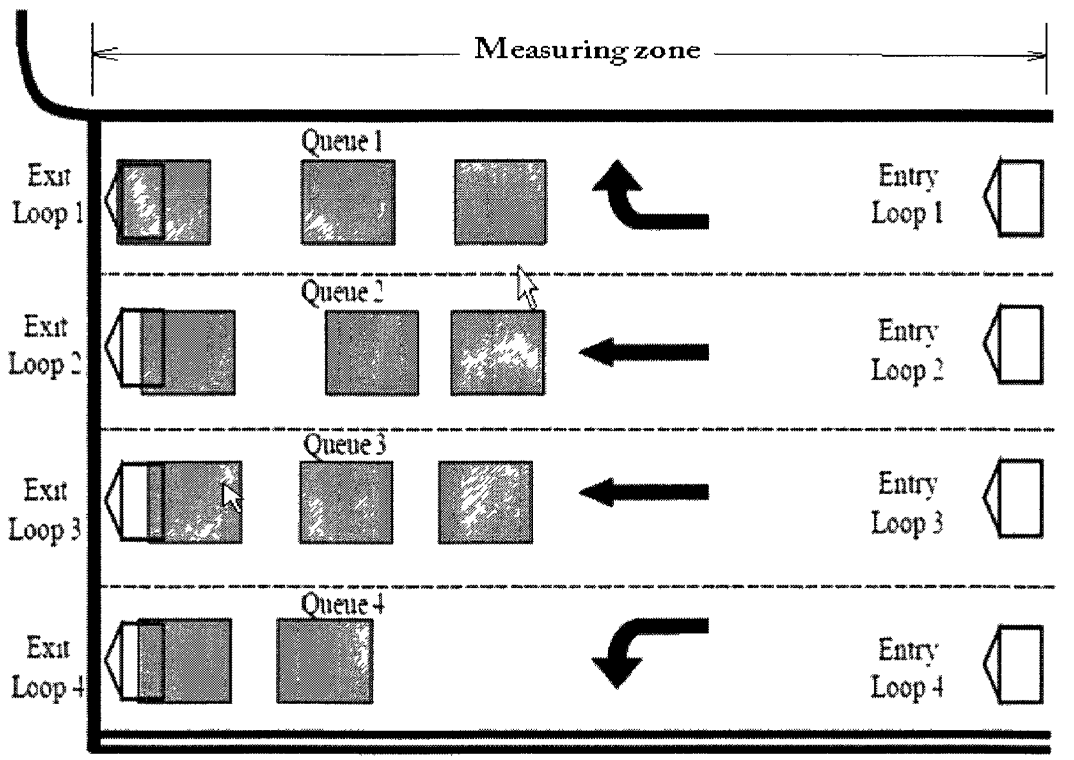
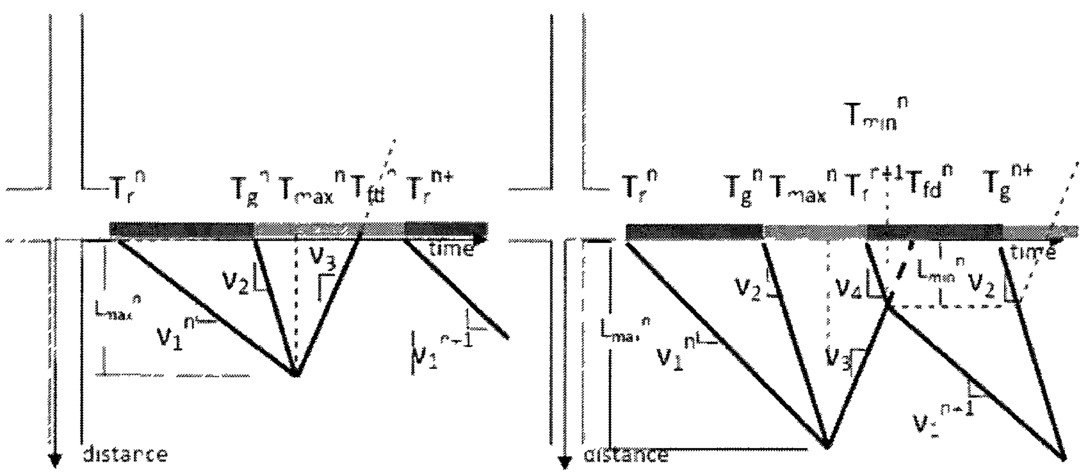


FIGURE 2-9: Layout and sensor configuration of the study site (Zheng et al., 2009)

In spite of considering lane-specific queue length, the method implicitly assumes that vehicles do not change lanes as they pass the detectors upstream of the stop line. If more vehicles change lanes after passing the detectors, the estimated queue length is likely to be less accurate.

Similar to the Input–Output method, this method cannot estimate the queue length if the vehicle queue extends beyond the entry loop. Lane change activity within measuring zone was not taken into consideration. This study assumes the constant vehicle length of 5.5 m (18 ft).

Ban and Hao (2010) recently developed a new method to estimate queue length at signalized intersections using sampled travel times of individual vehicles. Sampled travel times are directly measured using the GPS equipped mobile traffic sensors within Virtual Trip Lines (VTL). VTL is referred to as the upstream and downstream locations of an intersection for travel time collection. The key concept of this method is the Queue Fully Discharge Time (QFDT), which is the time when the queue would have been fully discharged if there had been enough green time (Ban and Hao, 2010). This QFDT concept is based on shockwave theory. Figure 2-10 depict the propagation of shockwaves in undersaturated and over-saturated condition respectively at a signalized intersection. T_r^n and T_g^n indicates the end and start time of the effective green during the nth cycle.



(a) Undersaturated condition (b) Oversaturated condition

FIGURE 2-10: Shockwave propagation (Ban and Hao, 2010)

The Queue Fully Discharge Time (QFDT) is defined as T_{fd} in Figure 2-10(a) and 2-10(b). For unsaturated conditions, QFDT is the time when the departure shockwave (v_3) reaches the stop line, (i.e. the time that the queue is fully discharged). For over-saturation conditions, shockwave v_3 meets residual shockwave (v_4) before it reaches the stop line, so vehicles in the residual queue have to wait for an extra red time. Based on the definition of QFDT, all of the queuing vehicles could be fully discharged from T_g^n to T_{fd}^n at a saturation flow rate (q_m). Therefore the maximum queue length of the n^{th} cycle is

$$Q^n_{max} = (T^n_{fd} - T^n_g)\, q_m \qquad (2\text{-}6)$$

Similarly, the minimum queue length is

$$Q^n_{min} = (T^n_{fd} - T^{n+1}_r)\, q_m \qquad (2\text{-}7)$$

However, mobile sensor data are only collected from the sample vehicles. They do not represent the involvement of entire vehicles in a traffic stream. This study assumes that arrival flow is uniform and the queue does not spill over the upstream VTL. Also, the assumption that a queue clears in two cycles may not be valid in many arterial intersections where the queue comprises long vehicles. Furthermore, lane changes activity within the area between virtual trips lines are not taken into consideration in this study.

2.3 Optimal location of detectors

Traffic data collected from loop detectors vary at different detector locations upstream of the stop line at signalized intersections. In particular, the likelihood of queue spillback and lane change is affected by the location of detectors. For instance, if loop detectors are closer to the stop line, vehicles are less likely to change lanes after they pass the detectors. Thus, lane-specific queue length estimation is more likely to be accurate. On the other hand, queue spillback is more likely to occur and it is harder to estimate the number of queued vehicles beyond the detector location. Thus, detector locations significantly affect the accuracy of queue length estimation.

The past empirical studies reported that detectors were located at various locations - 30-123 m (100-405 ft) upstream of stop line (Sharma et al., 2007; Liu et al., 2009; Federal Highway Administration, 2006; Smaglik et al., 2007; Wang and Wu, 2007) as shown in Table 2-1.

TABLE 2-1: Detector Location in Several Projects

Study	Project	Distance from stop line	Location
Wang and Wu, 2007	Google Map based online platform for arterial traveler information (GATI) broadcast and analysis	30 m - 40m	Collected volume and occupancy data via loop detection placed at 30-40 m (100-130ft) in advance of intersection stop bars. City of Bellevue, Washington
Liang ,2006	Development of the real time arterial traffic flow map	30 m - 42m	Used advance loop detectors to collect volume and occupancy information placed approximately 30-42 m (100-140 ft) in advance of signalized intersections. City of Bellevue, Washington
Oh and Choi, 2004	Optimal detector location for estimating link travel speed in urban arterial roads	61m	For links approximately 2000 ft in their lengths, the optimal detector location were identified to be about 61 m (200 ft) from downstream intersection for green times of 20, 30,40 and 50 seconds, respectively. University Avenue, Madison
Federal Highway Admin., 2006	Traffic detector handbook: Third edition – volume 1, chapter 4,operations and intelligent transportation systems research	61m	Where a major driveway is located within a link, the loop should be located at least 15m (50 ft) downstream from the driveway, provided that the loop is at least 61m (200 ft) upstream of the stop line

TABLE 2-1: Detector Location in Several Projects (Continued)

Study	Project	Distance from stop line	Location
Gerolminis, 2009	Queue spillovers in city street networks with signal-controlled intersections	75m	System loop detectors are located on each lane approximately 75 m (250 ft) upstream of the intersection stop line. Lincoln Avenue, Los Angeles international Airport
Liu et al., 2009	Real-time queue length estimation for congested signalized intersections	122m	Collected the data from the advance detector placed at 122 m (400 ft) away from the stop line. Rhode Ave., Minnesota
Sharma et al., 2007	Input-Output and Hybrid techniques for real-time prediction of delay and maximum queue length at signalized intersections	123m	Collected data for arrival profile from the detector e placed at a distance 123 m (405 ft) in advance of the stop line. Noblesville, Indiana
Smaglik et al., 2007	Event-based data collection for generating actuated controller performance measures	123m	Placed set-back detectors 123 m (405 ft) back from the stop bar at the northbound and southbound approaches at the INDOT intersection test bed. Noblesville, Indiana
Thomas, 1998	Simulation of detector locations on an arterial street management system	91m, 183m, 274m	Varying the location of the downstream detector did not have a big impact on the output variables when the distance was greater than 122m (400 ft.). If detectors are placed less than 122m, the results could be misleading because vehicles are still accelerating close to the intersection. Southern Avenue, Arizona

Although the location of detectors is important for capturing shock waves at signalized intersections (Abbas and Bullock, 2003), no studies analyzed the impact of detector location on queue length estimation and identified the optimal (or near optimal) detector location to minimize estimation error.

Given that the accuracy of the estimated queue length by the most real-time models depends on the location of detectors, the determination of optimal location for accurate queue length estimation is important. Some studies have examined determination of the detector location for freeways and urban arterials.

Thomas (1998) derived the relationship between detector location and travel characteristics (link speed, travel time, intersection delay) on arterial streets. The study examined the relationship between traffic characteristics and detector locations for 92 m (300 ft), 183 m (600 ft) and 275 m (900 ft) from the downstream intersection and 183 m (600 ft) from the upstream intersection (Figure 2-11). Regression analysis was used to evaluate the relationship between detector output values (occupancy and speed) and link travel characteristics (link speed, travel time, intersection delay).

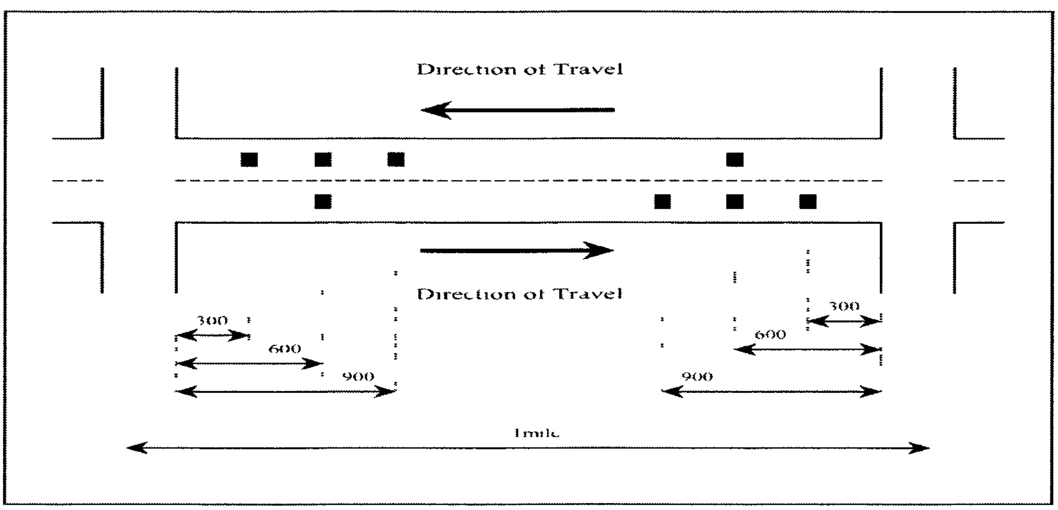


FIGURE 2-11: Surveillance detector locations (Thomas, 1998)

The study found that it is generally difficult to find "optimal" detector locations, for all links in all cases. Although the results shows that 92 m (300 ft) point from the downstream intersection was the optimal detector location, it is valid only for existing traffic volumes, green time, intersection geometric, lanes, and speed limit. Therefore, the results might not be transferable to different sites and/or areas.

Thomas et al. (2000) have done a similar study in the same area. They used CORSIM simulation in their research. CORSIM is a combination of microscopic network simulation and microscopic freeway simulation model. CORSIM stimulates the movement of individual vehicles by category in every second (Mystkowski and Sarosh, 1998). They found that varying the location of the downstream detector did not have a significant impact on the output variables (volume, occupancy, speed) when the distance was greater than 122 m (400 ft) from the downstream intersection. If the distance is less than 122 m, the results could be misleading because vehicles are still accelerating close to the intersection. They also found that there was no particular detector location with higher accuracy of travel time than the other detector locations. The research showed that detector data obtained on one link could not accurately predict link travel characteristics on an adjacent link.

Oh and Choi (2004) developed an extended model based upon the research by Sisiopiku et al. (1994) to find the optimal detector location using link travel speed in urban arterial roads. They used the CORSIM traffic simulation with changing link length, traffic volume, average link speed, detector location, number of lanes and estimated the average speed at each detector location. The optimal detector location was selected such that the link average speed is similar to the speed measured at the detector (Oh and Choi, 2004).

It was found that the optimal detector location is determined by link length and green time with other factors (number of lanes, traffic volumes, and speed limits) also. The result shows that for links with approximately 610m (2000 ft), the optimal detector locations were about 61m (200 ft) from downstream intersection for diverse green times of 20 to 50 seconds. It was also found that with the increase of link length, optimal locations were more dependent on green times. Figure 12 shows the results of optimal detector locations.

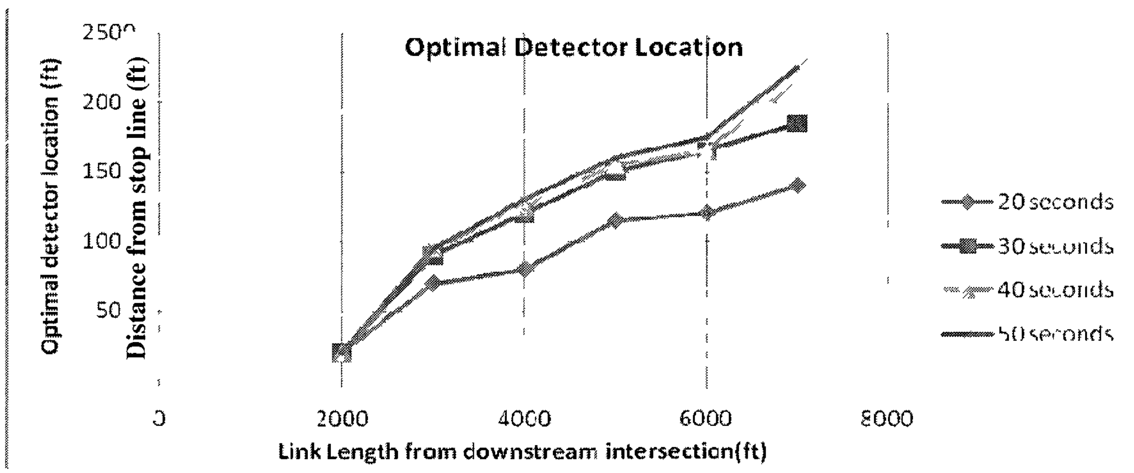


FIGURE 2-12: Optimal detector location (Oh and Choi, 2004)

The research focused on the detector locations over medium size links of 610 m - 2134 m (2000-7000 ft) in length. This limits the applicability of the results obtained in this research if links which length is shorten than 610 m (2000 ft.) This research did not include the issues of access, turning movement; effect of pedestrian's movement, geometric components such as slope and lane width variations.

Abbas and Bullock (2003) developed a model to determine the location of detector that can best capture the effect of shockwaves formed at signalized intersection. It was found that when the detector is placed very far from a signalized intersection, it would not capture the existence of the shockwave caused by the downstream bad offset. On the contrary, if the detector was very close to the intersection, it would be affected by the weaker shockwave generated by the traffic turning from the side street.

Edara et al. (2008) developed a methodology to identify the optimal locations of detectors on freeways in order to minimize the error in travel time estimation. They found that the placement of detectors for estimating accurate travel time will vary by location based on specific traffic and geometric conditions. They recommended that since the traffic conditions change over time, detector placement will require periodic validation and modification to ensure continued accuracy. The method showed that the detector density needs to be higher in congested areas of corridor. Uncongested sections of the corridor need only a nominal deployment.

An objective function is used to minimize the travel time estimation error. Estimation error is the difference between the estimated travel time and the ground truth travel time for the freeway section. The freeway section was divided into discrete segments. This means that the detectors can be deployed only at the mid points of these discrete segments. If there are m discrete locations then n detectors can be placed in ${}^{m}c_{n}$ possible ways (e.g. if m is 50 and n is 5, then the size of the solution space will be ${}^{50}c_{5}$ =2 million combinations) (Edara et al., 2008). Exact solutions to such combinatorial optimization problems were complex and not straight forward. A heuristic search algorithm known as Genetic Algorithms (GA) was used to generate useful solutions to optimization using a population (set) of solutions (Goldberg, 1989). Steps involved in GA are as follows (Edara et al., 2008):

(1) The parameter set for the problem has to be encoding first, as a binary or real number representation.

(2) The initial population of P solutions (strings) has to be generating randomly and the fitness value (objective function value) for each of these solutions has to be evaluated.

(3) Two strings from the current generation (parents) have to be selected for participating reproduction, the selection probability being proportional to the fitness value.

(4) Parents selected in step 3 are mated by exchanging genetic material to produce two offspring.

(5) Mutation operator is applied to the newly born offspring.

(6) Steps 3, 4 and 5 have to be repeated until offspring are generated. These offspring constitute the new generation of solutions.

(7) The old population of solutions will have to be replaced with the newly generated offspring and steps 3 through 7 will have to be repeated until a pre-specified number of generations or other convergence criteria are met. Final solution is the best solution from those discovered during the search.

However, this method is developed for a freeway with long distance to minimize the error in travel time estimation. Thus, it may not be suitable for identifying the location of detectors at signalized intersections for queue length estimation.

CHAPTER III

DATA

3.1 Study area

This study analyzes the signalized intersection at Huron Church Road and Tecumseh Road in Windsor, Ontario, Canada as shown in Figure 3-1. The intersection is one of the busiest intersections in Windsor due to its proximity to the Ambassador Bridge, the busiest Canada-U.S. international border crossing. This intersection was chosen since high volume of trucks pass through the intersection and a long queue of vehicles frequently occurs on the northbound road. The posted speed limit at this intersection is 60 km/h and the cycle length is 120 sec. The durations of displayed green and red intervals for northbound through approach are 42 seconds and 71 seconds, respectively.

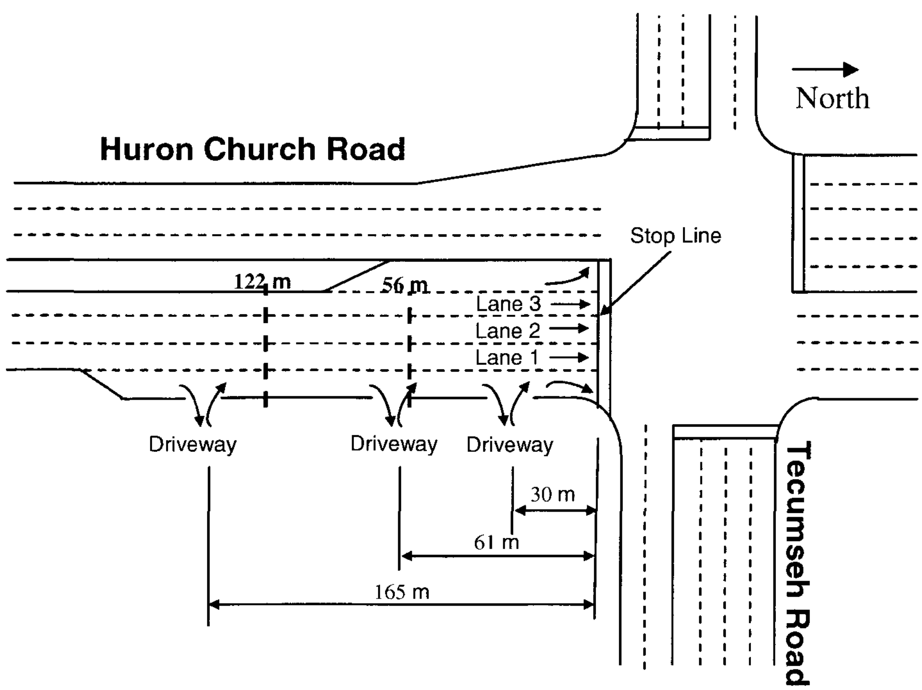


FIGURE 3-1 Schematic drawing of Huron Church-Tecumseh intersection

Huron Church Road has three through lanes (Lanes 1, 2 and 3), an exclusive left-turn lane and an exclusive right-turn lane. There are three driveways at 30 m, 61 m, and 165 m upstream of the stop line. To better understand the existing traffic conditions, data were collected from this intersection during several weekdays of May, June and July 2009. Northbound traffic counts by vehicle type (car or truck), individual vehicle's length and distance headway, queue length, and lane change frequency were observed in each lane for each cycle.

3.2 Data collection

Data were collected during several days in May, June and July 2009 at the Huron Church-Tecumseh intersection. Northbound traffic counts (left turn, through and right turn) by vehicle type (car or truck) were collected in each lane, each phase and each cycle. Signal timing, vehicle gap, distance headway (spacing), queue length, queue spillback and mid-block traffic activities, e.g. lane changes, side encroachments, etc.

To observe the representative peak traffic volume, a long queue length and a high percentage of trucks, data were collected from 11:00 am-12:00 pm and 3:30 pm-4:30 pm in the weekdays. Data were collected under normal weather and daylight conditions.

To measure the vehicles length, gap (distance headway) between the vehicles and the queue length, a scale was drawn on the sidewalk of the intersection with marks every 1.52 m (5 ft) from the stop line. The observer counted the number and type of vehicles passing the intersection in each lane during the green interval and the number and type of vehicle queued in each lane during the red interval within a cycle.

At the same time the observer also measured the length of the maximum queue formed within the cycle using the marked scale on the sidewalk. The observer recorded the time when the maximum queue was formed within the cycle using a stop watch. A sample data sheet is enclosed in Appendix A-1.

3.2.1 Traffic counts

Only two types of vehicles, car and truck were observed in queue but other types of vehicles (e.g. school bus, motor cycle) were not observed in queue during the data collection periods at the intersection. The observed length of car at the intersection were 3.65 m, 4.26 m, 4.57 m, 4.87 m, 5.48 m and the observed length of trucks are 16.76 m, 21.34 m and 21.95 m. It was observed that the total volume was almost the same from 11 am-12 pm and 3:30 pm – 4:30 pm but the percentage of trucks was higher from 11 am - 12 pm in northbound traffic. Table 3-1 and Table 3-2 show the numbers of northbound cars and trucks in each lane of Huron Church Road for 30 cycles during 11 am-12 pm and 3:30 pm – 4:30 pm on June 5, 2009. A majority of trucks (78-83%) used the center through lane (Lane 2). Due to high percentage of trucks in Lane 2 (45-56%), queue length was longest in Lane 2 for most cycles. The percentage of trucks during this study period was 13.5-16% and about two-third of total vehicles were through vehicles in northbound traffic. Similar distributions of vehicles across lanes were also observed on the other weekdays during the study period.

Table 3-1: Summary of Traffic Counts (11 am-12 pm, June 5, 2009)

	Total		Left-turn Lane		Lane 3 (Through)		Lane 2 (Through)		Lane 1 (Through)		Right-turn Lane	
	C	T	C	T	C	T	C	T	C	T	C	T
Total arrival (%)	740 (84)	145 **(16)**	44 (96)	2 (4)	210 (97)	7 (3)	**90 (44)**	**114 (56)**	208 (95)	11 (5)	188 (95)	11 (5)
Total	885		46(6%)		217 (24%)		204 (23%)		219 (25%)		199 (22%)	
					Car - 508 (79%) Truck - 132 (21%) 640 (72 %)							
No. of stopped vehicles on red	183	17	38	2	25	0	12	12	57		51	3
	200		40		25		24		57		54	
No. of passing vehicles on green	557	128	6	0	185	7	78	102	151	11	137	8
Total	685		6		192		180		162		145	

Note: C = Car; T = Truck

Table 3-2: Summary of Traffic Counts (3:30 pm-4:30 pm, June 5, 2009)

	Total		Left-turn Lane		Lane 3 (Through)		Lane 2 (Through)		Lane 1 (Through)		Right-turn Lane	
	C	T	C	T	C	T	C	T	C	T	C	T
Total arrival (%)	768 (85)	120 **(13.5)**	35 (100)	0 (0)	245 (99)	3 (1)	**121 (55)**	**100 (45)**	180 (95)	9 (5)	187 (96)	8 (4)
Total	888		35(4%)		248 (28%)		221 (25%)		189 (21%)		195 (22%)	
					Car - 546 (83%) Truck - 112 (17%) 658 (74 %)							
No. of stopped vehicles on red	208	19	29	0	37	0	26	15	53	3	63	1
	227		29		37		41		56		64	
No. of passing vehicles on green	560	101	6	0	208	3	95	85	127	6	124	7
Total	661		6		211		180		133		131	

Note: C = Car; T = Truck

Since the traffic data were manually collected at the intersection, there were some possible errors associated with counting the number of queued vehicles, measuring the length of vehicles, the length of queue, and gap between the queued vehicles.

3.2.2 Lane change

The number of lane changes was also observed at two areas - the areas within 56 m (184 ft) and 122 m (400 ft) in advance of the stop line during the study period as shown in Table 3-3.The count of number of lane changes could be easily done manually in the field and there were a possibility of miscounting the number of lane changes but the total impact of such error was insignificant in our collected data. Within the area of 56 m and 122 m from the stop line, on average 30 and 74 lane changes (excluding lane changes by the vehicles entering or exiting the driveways) respectively occurred during different one-hour periods on different weekdays. This indicates that after vehicles pass the location closer to the stop line, they are less likely to change lanes.

TABLE 3-3: Number of Lane Changes

Distance from stop-line	Date	Time	Phase	No. of lane changes
56 m	July 17, 2009	5:00 pm - 6:00 pm	Red+ Green	59
56 m	July 21, 2009	12:30 pm -1:30 pm	Red+ Green	41
122 m	July 17, 2009	4:00 pm - 5:00 pm	Red+ Green	119
122 m	July 21, 2009	1:30 pm - 2:30 pm	Red+ Green	108

3.2.3 Mid-block driveways

There are two driveways for vehicles to enter or leave from the gas station located at the southeastern corner of Huron Church-Tecumseh intersection. The driveways are 30 m (99 ft) and 61 m (185 ft) upstream of the stop line. The width of the driveways is 14 m

(45 ft) and 15 m (50 ft). Another driveway to the American Plaza is located 165 m (540 ft) from the stop line as shown in Figure 3-1.Table 3-4 shows the total number of vehicles entering and exiting from the driveways at two locations 56 m and 122 m from the stop line.

Table 3-4: Number of Vehicles Entering and Exiting Driveway

Date	Time	No. of vehicles entering and exiting from the first driveway [56 m upstream of the stop line]	No. of vehicles entering and exiting from the first & second driveway [122 m upstream of the stop line]
July 17, 2009	5:00 pm - 6:00 pm	11	29
July 31, 2009	5-00 pm -6:00 pm.	7	24
July 21, 2009	12:30 pm -1:30 pm	8	21
August 4, 2009	12:30 pm -1:30 pm	11	14
July 17, 2009	4:00 pm -5:00 pm	9	45
July 31, 2009	4:00 pm -5:00 pm	8	28
August 4, 2009	12:30 pm -1:30 pm	5	18
July 21, 2009	1:30 pm - 2:30 pm	5	19

The number of vehicles entering and exiting the driveways was observed at two observation points - 56 m (185 ft) and 122 m (400 ft) upstream of the stop line. At 56 m from the stop line, the vehicles entering and exiting the driveway at 30 m from the stop line could not be counted. However, these "missing" vehicles were only 1.1% of total northbound vehicles. At 122 m from the stop line, the vehicles entering and exiting the two driveways at 30 m and 61 m from the stop line could not be counted. These missing vehicles were only 2.8% of total northbound vehicles. Thus, the vehicles entering or exiting the driveways are negligible for queue length estimation.

3.2.4 Queue spillback

It was observed that queue sometimes spills over the two potential locations of detectors- 56 m and 122 m from the stop line. As expected queue spillback occurs more frequently at 56 m from the stop line than 122 m from the stop line. Frequent queue spillback makes difficult to detect the number of queued vehicles beyond the location of detectors. Table 3-5 summarizes the results of field observation in each day.

It was observed that maximum queue length was longer than the distance between the stop line and observation point for 7 out of 30 cycles at 56 m from the stop line and 1.4 out of 30 cycles at 122 m from the stop line. This indicates that if detectors are located further away from the stop line, queue is less likely to pass detectors and the number of vehicles beyond the detector location is lower. Clearly, for more accurate estimation of queue length, detectors should be neither too close to the stop line (due to frequent queue spillback) nor too distant from the stop line (due to frequent lane changes).

3.2.5 Right-turn and left-turn lane

At the Huron Church–Tecumseh intersection, the right-turn lane with full width (3.66m) starts 164m from the stop line and the left-turn lane with full width start 56m from the stop line. If the location of data collection points is placed at or within 56 m it would be possible to capture all the through, left-turn and right-turn vehicles. On the other hand, if the data collection points are placed at 122m, left-turn vehicles cannot be captured because the left-turn starts beyond the data collection point.

TABLE 3-5: Number of Queue Spillback

Distance from the stop line	Date	Time	No. of queue spillback
56 m	June 30, 2009	12:30 pm -1:30 pm	7
	June 30, 2009	2:00 pm -3:00 pm	20
	July 01, 2009	11:30 pm - 12:30 pm	5
	July 01, 2009	12:30 pm -1:30 pm	5
	July 01, 2009	1:30 pm - 2:30 pm	2
	July 01, 2009	2:30 pm -3:30 pm	6
	July 21, 2009	12:00 pm - 1:00 pm	5
122 m	June 30, 2009	12:30 pm - 1:30 pm	1
	June 30, 2009	2:00 pm - 3:00 pm	2
	July 01, 2009	11:30 pm -12:30 pm	2
	July 01, 2009	12:30 pm - 1:30 pm	1
	July 01, 2009	1:30 pm - 2:30 pm	0
	July 01, 2009	2:30 pm - 3:30 pm	2
	July 21, 2009	12:00 pm - 1:00 pm	2

It is also observed in the study area that the length of both turning lanes is sufficient to store all queued vehicle since these lanes are normally used by the short vehicles (car).However, the long queue in lane 2 (which is generally created due to long vehicles) sometimes prevents from entering the left-turn lane.

3.2.6 Length of vehicles, gap between vehicles and queue length

It was observed that the length of cars was 3.7-5.5 m (12-18 ft) and the length of trucks was 16.8-21.9 m (55-72 ft). The average distance headway between vehicles was different for different vehicle types. The headways for car following car, truck following car, and truck following truck (car following truck was not observed during the study period) were 3.7 m, 4.6 m and 5.7 m, respectively. This indicates that trucks tend to maintain longer headway than cars, particularly when they follow other trucks. Due to longer length and headway of trucks, queue length longer when there are more trucks in a queue.

TABLE 3-6: Observed Queue Length by Vehicle Type

Vehicles in a queue	Observed queue length (ft)	Vehicles in a queue	Observed queue length (ft)
3 cars	70 – 80	1 truck + 2 cars	135
4 cars	**88 – 125**	1 truck + 5 cars	215
2 trucks	147 – 168	2 trucks + 1 car	200-205
3 trucks	200- 269	4 trucks + 1 car	385
4 trucks	**310 – 365**	5 trucks + 1 car	470
5 trucks	390 – 445		

3.2.7 Signal timing

The cycle length at the Huron Church-Tecumseh intersection is 120 seconds. Displayed green intervals are shown in Table 3-7. Intergreen period (red + amber) between phases are 3-4 sec. The effective green time is the time allocated for a given traffic movement (green plus yellow) at a signalized intersection less the start-up and clearance lost times for the movement (Kang, 2000). Highway Capacity Manual (2000) states the effective green time is equal to the actual (displayed) green time plus the change-and-clearance interval minus the lost time for the movement. The effective red interval is cycle length minus the effective green interval. The effective green time and effective red time were calculated using different parametric values available at the study site in the following equation developed by Mannering et al. (2009):

$$g = G + Y + AR - tL \qquad (3\text{-}1)$$

where g = effective green time (seconds); G = displayed green time (seconds); Y = displayed yellow time (seconds); AR = displayed all-red time (seconds); tL = total lost time (seconds) (i.e. sum of start-up lost time and clearance lost time).The effective green and red times are 43 and 77 seconds, respectively.

The same values of effective green time and effective red time were calculated using the Canadian Capacity Guide for Signalized Intersections (2008). Details calculation is shown in the Appendix B. Thus, the effective green interval is 1 second longer than the displayed green interval (42 sec).

Table 3-7: Duration of Green Intervals at Huron Church-Tecumseh Intersection (City of Windsor, 2009)

Phase	Lane	Green intervals (sec)
1	Eastbound Left	8
	Westbound Left	8
2	Eastbound Through	35
	Westbound Through	35

3.3 Estimation of queue length

Using the traffic count, gap and signal timing data, collected from the intersection, the queue length can be estimated as follows. The time period during which a queue formed "detector actuation time" (DAT) is estimated. The estimation of DAT is shown in Figure 3-2. DAT was found to be 77 seconds for northbound approach of the Huron Church Road-Tecumseh Road intersection. This is exactly equal to the effective red interval as calculated earlier.

DAT = Yellow interval + Red interval + Start-up lost time

```
| Yellow |               Red               |Start-up
                                          Lost time |          Green        |

   4 sec                71 sec               2 sec              42 sec

0        4                                 75        77                       120
|--------|---------------------------------|---------|------------------------|

|---------------------77 sec-----------------------|

|Start time --------------- DAT--------------- Stop time|
```

Figure 3-2: Estimation of Detector Actuation Time (DAT)

Since a majority of drivers (73%) stop during 4-sec yellow intervals at 70 km/h when they are 200 ft in advance of the stop line (Rakha, 2007), it was assumed that yellow interval is included in DAT.

After determining DAT, the maximum queue length formed in each lane can be estimated by using the observed number of queued cars and trucks, the observed length of vehicles and the length of gaps between queued vehicles during DAT as follows:

$$Q_{est} = N_C \times L_C + N_T \times L_T + (N - 1) \times \bar{h} \tag{3-2}$$

where, Q_{est} is estimated queue length; N_C is the number of cars passing the detector during DAT; N_T is the number of trucks passing the detector during DAT; L_C is the average length of car; L_T is the average length of truck; N is the total number of vehicle passing the detector during DAT and $\bar{h}$ is the weighted average headway between the vehicles in a queue.

The estimated queue length and field-measured queue length are compared in Table 3-8. It was observed that due to shorter length of car and shorter gap between the cars, the queue length formed by the same number of cars is shorter than that of trucks. The developed method can successfully estimate the queue length using the manually collected traffic data. The mean absolute percentage error and standard deviation of errors for the estimated queue length at the study site was 3.48% and 2.25% respectively.

In summary the preferred location of data collection points for each criterion is shown in Table 3-9. It is event from the observed results that there are advantages and disadvantages of selecting data collection points closer to the stop line or further away from the stop line for accurate estimation of queue length. More detailed method of determining optimal location of data collection points will be discussed in Chapter 5.

TABLE 3-8: Observed and Estimated Queue Length

Vehicles in queue	Observed queue length (ft)	Mean observed Queue length, Q_{obs} (ft)	Estimated Queue length, Q_{est} (ft)	Absolute Error (%) = $[(Q_{obs} - Q_{est}) / Q_{obs}] \times 100$
3 cars	70 – 80	75	72	4.00
4 cars	88 – 125	**106**	**100**	5.66
2 trucks	147 – 168	157	163	3.82
3 trucks	200- 269	235	254	8.08
4 trucks	310 – 365	**337**	**345**	2.36
5 trucks	390 – 445	417	436	4.55
1 tr.+ 2 cars	135	135	131	2.96
1 tr. +5 cars	215	215	215	0.00
2 trs.+ 1 cars	200-205	202	194	3.96
4 trs. +1 cars	385	385	376	2.33
5 trs.+ 1 cars	470	470	467	0.63
				Mean absolute error = 3.48% Standard Deviation = 2.25%

TABLE 3-9: Preferred Data Collection Points Location for Different Criteria

Criterion	Preferred location (distance from the stop-line)	Reasons
Number of lane changes	56 m	The number of lane changes in peak hour was substantially higher (about 1.5 to 2 times) within the area of 122 m than 56 m.
Mid-block driveways	56 m	The number of vehicles entering and exiting from the driveways is higher (about 3-5 times) at 122 m than 56 m.
Queue spillback	122 m	The number of queue spillback is higher (about 3 to 6 times) at 56 m than 122 m.
Right-turn and left-turn lane	56 m	Left-turn and right-turn vehicle can be captured at 56 m from the stop line but not 122 m from the stop line.
Types of vehicles and gap between vehicles	122 m	Due to high percentage of trucks and longer length and headway of trucks, queue length is longer and queue spillback occurs more frequently at 56 m from the stop line than 122 m.

CHAPTER IV

QUEUE LENGTH ESTIMATION MODEL

4.1 Shock wave method

Most conventional queue length estimation methods (e.g. input-output technique) cannot estimate the queue length if the queue spills over detectors. The advance loop detector is commonly used to detect the presence and passage of vehicles over a short segment of roadway. To overcome this limitation, Liu et al. (2009) developed a queue length estimation method using shock wave theory. For a given cycle at a typical signalized intersection, the following three traffic states exist: 1) normal traffic state: when the vehicles arrive the signalized intersection at normal speed; 2) queued traffic state: when the vehicles stop behind the stop line and form a queue during the red interval; and 3) saturation traffic state: when the queued vehicles are discharged at maximum flow rate during the subsequent green interval.

Each traffic state is characterized by flow and density as follows (Liu et al., 2009): 1) normal traffic state: q_a (arrival flow rate) and k_a (density of arrival flow); 2) queued traffic state: zero flow (due to stoppage of vehicles) and k_j (jam density or maximum density); and 3) saturation traffic state: q_m (maximum flow rate or capacity) and k_m (density at capacity). The three shock wave speeds are calculated using the following equations:

Queuing shock wave speed: $$v_1 = \frac{0 - q_a}{k_j - k_a} \tag{4-1}$$

Discharge shock wave speed: $$v_2 = \frac{q_m - 0}{k_m - k_j} \tag{4-2}$$

Departure shock wave speed: $v_3 = \dfrac{q_a - q_m}{k_a - k_m}$ (4-3)

Figure 4-1 shows the movements of these three shock waves in the fundamental diagram and the time-space diagram.

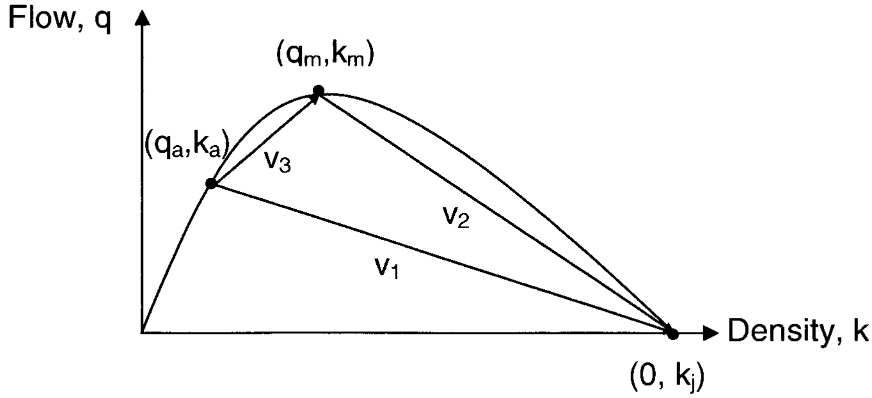
When these shock waves pass the detector upstream of the stop line, a significant change in speed, flow and occupancy (surrogate measure of density) can be observed at the location of detectors (Liu et al., 2009). The time when this change occurs is called "Break Point". Three Break Points (A, B and C) were defined as follows (refer to Figure 4-1(b)):

Break Point A: when the queuing shock wave (v_1) passes detectors;

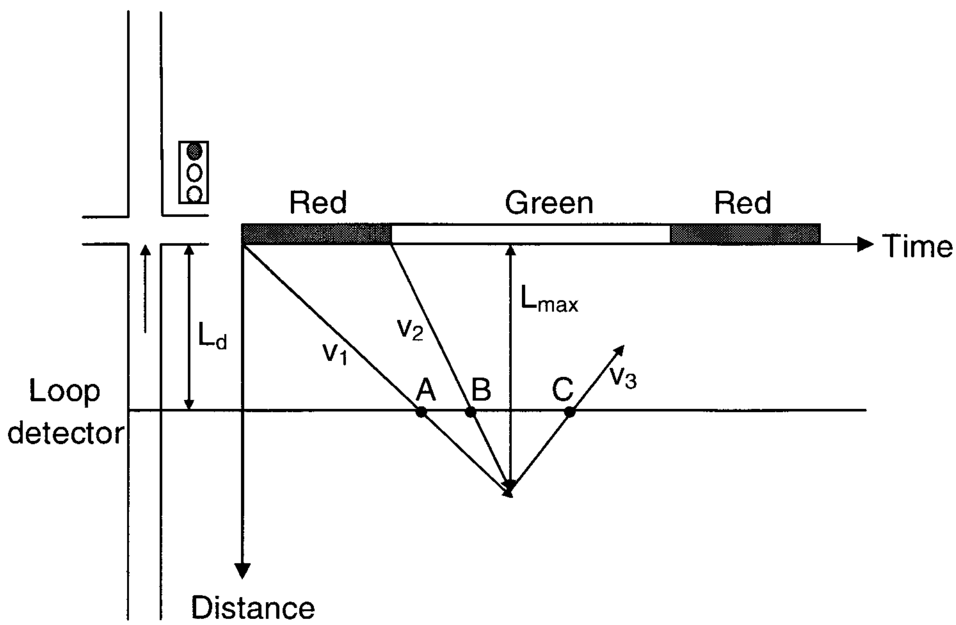
Break Point B: when the discharge shock wave (v_2) passes detectors;

Break Point C: when the departure shock wave (v_3) passes detectors.

Figure 2-5 shows that the occupancy times of the first 11 vehicles are generally low (less than 2.5 sec) but the occupancy time of the 12th vehicle suddenly increases to more than 45 sec when the vehicle completely stops at red. The point immediately before the occupancy time significantly increases is defined as the "Break Point A" and the corresponding time instant as T_A. After the occupancy time significantly increases and it drops after some time when the vehicles start moving over the detector. This changing point is defined as the "Break Point B" as shown in Figure 2-5 and the corresponding time instant as T_B.



(a) Fundamental diagram



(b) Time-space diagram

FIGURE 4-1: Shock waves at signalized intersections (Liu et al., 2009)

After the "Break Point B", the traffic flow condition becomes saturation flow state. After the queue clears, a large time gap between queue discharge flow and new arrival flow can be observed as shown in Figure 2-6. The time when this large gap occurs is called the "Break Point C" and the corresponding time instant is T_C.

After identifying the break points, the second-by-second occupancy data needs to process to find out the individual occupancy time of each vehicles and time gap between the two consecutive vehicles in the flow within the cycle. The time gap can be easily estimated by using the following headway relationship. Time gap between i^{th} and $(i+1)^{th}$ vehicle is $[t_{(i+1)} - (t_i + iocc_i)]$; where, $t_{(i+1)}$ is the time instants of the following vehicle and t_i is the time instant of preceding vehicle with occupancy time $iocc_i$

The flow and density of the saturation state and arrival state are then estimated using the average time gap and space mean speed of the two flows. Differences between the densities and flow rates provide the value of departure shockwave, v_3 (Equation 4-3). To estimate the shockwave, we will need to estimate two traffic states, i.e. saturation traffic state (q_m, k_m) and the arrival traffic state (q_a, k_a) before and after the "Break Point C". Flows and densities in these traffic states are estimated in the following steps:

Step 1: The spot speed (the speed of the vehicle at the designated point on the road) of the i^{th} vehicle (u_i) can be estimated by dividing the effective vehicle length by the occupancy time of the i^{th} vehicle, t_{oi} and is expressed as $u_i = L_e / t_{oi}$, (For example, if the effective vehicle length is 25 ft and the occupancy time is 0.436 s, u = 25 feet / 0.436 sec =57 feet/sec).

Step 2: The space mean speed, u_s (speed of the vehicles that traverse the same length of roadway) can be calculated by using the number of vehicles that arrive between Break Point B and Break Point C and their spot speeds (u_i). The space mean speed (u_s) for the n vehicles is expressed as $u_s = 1 / [1 / n \sum_{i=1}^{n} \frac{1}{u_i}]$. For example, if there are 7 vehicles with the spot speeds of 57, 59, 55, 50, 60, 57 and 56 feet / sec, u_s = 1 / [1/7 x (1/57 + 1/59 +1/55 + 1/58 + 1/60 + 1/57 + 1/56)] = 57.43 feet / sec. It should be noted that the occupancy time of individual vehicle passing the loop detector can be calculated using second-by-second traffic data. Effective vehicle length divided by the individual occupancy time equals the spot speed of each vehicle.

Step 3: The number of vehicles (n) arriving between Break Point B and Break Point C, and their individual time gaps h_i can be counted from the time gap profile shown in Figure 4-3.Using the time gaps of the vehicles in the saturation, the average flow (q_m) can be estimated. The estimated average flow (q_m) is expressed as $q_m = 1/ [1/n \sum_{i=1}^{n} \frac{1}{h_i}]$. For example, if time gaps are 2.34, 2.42, 3.39, 2.42, 2.38, 2.43 and 2.30 sec., q_m = 1/ [1/7 x (2.34 + 2.42 + 2.39 + 2.42 + 2.38 + 2.43 + 2.30)] = 0.42 veh /sec, i.e.1500 veh / hour.

Step 4: The density of flow (k_m) in the saturation state can easily be calculated using the equation, $k_m = q_m / u_s$. For example, k_m = 0.42 / 57.43) =0.0073 veh/ft or 38 veh/mile.

Step 5: Similarly, the flow and density of the new arrival flow (q_a, k_a) after Break Point C can be estimated using the occupancy time and the headway of the vehicles in that state from the occupancy profile and the time gap profile. The departure shockwave can be calculated using Equation 4-3.

To account for the variation in capacity (q_m) and critical density (k_m), discharge shock wave speed can alternatively be calculated based on the time of Break Points B

(T_B) identified from detector occupancy time and time gap profiles as follows (Liu et al., 2009):

$$v_2 = \frac{L_d}{T_B - T_r} \tag{4-4}$$

where L_d is the distance between the advance detector, T_B is the time instant when the discharge shockwave reaches the advance detector and T_r is the time of green start. However, since the variation in jam density due to change in truck percentage was not still considered, the calculation of queuing and discharge shock wave speed using Equation (4-1) and (4-2) will be subject to errors.

After determining v_2 and v_3, the maximum queue length for any given cycle (L_{max}) is calculated as follows (Liu et al., 2009):

$$L_{max} = L_d + \frac{T_C - T_B}{\left(\frac{1}{v_2} + \frac{1}{v_3}\right)} \tag{4-5}$$

In spite of strong theoretical background, the shock wave method cannot be used when the queue does not spill over detectors (called "short queue") since shock waves cannot be determined at detector location. Also, even if the queue spills over detectors (called "long queue"), it is difficult and cumbersome to determine Break Points B and C, and calculate discharge and departure shock wave speeds. The calculation time will drastically increase if the length of queue in multiple lanes should be estimated.

The shock wave method has a limitation when the queue length is estimated at signalized intersections with high truck volume such as the studied intersection. The method cannot account for the variation in flow-density relationship as the percentage of trucks changes. To evaluate the impact of truck percentage on flow-density relationship, hypothetical cases of vehicle mix in traffic flow were considered. In each case, it was assumed that there are two types of vehicles – cars and trucks – and percentage of trucks increases from 0% to 100% in a 10% increment. Since it is hard to generalize the impact of truck percentage on space mean speed, constant space mean speeds of 60 km/h and 40 km/h were assumed for the normal and saturation traffic states, respectively. Capacity and jam density were calculated based on length and distance headways of vehicles.

Since trucks are longer and have longer distance headway than cars, capacity and jam density are expected to be lower when percentage of trucks is higher. Clearly, flow-density relationship, and three shock wave speeds (v_1, v_2 and v_3) are affected by percentage of trucks as shown in the fundamental diagram (Figure 4-2). Table 4-1 shows that the speeds of backward-moving (negative) shock waves increase as the percentage of trucks increases. Sample calculation is shown in Appendix C and individual flow- density relationship is shown in Appendix D.

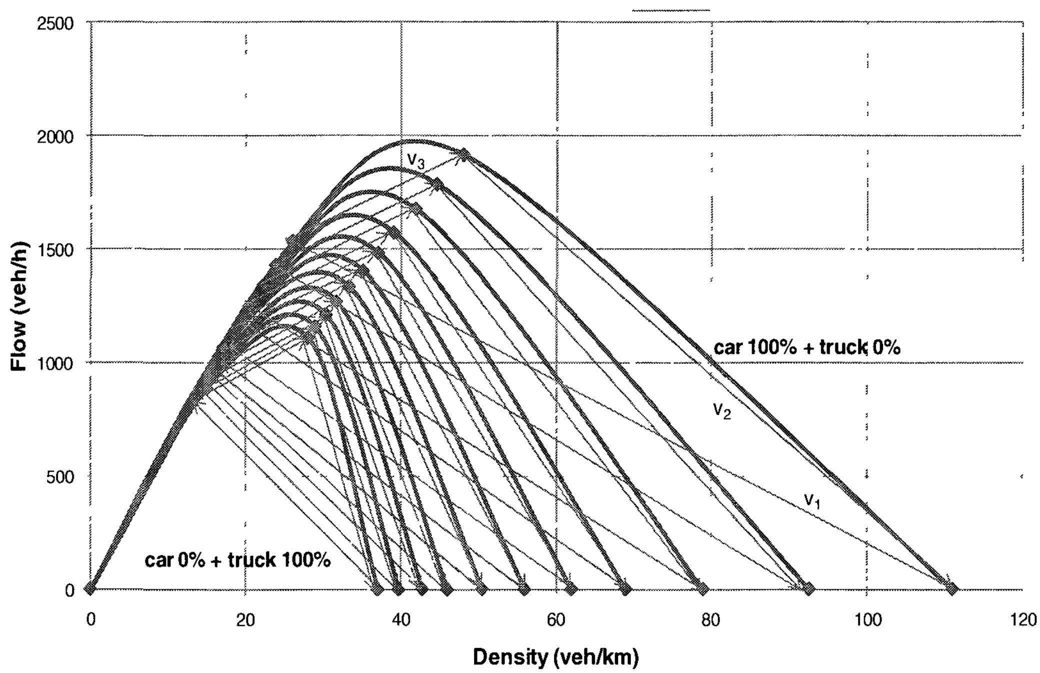


FIGURE 4-2: Fundamental diagrams for different percentage of trucks

Table 4-1: Shock Wave Speeds for Different Percentage of Trucks

Truck %	Jam density (vehicle/km)	Queuing shock wave speed (v_1) (m/sec)	Discharge shock wave speed (v_2) (m/sec)	Departure shock wave speed (v_3) (m/sec)
0	111	-5.0	-8.40	4.80
10	93	-5.95	-11.11	4.75
20	79	-6.50	-12.50	4.75
30	69	-7.30	-14.50	4.80
40	62	-7.80	-16.50	4.77
50	56	-8.30	-18.50	4.76
60	51	-8.88	-21.51	4.78
70	46	-9.70	-24.63	4.72
80	43	-10.10	-26.93	4.76
90	40	-10.59	-29.78	4.76
100	37	-11.2	-34.20	4.70

4.2 Simplified method

As explained in Chapter 3, a simple method (Equation 3-2) was developed to estimate the queue length in each lane at the intersection using the data manually collected from the field. The simplified method of queue length estimation can be applied to both short and long queue. Equation 3-2 is refined to estimate the maximum queue as follows:

$$L_{max} = L_f + N_C L_C + N_T L_T + (N-1)\bar{h} \tag{4-6}$$

where L_{max} = estimated maximum queue length; L_f = average distance between stop line and the front end of the first queued vehicle; N_C = the number of queued cars; N_T = the number of queued trucks; L_C = the average length of car; L_T = the average length of truck; N = the total number of vehicles in a queue; and $\bar{h}$ = the volume weighted average distance headway. Based on the field observation that headways are different for different types of vehicles, $\bar{h}$ is calculated using the following equation:

$$\bar{h} = \frac{N_{CC}h_{CC} + N_{TT}h_{TT} + N_{TC}h_{TC} + N_{CT}h_{CT}}{N} \tag{4-7}$$

where N_{CC} is the number of car following car; N_{TT} is the number of truck following truck; N_{TC} is the number of truck following car; N_{CT} is the number of car following truck; and h_{CC}, h_{TT}, h_{TC} and h_{CT} are headways between the vehicles in a queue. In this method, the distance headway is defined as the distance between the rear end of the lead vehicle to the front end of the following vehicle. The average length of car and truck, and headway between the consecutives queued vehicles were measured from the field observation.

Figure 4-3 illustrates the flow diagram of the simplified method for queue length estimation. First, traffic data are imported from detectors. If a queued vehicle occupies detectors, the queue is classified as a long queue. Otherwise, the queue is classified as a short queue. Second, for a long queue, Break Point C should be identified. Third, the number of vehicles for each type should be counted up to Break point C. On the other hand, for short queue, the number of vehicles for each type should be counted up to the effective red. Finally, the maximum queue length for a given cycle can be calculated using Equations 4-6 and 4-7.

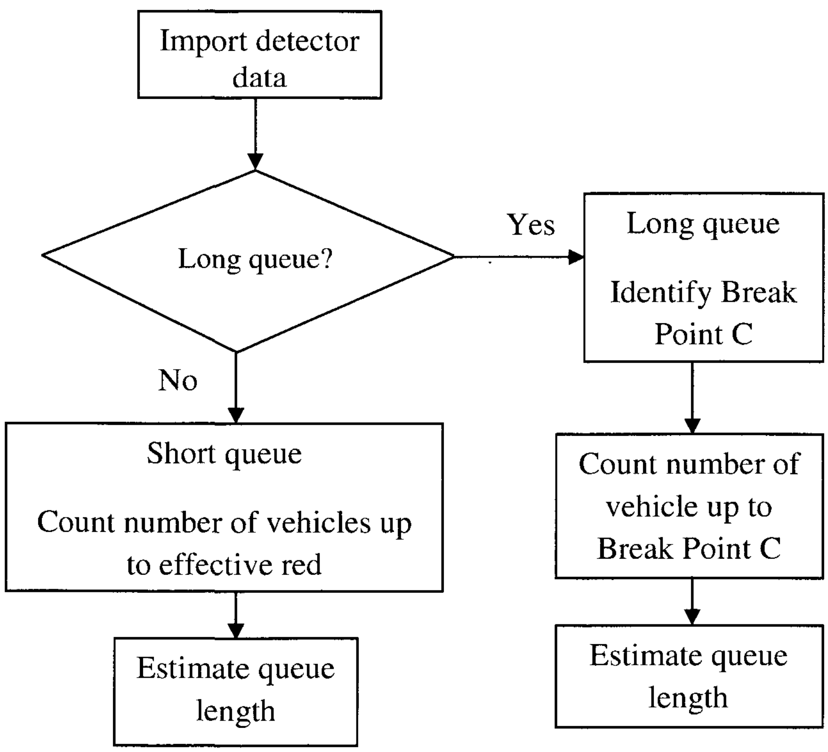


FIGURE 4-3: Flow Charts for Queue Length Estimation by Simplified Method

CHAPTER V

RESULTS AND DISCUSSION

5.1 Queue length analysis using simulation

To compare the accuracy of queue estimation between the two methods – the shock wave method and the simplified method – second-by-second detector data (occupancy and headways) and cycle-by-cycle queue length data are needed. However, these data could not be directly obtained from the study site due to absence of loop detectors. Thus, fictitious loop detectors (data collection points) were created and traffic data were collected from them using VISSIM 5.1 traffic simulation (PTV AG, 2008). The details of the simulation model are explained in the following subsections.

5.1.1 Overview of VISSIM

VISSIM is a microscopic step and driver behavior-based traffic simulation tool, employed widely by transportation modelers and researchers. VISSIM can model integrated roadway networks and various modes, including general-purpose traffic, buses, high-occupancy vehicles (HOV), light rail, trucks, bicyclists, and pedestrians (PTV AG, 2008). VISSIM can simulate networks of all sizes and all roadway functional classifications ranging from individual intersections to freeway and arterial corridors and entire metropolitan areas. It can model individual vehicle or pedestrian movements on a second or sub-second (up to one-tenth of a second) basis on a variety of constraints such as road geometry, vehicle characteristics, driving behavior, and traffic controls. VISSIM can track individual vehicle movements and accurately measure the physical length of a queue.

5.1.2 VISSIM network component and workflow

The VISSIM simulation network requires four major components: Network, Traffic, Control and Output (PTV AG, 2008). The "Network" represents the physical infrastructure of roadway and tracks. An intersection in VISSIM is modeled by a series of links and link connectors for through and turning movements. The "Traffic" represents the vehicular movements in the network such as speed distribution, vehicle model distribution, vehicle type and class, traffic composition and traffic demand. The "Control" defines the method of controlling conflict movements such as speed changes, traffic signals, and priority rules. The "Output" defines the types of results (e.g. travel time, delay) generated from the simulation. These four components are shown in Figure 5-1.

The queue length in northbound through lanes was only analyzed. However, to realistically reflect actual traffic conditions, traffic movements in all lanes and both directions on Huron Church Road and Tecumseh Road were also simulated.

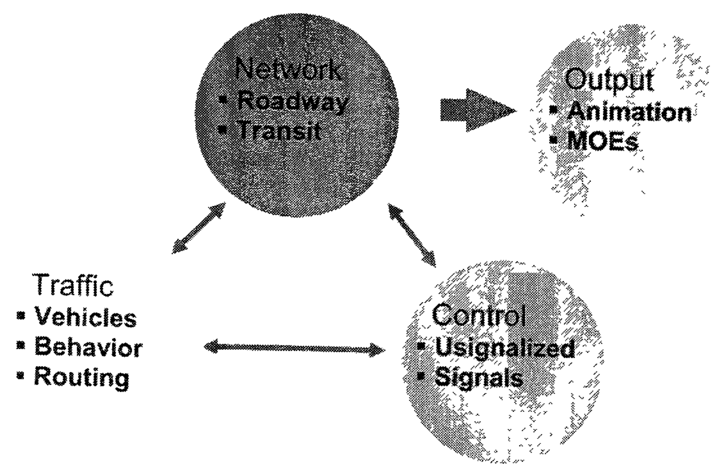


FIGURE 5-1 Components of VISSIM simulation

For the simulation purpose, the hourly traffic data collected at the Huron Church-Tecumseh intersection by the City of Windsor were first imported to VISSIM. But in the initial runs of the simulation, it was found that vehicle types were evenly distributed across the three through lanes. To reflect higher truck volume in the middle through lane (Lane 2) than the other two through lanes as observed in the field, the following adjustments were made.

First, all cars and trucks enter the same link and split into three northbound through lanes. Among the three lanes, the right through lane (Lane1) and the left through lane (Lane 3) were closed to trucks using link dialog "lane closure". This way, trucks are released to only the middle through lane (Lane 2) while cars are released to any lane. After trucks enter the road, trucks are still allowed to change lanes as they approach the stop line. After this adjustment, the proportions of trucks in the through lanes were similar to the observed proportions.

In the VISSIM network configuration three data collection points were placed in the three northbound through lanes at the same distance upstream of the stop line. Second-by-second speed, vehicle count by type of vehicle, and occupancy in each lane were obtained from the data collection points. To determine the number of lane changes in a given road section, the position of each vehicle in each time interval was extracted from the individual vehicle records generated by VISSIM. In the vehicle records, the position of the vehicle is described longitudinally (along the length of road section) and laterally (along the width of lane). Thus, the times when the lane change starts and ends can be determined when the vehicle starts leaving the current lane and reaches the center of the target lane, respectively. The example vehicle record generated by VISSIM simulation is shown in Table 5-1.

TABLE 5-1: Example Vehicle Records Generated by VISSIM for One Cycle

Time (sec)	Vehicle No.	Link No.	Lane No.	Longitudinal Position (x*) (m)	Longitudinal Position (x**) (m)	Lateral position (y) (m)	Vehicle type
3485.2	3551	13	1	57.1	149.9	0.97	Car
3485.4	3551	13	2	60.2	146.8	0.03	Car
3490.8	3551	13	2	134.7	72.3	0.97	Car
3491	3551	13	3	137.6	69.4	0.03	Car
3489.4	3555	13	2	71.3	135.7	0.03	Car
3489.6	3555	13	1	74.5	132.5	0.97	Car
3508.2	3563	13	3	253.5	-46.5	0.03	Car
3508.4	3563	13	2	256.5	-49.5	0.97	Car
3508.2	3563	13	3	253.5	-46.5	0.03	Car
3496.6	3564	13	1	61.4	145.6	0.97	Car
3498	3566	13	1	60.6	146.4	0.97	Car
3498.2	3566	13	2	63.9	143.1	0.03	Car
3496.4	3567	13	2	29.3	177.7	0.97	Car
3496.6	3567	13	3	32.3	174.7	0.03	Car
3521.6	3587	13	2	5.2	201.8	0.03	Car
3521.8	3587	13	1	6.2	200.8	0.97	Car
3530.6	3593	13	2	116.1	90.9	0.03	Car
3530.8	3593	13	1	118.4	88.6	0.97	Car
3528.8	3594	13	3	68.6	138.4	0.03	Car
3529	3594	13	2	71.6	135.4	0.97	Car
3531.8	3594	13	2	109	98	0.05	Car
3532	3594	13	1	110.9	96.1	0.99	Car
3529.8	3597	13	1	54.8	152.2	0.97	Car
3530	3597	13	2	57.6	149.4	0.03	Car
3533	3597	13	2	100.7	106.3	0.99	Car
3533.2	3597	13	3	103.7	103.3	0.05	Car
3528.2	3602	13	2	15.5	191.5	0.95	Car
3528.4	3602	13	3	18.6	188.4	0.01	Car
3533.4	3606	13	2	69	138	0.03	Car
3533.6	3606	13	1	71	136	0.97	Car

Note:

x* = distance from the starting end of link 13 (northbound Huron Church Road)
x** = distance from the stop line (stop line is 207 m upstream of the starting point of link 13)

TABLE 5-1: Example Vehicle Records Generated by VISSIM for One Cycle (Continued)

Time (sec)	Vehicle No.	Link No.	Lane No.	Longitudinal Position (x*) (m)	Longitudinal Position (x**) (m)	Lateral position (y) (m)	Vehicle type
3545.8	3616	13	1	70.9	136.1	0.97	Car
3546	3616	13	2	72.5	134.5	0.03	Car
3549	3616	13	2	96.1	110.9	0.99	Car
3549.2	3616	13	3	97.9	109.1	0.05	Car
3545.8	3617	13	1	50.3	156.7	0.97	Car
3546	3617	13	2	52.3	154.7	0.03	Car
3549.4	3617	13	2	81.9	125.1	0.97	Car
3549.6	3617	13	3	83.7	123.3	0.03	Car
3552.6	3632	13	2	4.8	202.2	0.03	Car
3552.8	3632	13	1	6.3	200.7	0.97	Car
3577.2	3637	13	3	105.4	101.6	0.03	Car
3577.4	3637	13	2	107.4	99.6	0.97	Car
3580.4	3648	13	2	125	82	0.97	Car
3580.6	3648	13	3	127.3	79.7	0.03	Car
3574.4	3648	13	3	61.9	145.1	0.03	Car
3574.6	3648	13	2	63.4	143.6	0.97	Car
3578.4	3650	13	1	126.3	80.7	0.97	Car
3578.6	3650	13	2	129	78	0.03	Car
3584.8	3651	13	1	177.7	29.3	0.97	Car
3585	3651	13	2	180.5	26.5	0.03	Car
3569	3652	13	2	11.8	195.2	0.03	Car
3569.2	3652	13	1	13.4	193.6	0.97	Car
3578.8	3664	13	1	21.3	185.7	0.97	Car
3579	3664	13	2	24.6	182.4	0.03	Car
3582	3664	13	2	72.7	134.3	0.97	Car
3582.2	3664	13	3	75.8	131.2	0.03	Car
3588	3664	13	3	162.2	44.8	0.03	Car
3588.2	3664	13	2	165.6	41.4	0.97	Car

Note:

x* = distance from the starting end of link 13 (northbound Huron Church Road)
x** = distance from the stop line (stop line is 207 m upstream of the starting point of link 13)

5.2 Determination of optimal location of detectors

Loop detectors in actual roads are represented by data collection points in the VISSIM network configuration. When vehicles pass data collection points in VISSIM, second-by-second speed, volume and occupancy data are automatically stored in a file.

Since the accuracy of queue length estimation depends on the location of data collection points, it is important to determine optimal location of data collection points such that the estimation error is minimized. In this study, a total of 7 locations – 40 m, 60 m, 80 m, 100 m, 120 m, 140 m and 160 m upstream of the stop line were considered. Data collection points were only created upstream of the stop line, but not on the stop line, because queue length estimation using two detectors was not as accurate as the estimation using only one detector (Sharma et al., 2007).

Given that queue length varies in each cycle, second-by-second simulated traffic data were collected from the data collection points for 30 cycles in a one-hour simulation period (cycle length of 120 seconds) using VISSIM. In every cycle, queue was fully discharged before the end of green interval and residual queue did not occur in the following cycle. Thus, the simulated flow is undersaturated for all cycles.

Since the queue length varies across lanes, the times of Break Points B and C for each lane also vary. These times can be identified from the individual occupancy time and time gaps in each lane separately (Liu et al., 2009). However, it was difficult to objectively determine Break Point C based on time gaps. Instead, these times were identified from the second-by-second vehicle count and occupancy. Break Point B occurs when occupancy suddenly drops after occupancy is 100% during red interval (i.e. vehicles are stopped). This is the time when the queue discharge starts. Break Point C occurs immediately before zero occupancy is observed for 2 or more consecutive seconds after Break Point B and then occupancy increases again (Liu et al., 2009). This is the time when the rear end of the last queued vehicles passes data collection points.

Since only the maximum of actual queue length in all lanes can be obtained from VISSIM, the queue length was only estimated in the lane where the maximum queue length occurs (called "the analysis lane"). The analysis lane was identified after the queue length was estimated in each lane based on the number of vehicles in a queue, distance headway and vehicle length using the simplified method. The length of car and truck was assumed to be 4.55 m and 22 m, respectively, based on the field observation. However, since it is complicated to differentiate distance headways between different types of vehicles in VISSIM, default constant headway of 2 m was used. The default distance between the stop line and the front end of the first queued vehicle is 1.2 m in VISSIM. It was observed that maximum queue length was more likely to occur in the center through lane (Lane 2 in Figure 1) due to high number of trucks in the lane. This is similar to the actual observed conditions at the study site.

Since the shock wave method can be used for long queue only, queue length was estimated only at the data collection points where the queue spillback occurs. For instance, if the estimated queue length in the analysis lane was 90 m, queue length was estimated at 40 m, 60 m and 80 m. Then the queue length was estimated for the analysis lane using the shock wave method and the simplified method, and compared with the actual queue length.

For example, sample simulated data were collected from data collection points 60 m upstream of the stop line for one 120-sec. cycle as shown in Table 5-2. In this cycle, Break Point A occurred at 31 sec. after red starts when the queued vehicle passed the detectors. Break Point B occurred at 82 sec. (= T_B) when the queue discharge started after the signal turned to green at 77 sec. (= T_r). Break Point C occurred at 91 sec. (= T_C) when the last vehicle in the queue passed the data collection points. This point was identified by the following zero occupancies for 3 seconds which represent time gap between queue discharge flow and new arrival flow. Individual vehicle data are summarized in Table 5-3. Maximum queue length was estimated using the shock wave method as follows:

1. Saturation traffic state

   Average time gap = (1.452 + 2.587) / 2 = 2.0195 sec

   q_m = 1 / Average time gap = 1 / 2.0195 = 0.495 veh/sec

   Average individual occupancy time/effective length =(0.065 + 0.0593)= 0.0622 sec/m

   u_m = 1 / [Average individual occupancy/effective length] = 1 / 0.0622 = 16.07 m/sec

   $k_m = q_m / u_m$ = 0.495 / 16.07 = 0.0308 veh/m

2. Normal traffic state

   Average time gap = (4.638 + 5.686 + 4.686 + 7.736) / 4 = 5.685 sec

   q_a = 1 / Average time gap = 1 / 5.685 = 0.175 veh/sec

   Average individual occupancy/effective length = (0.0494 + 0.0499 + 0.0415 + 0.0434) / 4 = 0.046 sec/m

   u_a = 1 / [Average individual occupancy/effective length] = 1 / 0.046 = 21.74 m / sec

   $k_a = q_a / u_a$ = 0.175 / 21.74 = 0.008 veh/m

3. Departure shock wave (v_3) can be calculated using Eq. (4-3) as follows:

$$v_3 = \frac{q_a - q_m}{k_a - k_m} = \frac{0.175 - 0.495}{0.008 - 0.0308} = 14.03 \text{ m/s}$$

4. Discharge shock wave (v_2) can be calculated using Eq. (4-4) as follows:

$$v_2 = \frac{L_d}{T_B - T_r} = \frac{60}{82 - 77} = 12 \text{ m/s}$$

5. Maximum queue length can be calculated using Eq. (4-5) as follows:

$$L_{max} = L_d + \frac{T_C - T_B}{\left(\frac{1}{v_2} + \frac{1}{v_3}\right)} = 60 + \frac{91 - 82}{\left(\frac{1}{12} + \frac{1}{14.03}\right)} = 118 \text{ m}$$

TABLE 5-2: Second-by-second VISSIM Simulated Data (Lane 2 at 60 m Upstream of Stop Line)

Time(sec)	car	Truck	Speed(km/h)	Occupancy (%)	Traffic state
1	0	0	0	0	Beginning of Red
2	0	1	42 4	50 4	
3	0	0	0	100	
4	0	0	0	58	
5	0	0	0	0	
.	.	.	.	.	
.	.	.	.	.	
22	1	0	52 8	30 8	
23	0	0	0	0	
24	0	0	0	0	
25	1	0	49	16 2	
26	0	0	0	18	
27	0	0	0	0	
.	.	.	.	.	
.	.	.	.	.	
31	**0**	**1**	**27**	**66.3**	**Break Point A**
32	0	0	0	100	
33	0	0	0	100	
34	0	0	0	90 8	
35	0	0	0	0	
.	.	.	.	.	
.	.	.	.	.	
40	0	1	12	2 2	Queued traffic state
41	0	0	0	100	
.	.	.	.	.	
.	.	.	.	.	
77	0	0	0	100	End of red
78	0	0	0	100	Beginning of green
79	0	0	0	100	
80	0	0	0	63 7	
81	0	0	0	0	
82	**0**	**1**	**27.1**	**34**	**Break point B**
83	0	0	0	100	Saturation traffic state
84	0	0	0	100	
85	0	0	0	20 8	
86	1	0	39 4	19 1	
87	0	0	0	22 2	
88	0	0	0	0	
89	1	0	42 1	29 9	
90	0	0	0	7 8	
91	**1**	**0**	**44**	**15.2**	**Break point C**
92	0	0	0	21	
93	0	0	0	0	
94	0	0	0	0	
95	0	0	0	0	
96	1	0	52 5	31 4	
97	0	0	0	0	
.	.	.	.	.	
.	.	.	.	.	
102	1	0	50 7	31 7	
103	0	0	0	0	Normal traffic state
.	.	.	.	.	
.	.	.	.	.	
107	1	0	61 9	26 4	
108	0	0	0	0	
.	.	.	.	.	
.	.	.	.	.	
115	1	0	59 5	18 7	
116	0	0	0	8 9	
117	0	0	0	0	
.	.	.	.	.	
.	.	.	.	.	
120	0	0	0	0	End of green

TABLE 5-3 Individual VISSIM Simulated Vehicle Data (Lane 2 at 60 m Upstream of Stop Line)

Time (sec)	Veh. No.	vehicle type	Individual Occupancy time (sec)	Time gap (sec)	Effective length (m)	Individual Occupancy time / Effective length	Traffic state
2	1	Truck	2.084	0	23.8	0.0875	
22	2	Car	0.308	17.916	6.35	0.0485	
25	3	Car	0.342	2.692	6.35	0.0538	Break Point A
31	4	Truck	3.571	5.658	23.8	0.15	Queued traffic state
40	5	Truck	39.659	5.429	23.8	1.6663	
82	6	Truck	2.548	2.341	23.8	0.107	Break Point B
86	7	Car	0.413	1.452	6.35	0.065	Saturation traffic state
89	8	Car	0.377	2.587	6.35	0.0593	
91	9	Car	0.362	1.623	6.35	0.057	Break Point C
96	10	Car	0.314	4.638	6.35	0.0494	Normal traffic state
102	11	Car	0.317	5.686	6.35	0.0499	
107	12	Car	0.264	4.683	6.35	0.0415	
115	13	Car	0.276	7.736	6.35	0.0434	

Maximum queue length can also be estimated using the simplified method. Since there are 5 cars and 4 trucks from the beginning of red interval to Break Point C (0-91 seconds), queue length is calculated using Eq. ((4-6) as follows:

$$L_{max} = L_f + N_C L_C + N_T L_T + (N-1)\bar{h} = 1.2 + 5(4.55) + 4(22) + (9-1)(2) = 128 \text{ m}$$

These estimated queue length, $(L_{max})_{est}$ was compared with actual observed queue length, $(L_{max})_{obs}$ and the estimation error was calculated using the following equation:

$$\varepsilon = \frac{\left|(L_{max})_{est} - (L_{max})_{obs}\right|}{(L_{max})_{obs}} \times 100 \tag{5-1}$$

The estimated queue length at 7 locations of data collection points using the two methods is shown in Tables 5-4 and 5-5. Queue length was estimated only in the analysis lane. However, queue length could not be estimated using the shock wave method for some cycles due to the following reasons:

(1) When the data collection points are too distant from the stop line, queue did not spill back over the data collection point and shock waves could not be detected at the location of data collection points. For this reason, when the data collection points were located 160 m from the stop line, the queue length was estimated for only 8~13 cycles out of 30 cycles.

(2) In some cycles, the flow and density of new arrival flow was greater than or equal to the flow and density of queue discharge flow. This results in a negative departure shock wave which contradicts the assumed flow-density relationship as shown in Figure 4-1. Consequently, the calculated queue length will be erroneous.

(3) When the time gap between the queue discharge flow and the new arrival flow was smaller or similar to the saturation time gap, Break Point C could not be identified (Liu et al., 2009). Consequently, departure shock wave (v_3) could not be determined.

Due to the first reason, too distant location of the data collection points from the stop line is not practical for queue length estimation. However, queue length could be estimated for more number of cycles using the simplified method than the shock wave speed for all data collection point locations. This is because the simplified method can avoid the error associated with the calculation of departure shock wave as explained in the second reason. The third reason applies to both methods since Break Point C must exist to estimate the queue length regardless of the queue estimation method.

TABLE 5-4 Estimated Queue Length Using Shock Wave Method (Car-truck mix)

Cycle	Actual Max. queue lane	40m	60m	80m	100m	120m	140m	160m	Average (%)
1	82 L3**	65.12 (20.57)*	71.15 (13.22)	80 (2.50)	a	a	a	a	72.09 (12.09)
2	133 L2	140.9 (5.99)	237.1 (78.29)	178.8 (34.49)	100 (24.81)	c	a	a	164.2 (35.87)
3	109 L2	90.9 (16.58)	132.6 (21.66)	109.5 (0.45)	100 (8.26)	a	a	a	108.25 (11.73)
4	95 L2	d	d	141 (48.49)	b	a	a	a	141 (48.49)
5	108 L3	43.65 (59.57)	64.6 (40.18)	93.52 (13.39)	100 (7.41)	a	a	a	75.44 (30.13)
6	113 L2	d	92 (18.50)	156.4 (38.45)	118.3 (4.66)	a	a	a	122.23 (20.53)
7	105 L2	247.08 (135.32)	165.7 (57.80)	141.6 (34.88)	129.9 (23.73)	a	a	a	171.07 (62.93)
8	133 L2	145.7 (9.55)	195.8 (47.24)	c	109 (17.99)	123. 9 (6.77)	a	a	150.16 (20.38)
9	132 L3	252.83 (91.53)	131.59 (0.30)	207 5 (57.17)	139.11 (5.38)	a	a	a	174.51 (38.59)
10	104 L2	176.3 (69.53)	135.9 (30.74)	140.3 (34.92)	120.72 (16.08)	a	a	a	143.30 (37.81)
11	165 L2	285.9 (74.37)	149.7 (8.70)	148.6 (9.41)	130.18 (20.61)	157 (4.23)	c	208.09 (26.88)	179.91 (24.03)
12	107 L2	92.11 (13.91)	128.4 (20.03)	127.2 (18.89)	100.6 (6.00)	a	a	a	112.07 (14.70)
13	184 L2	b	b	b	124.5 (32.34)	202.7 (10.16)	177.4 (3.56)	c	168.20 (15.35)
14	164 L2	112.72 (31.26)	114.85 (29.96)	183.5 (11.87)	138.4 (15.60)	153.3 (6.47)	142.7 (12.97)	206.53 (25.93)	150.28 (19.15)
15	177 L1	220.7 (24.70)	228.20 (28.92)	155.07 (12.38)	177.45 (0.25)	189.57 (7.10)	189.6 (7.13)	c	193.43 (24.7)
16	148 L2	b	2 03.8 (37.71)	d	197 (33.13)	141.3 (4.52)	176.83 (19.48)	a	171.71 (23.71)
17	145 L2	131.9 (9.01)	94.65 (34.71)	100.46 (30.71)	164.2 (13.27)	153.9 (6.13)	144.08 (0.63)	b	131.53 (15.74)
18	177 L2	105.65 (26.73)	113.7 35.76)	102 (42.31)	c	159.4 (9.95)	152 (4.06)	172.9 (2.29)	134.27 (20.18)
19	173 L2	70.4 (59.28)	115.76 (33.08)	104. 1 (39.81)	186.7 (7.91)	164.8 (4.69)	155 (10.39)	c	138.53 (25.86)
20	114 L2	143.9 (26.27)	241.3 (111.66)	234.1 (105.36)	a	a	a	a	206.43 (81.09)

TABLE 5-4 Estimated Queue Length Using Shock Wave Method (Car-truck mix) (Continued)

Cycle	Actual Max. queue lane	40m	60m	80m	100m	120m	140m	160m	Average (%)
21	155 L2	49 (68.27)	c	238.7 (54.02)	114.40 (26.19)	205.4 (32.51)	211.3 (36.32)	186.8 (20.56)	167.60 (39.64)
22	153 L2	d	c	d	180.8 (18.22)	171.5 (12.08)	c	a	176.15 (18.22)
23	123 L2	d	511.0 (315.45)	276.8 (125.06)	114.9 (6.50)	c	a	a	300.9 (149.0)
24	121 L2	d	d	143.1 (18.30)	134 (10.81)	134.86 (11.46)	a	a	137.32 (13.52)
28	185 L3	d	d	235.3 (27.19)	152.3 (17.68)	c	243.25 (31.49)	c	210 (27.19)
29	177 L2	343 (93.82)	334.36 (88.90)	106.9 (39.56)	208.8 (17.96)	144.8 (18.19)	150.9 (14.75)	195.4 (10.39)	211.99 (40.50)
30	127 L2	204 (60.65)	177.3 (39.64)	274.8 (116.37)	251.7 (98.22)	213.83 (68.37)	a	a	224.32 (76.65)
No. of cycle with the lowest error		3	3	3	10	5	3	3	
Average error (%)		47.21	52.02	39.83	18.83	14.47	14.08	17.21	
Standard Deviation (%)		35.75	65.98	33.93	19.46	17.20	11.94	10.61	

Note:
*: Estimation error
**: Lane with maximum queue length (analysis lane). Average error and standard deviation of errors are calculated in this lane only.

Reasons for missing data:
a: No queue spillback occurred.
b: Break point C could not be determined.
c: New arrival flow was greater than or equal to queue discharge flow ($q_a \geq q_m$).
d: Density of new arrival flow was higher than or equal to density of queue discharge arrival flow ($k_a > k_m$).

TABLE 5-5 Estimated Queue Length Using Simplified Method (Car-truck mix)

Cycle	Actual Max. queue lane	40m	60m	80m	100m	120m	140m	160m	Average (%)
1	82 L3**	75.6 (7.80)*	69.05 (15.79)	88.7 (8.17)	a	a	a	a	77.78 (10.58)
2	133 L2	147.6 (10.97)	158.5 (19.17)	141.05 (6.05)	114.85 (13.64)	138.85 (4.39)	a	a	140.17 (10.84)
3	109 L2	108.3 (0.64)	127.95 (17.38)	110.5 (1.37)	103.95 (4.63)	a	a	a	112.67 (6.0)
4	95 L2	106.15 (11.73)	99.6 (4.84)	123.6 (30.10)	b	a	a	a	109.78 (15.55)
5	108 L3	112.70 (4.35)	106.15 (1.71)	99.6 (7.77)	99.6 (7.77)	a	a	a	104.51 (5.4)
6	113 L2	112.7 (0.26)	108.35 (4.11)	132.35 (17.12)	119.5 (5.53)	a	a	a	118.22 (6.75)
7	105 L2	112.7 (7.33)	132.35 (26.04)	132.35 (26.04)	112.7 (7.33)	a	a	a	122.52 (16.68)
8	133 L2	167.25 (25.75)	130.15 (2.14)	136.7 (2.78)	136.7 (2.78)	149.8 (12.63)	a	a	144.12 (9.21)
9	132 L3	143.25 (8.52)	117.05 (11.32)	136.7 (3.56)	130.15 (1.40)	a	a	a	131.78 (6.2)
10	104 L2	123.6 (18.84)	147.6 (41.92)	99.60 (4.23)	154.15 (48.55)	a	a	a	131.23 (28.38)
11	165 L2	103.95 (37.0)	103.95 (37.0)	117.07 (29.06)	154.15 (6.57)	136.7 (17.15)	112.7 (31.69)	169.45 (2.69)	128.28 (23.02)
12	107 L2	79.95 (25.28)	99.6 (6.91)	112.7 (5.32)	119.25 (11.44)	a	a	a	102.87 (12.23)
13	184 L2	b	b	b	173.8 (5.54)	173.8 (5.54)	154.15 (16.22)	160.7 (12.66)	165.61 (9.99)
14	164 L2	121.40 (25.97)	113.23 (30.95)	182.5 (11.28)	145.4 (11.34)	145.4 (11.34)	175.95 (7.28)	206.5 (25.91)	155.76 (17.72)
15	177 L1	165.1 (6.72)	147.65 (16.58)	154.20 (12.88)	180.4 (1.92)	189.57 (7.10)	167.3 (5.48)	154.2 (12.88)	165.48 (9.08)
16	148 L2	b	169.45 (14.49)	176 (18.91)	162.9 (10.06)	136.7 (7.63)	136.7 (7.63)	a	156.35 (11.74)
17	145 L2	223.95 (54.44)	114.85 (20.79)	121.40 (16.27)	169.4 (16.82)	175.95 (21.34)	151.95 (4.79)	b	159.58 (22.4)
18	177 L2	97.40 (44.97)	121.4 (31.41)	127.95 (27.71)	134.5 (24.01)	208.7 (17.90)	202.15 (14.20)	226.15 (27.76)	159.75 (26.85)
19	173 L2	77.75 (55.05)	132.5 (23.24)	121.4 (29.82)	145.4 (15.95)	162.85 (5.86)	193.4 (11.79)	217.4 (25.66)	150.1 (23.91)
20	114 L2	117.05 (2.67)	123.6 (8.42)	99.6 (12.63)	a	a	a	a	113.41 (7.9)
21	155 L2	101.75 (34.35)	114.85 (25.90)	199.95 (29)	108.30 (30.12)	210.85 (36.03)	162.85 (5.06)	204.3 (31.80)	157.55 (27.46)
22	153 L2	136.70 (10.65)	125.8 (17.77)	143.25 (6.37)	130.15 (14.93)	106 (30.72)	173.8 (13.59)	a	135.95 (15.67
23	123 L2	136.70 (10.65)	167.25 (35.97)	123.6 (0.48)	130.15 (5.81)	121.45 (1.26)	a	a	135.83 (10.83)
24	121 L2	134.5 (11.15)	134.5 (11.15)	121.25 (0.20)	134.55 (12.40)	147.6 (21.98)	a	a	134.48 (11.37)

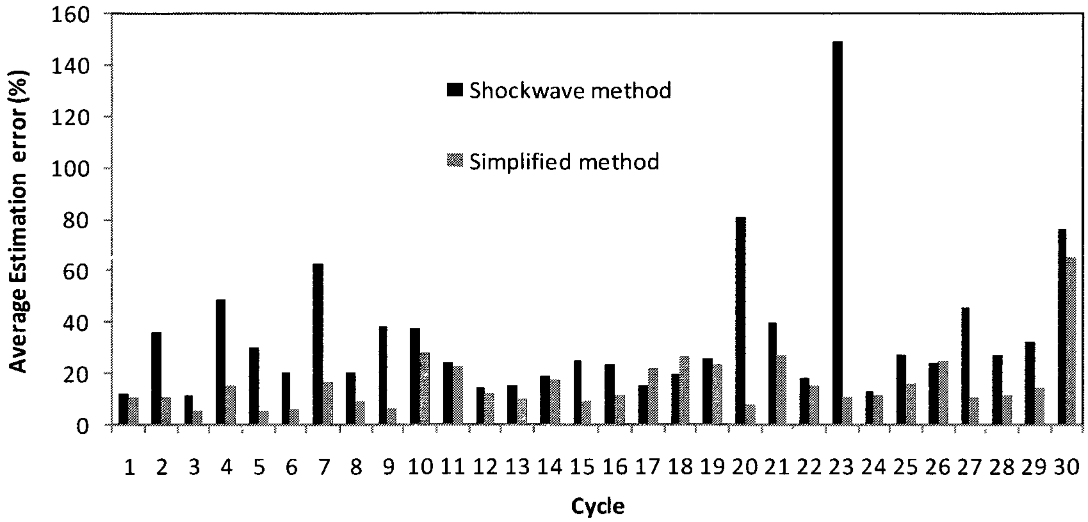
TABLE 5-5 Estimated Queue Length Using Simplified Method (Car-truck mix) (Continued)

Cycle	Actual Max. queue lane	40m	60m	80m	100m	120m	140m	160m	Average (%)
25	157 L2	130.15 (17.10)	169.45 (7.92)	82.15 (47.67)	158.55 (0.98)	195.65 (24.61)	162.9 (3.75)	176 (12.10)	153.55 (16.3.)
26	161 L2	189.05 (17.42)	189.05 (17.42)	189.05 (17.42)	202.15 (25.56)	175.95 (9.28)	219.6 (36.39)	243.6 (51.30)	201.20 (24.97)
27	124 L2	97.4 (21.45)	110.5 (10.88)	123.6 (0.32)	106.15 (14.39)	113 (8.87)	a	a	110.13 (11.18)
28	185 L3	182.55 (1.32)	176 (4.86)	213.10 (15.18)	206.55 (11.64)	169.45 (8.40)	217.45 (14.92)	136.70 (26.10)	185.97 (11.77
29	177 L2	165.05 (6.75)	171.6 (3.05)	110.5 (37.57)	171.6 (3.05)	141.05 (20.31)	147.6 (16.61)	171.6 (3.05)	151.23 (14.55)
30	127 L2	217.4 (71.18)	217.25 (71.18)	223.95 (76.33)	145.4 (14.48)	243.6 (91.81)	a	a	209.52 (64.99)
No. of cycle with the lowest error		5	4	5	7	4	4	1	
Average error (%)		19.65	18.62	17.29	12.17	18.2	13.52	21.08	
Standard deviation (%)		18.37	15.02	16.83	10.38	19.64	9.83	12.86	

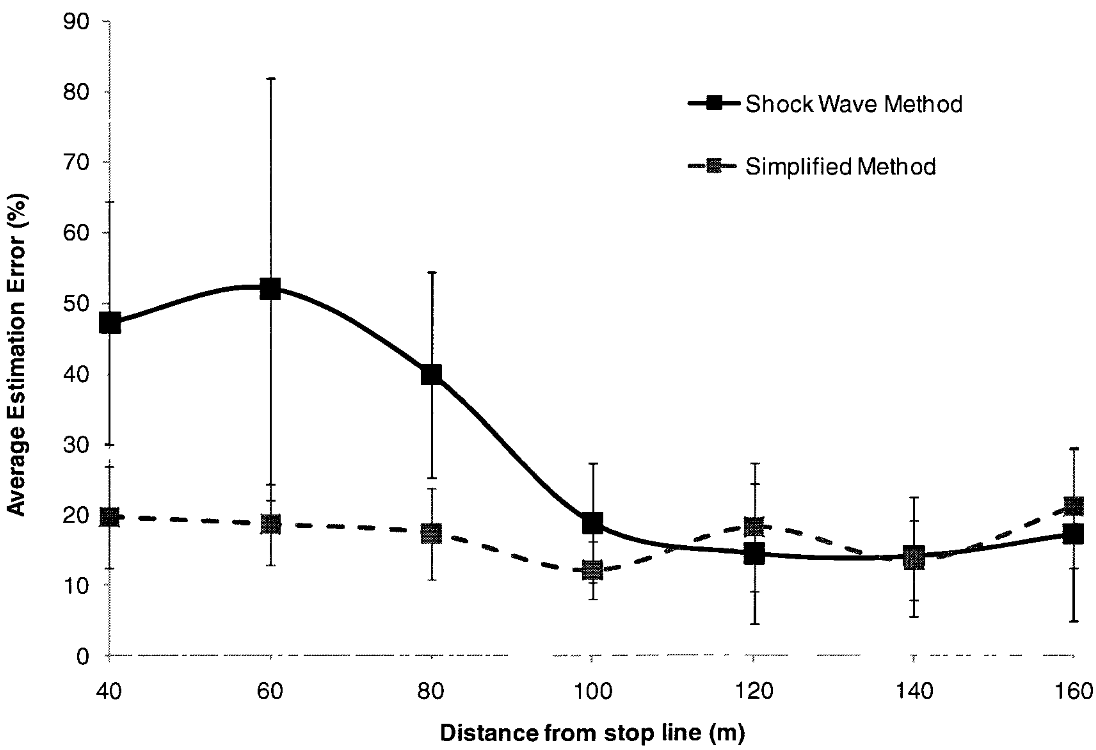
Note:
*: Estimation error
**: Lane with maximum queue length (analysis lane). Average error and standard deviation of errors are calculated in this lane only.

Reasons for missing data:
a: No queue spillback occurred.
b: Break point C could not be determined.

The average estimation errors in each cycle were generally lower for the simplified method than the shock wave method as shown in Figure 5-2(a). It was found that the average error per cycle was the lowest when the data collection points were located 140 m and 100 m from the stop line for the shock wave method and the simplified method, respectively. Figure 5-2(b) shows the average estimation error and standard error (95% confidence interval) at each distance from the stop line.



(a) Estimation errors for each cycle



(b) Average estimation errors at different data collection points' locations

FIGURE 5-2 Comparison of estimation errors between shock wave and simplified methods (Car-truck mix)

Standard error is defined as follows:

$$SE = t_{\alpha/2} \frac{s}{\sqrt{n}}$$

Where,

SE = standard error;

$t_{\alpha/2}$ = critical value of t statistics at the confidence level $(1-\alpha)$;

s = standard deviation;

n = number of samples.

It was found that average estimation error of the shock wave method gradually decreases as the data collection points are located further away from the stop line. This is partially because the number of samples is relatively lower for the location further away from the stop line and their variation is also smaller. On the other hand, the error of the simplified method is generally constant for all data collection point locations. Thus, the simplified method provides more reliable results regardless of distance from the stop line.

Although the average estimation error is the lowest at 100 m from the stop line for the simplified method, the errors are not significantly different among 7 locations of data collection points at a 95% confidence interval. This indicates that the optimal location for queue length estimation does not exist in this case.

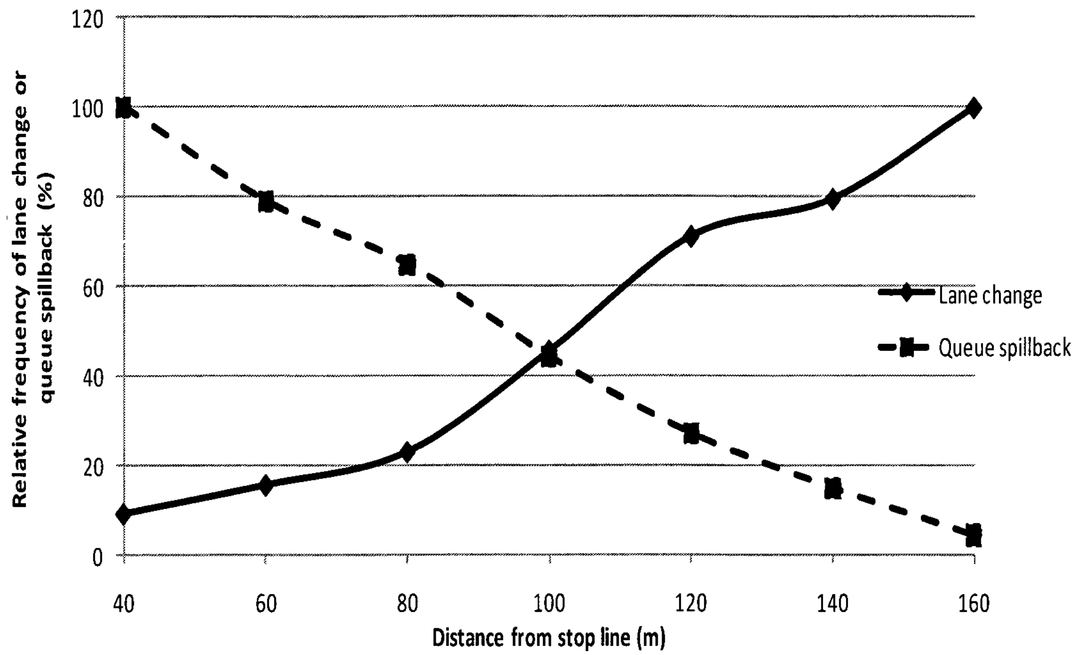


FIGURE 5-3 Relative frequencies of lane change and queue spillback at different data collection points (Car-truck mix)

Figure 5-3 shows that relative frequency of lane change increases and the relative frequency of queue spillback decreases as the detectors are further away from the stop line. It appears that as detectors are closer to the stop line, estimation errors of the shock wave method increases. This implies that shock waves speeds can be more accurately estimated at the locations closer to the end of actual queue. In other words, spatial variations in shock waves are less likely to be captured at the locations closer to the stop line.

5.3 Estimation of queue length (car only)

To estimate the effect of trucks on queue length estimation, the hypothetical case with cars only at the intersection was also considered. In this case, it was assumed that the same number of vehicles as the existing case with car-truck mix arrives the intersection. Unlike the existing case, cars were allowed to choose any lane of the northbound approach in the VISSIM simulation. Queue length was estimated at 6 locations of data collection points – 40 m, 60 m, 80 m, 100 m and 120 m - using the shock wave method and the simplified method as shown in Table 5-6 and Table 5-7. However, queue length could not be estimated using shockwave method for some cycles due to the same reasons described in section 5.2.

It was found that the average estimation errors in each cycle were also generally lower for the simplified method than the shock wave method as shown in Figure 5-4 (a). It was found that the average error per cycle was the lowest when the data collection points were located 80 m and 40 m from the stop line for the shock wave method and the simplified method, respectively. On the other hand, it was found that the standard deviation was the lowest when the data collection points were located 60 m and 100 m from the stop line for the shock wave method and the simplified method, respectively.

Figure 5-4(b) shows that average estimation error and standard error (95% confidence interval) for the shock wave method fluctuate as the data collection points are located further away from the stop line, whereas the error and standard error for the simplified method are almost constant for all data collection point locations. Thus, the simplified method also provides more reliable results regardless of data collection point locations and vehicle mix. Similar to the car-truck case, there was no significant difference in average estimation error among different data collection point locations at a 95% confidence interval.

TABLE 5-6 Estimated Queue Length Using Shock Wave Method (Car Only)

Cycle	Actual queue length (m)	40m	60m	80m	100m	120m	Average error (%)
1	46 L3**	76.86 (67.10)*	a	a	a	a	76.86 (67.10)
2	71 L3	74.18 (4.49)	c	a	a	a	74.18 (4.49)
3	79 L3	83.71 (5.96)	84.86 (7.42)	c	a	a	84.28 (6.69)
4	86 L3	163.27 (89.85)	105.35 (22.50)	c	a	a	134.31 (56.17)
5	76 L2	b	67.81 (10.76)	a	a	a	67.81 (10.76)
6	106 L2	115.37 (8.84)	128.95 (21.65)	b	140.26 (32.32)	129.18 (21.86)	128.44 (21.16)
7	79 L2	100.65 (27.41)	68.88 (12.80)	84.30 (6.71)	a	a	84.61 (15.64)
8	78 L2	108.13 (38.62)	87.54 (12.23)	87.98 (12.79)	a	a	94.55 (21.21)
9	94 L2	118.26 (25.8)	89.05 (5.25)	105.85 (12.61)	a	a	104.38 (14.55)
10	98 L3	143.58 (46.51)	118.37 (20.79)	96.38 (1.64)	a	a	119.44 (22.98)
11	86 L3	122.83 (42.83)	102.86 (19.60)	87.97 (2.29)	a	a	104.55 (21.57)
12	92 L1	b	130.37 (41.71)	138.98 (51.07)	a	a	134.67 (46.39)
13	92 L1	b	111.18 (20.84)	91.27 (0.79)	a	a	101.22 (10.81)
14	80 L2	124.85 (56.06)	86.69 (8.36)	87.65 (9.57)	a	a	99.73 (24.66)
15	121 L3	279.75 (131.20)	160.07 (32.29)	153.84 (27.14)	132.92 (9.85)	a	181.64 (50.12)
Average error (%)		45.39	18.16	13.84	21.08	21.86	
Standard deviation (%)		37.23	10.4	16.15	15.88	-	

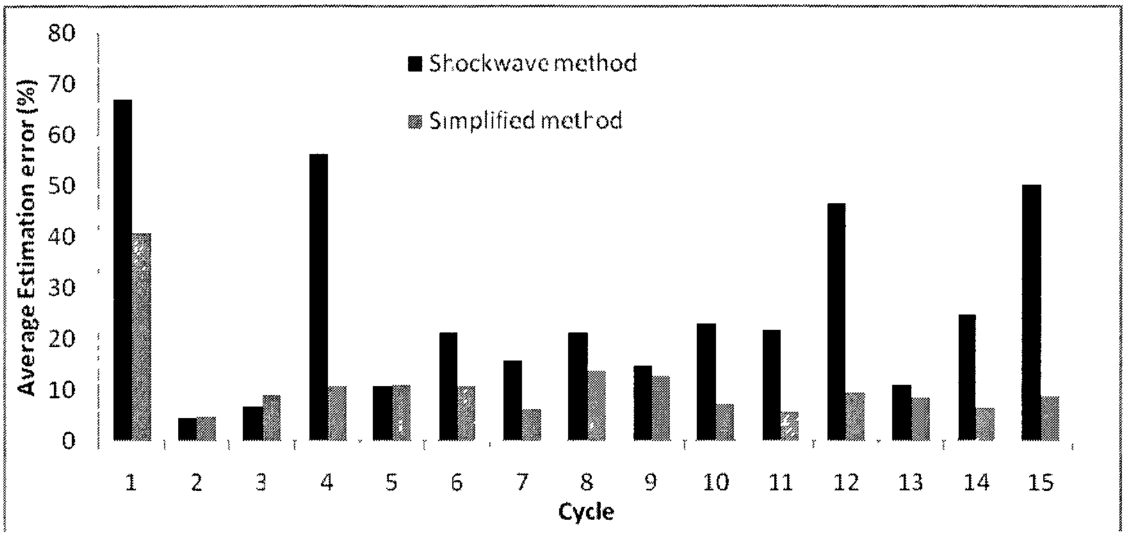
Note:
*: Estimation error
**: Lane with maximum queue length (analysis lane). Average error and standard deviation of errors are calculated in this lane only.
Reasons for missing data:
a: No queue spillback occurred
b: Break point C could not be determined
c: New arrival flow was greater than or equal to queue discharge flow ($q_a \geq q_m$)

TABLE 5-7 Estimated Queue Length Using Simplified Method (Car only)

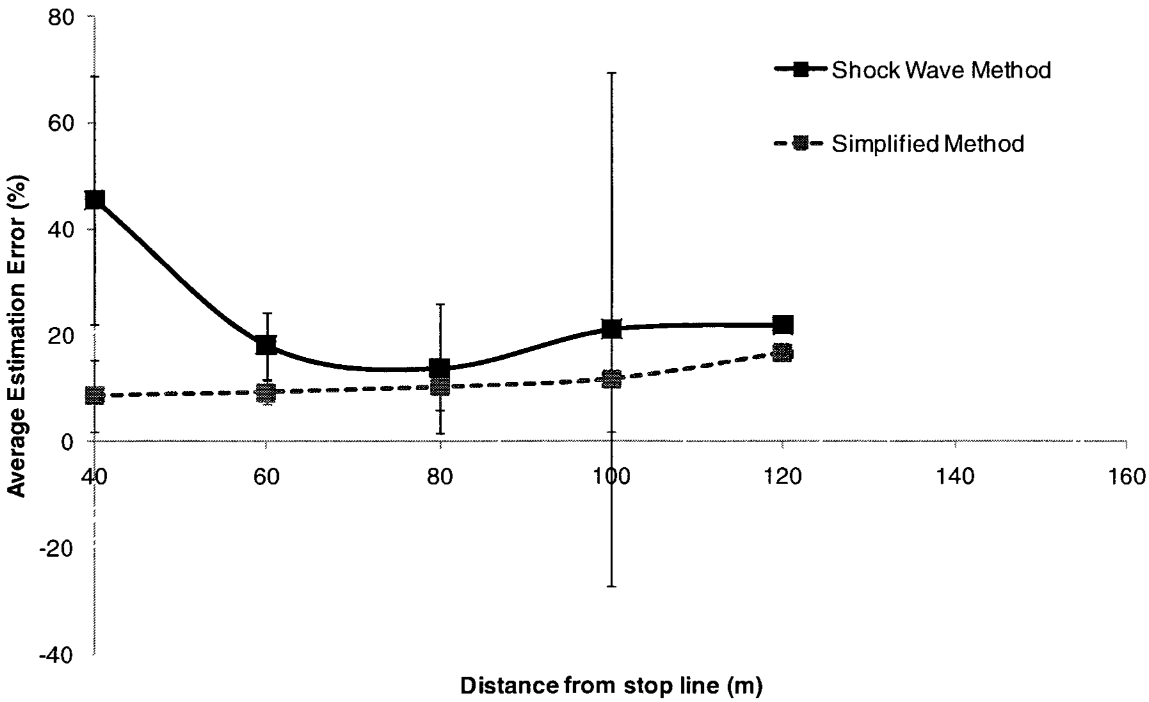
Cycle	Actual (m)	40m	60m	80m	100m	120m	Average error (%)
1	46 L3**	64.7 (40.65)*	a	a	a	a	64.7 (40.65)
2	71 L3	64.7 (8.87)	71.25 (0.35)	a	a	a	67.97 (4.61)
3	79 L3	77.8 (1.51)	71.25 (9.81)	90.9 (15.06)	a	a	79.98 (8.79)
4	86 L3	97.45 (13.31)	97.45 (13.31)	90.9 (5.69)	a	a	95.26 (10.77)
5	76 L2	b	84.35 (10.98)	a	a	a	84.35 (10.98)
6	106 L2	97.45 (8.06)	97.45 (8.06)	b	117.1 (9.47)	123.65 (16.65)	108.91 (10.56)
7	79 L2	77.8 (1.52)	71.25 (9.81)	84.35 (6.77)	a	a	77.8 (6.03)
8	78 L2	84.35 (8.14)	84.35 (8.14)	97.45 (24.93)	a	a	88.71 (13.73)
9	94 L2	97.45 (3.67)	77.8 (17.23)	77.8 (17.23)	a	a	84.35 (12.71)
10	98 L3	90.9 (7.24)	90.9 (7.24)	90.9 (7.24)	a	a	90.9 (7.24)
11	86 L3	90.9 (5.69)	90.9 (5.69)	90.9 (5.69)	a	a	90.9 (5.69)
12	92 L1	b	104 (13.04)	97.45 (5.92)	a	a	100.72 (9.48)
13	92 L1	b	84.35 (8.31)	84.35 (8.31)	a	a	84.35 (8.31)
14	80 L2	77.8 (2.75)	71.25 (10.93)	84.35 (5.43)	a	a	77.8 (6.37)
15	121 L3	123.65 (2.19)	130.2 (7.60)	134.45 (11.11)	104 (14.04)	a	123.07 (8.73)
Average error (%)		8.62	9.32	10.30	11.75	16.65	
Standard Deviation (%)		10.7	3.95	6.29	3.23	-	

Note:
*: Estimation error
**: Lane with maximum queue length (analysis lane). Average error and standard deviation of errors are calculated in this lane only.

Reasons for missing data:
a: No queue spillback occurred
b: Break point C could not be determined



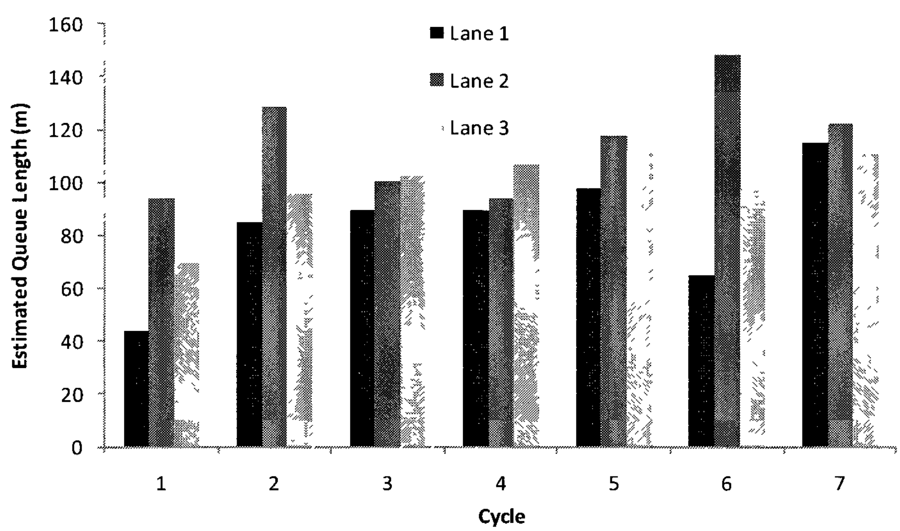
(a) Estimation errors for each cycle



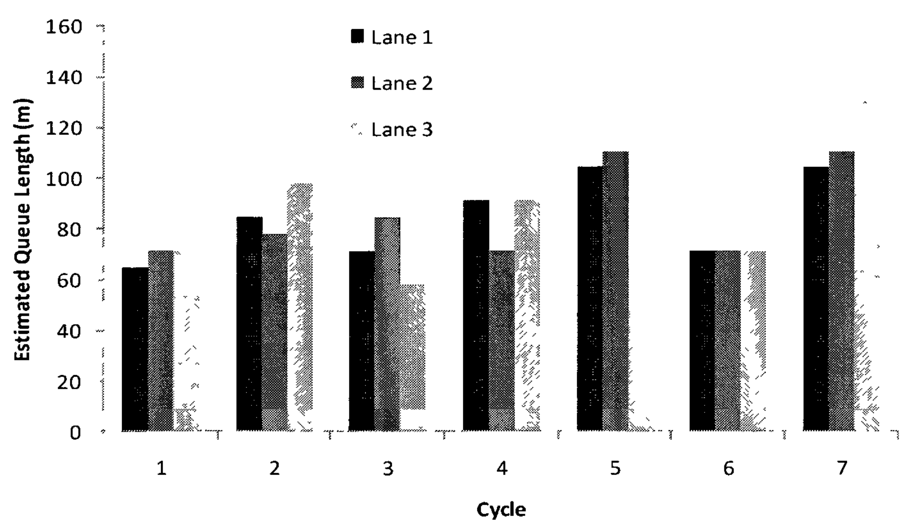
(b) Average estimation errors at different data collection points

FIGURE 5-4 Comparison of estimation errors between shock wave and simplified method (car only)

It is evident from the Figure 5-2(b) and Figure 5-4(b) that the estimated error is significantly lower for the flow with car only than that of mixed flow of cars and trucks when the simplified method is used. This may be because the variation in estimated queue length across the lanes is higher for the car-truck mix flow than car-only flow as shown in the Figure 5-4.



(a) Car-truck mix



(b) Car only

FIGURE 5-5: Distribution of queue length across lanes

5.4 Estimation of queue length (Short queue)

In the previous sections, queue length was estimated only when the queue spills over the detectors (i.e. long queue). However, the simplified method can also be applied to estimate queue length even when the queue does not spill over the detectors (i.e. short queue). As explained in Section 3.3, the number of queued vehicles is counted up to the effective red (= 77 sec.) and then estimated the queue length using Equation 4-6. The estimated queue length at 7 locations of data collection points using the simplified method for the mixed flow of cars and trucks is shown in Table 5-8. From the Figure 5-6 (b), it was found that the simplified method also estimated the queue length for short queue at a similar level of accuracy for long queue. Again, difference in errors was not significant among different data collection point locations at a 95% confidence interval.

TABLE 5-8 Estimated Queue Length Using Simplified Method (Short Queue)

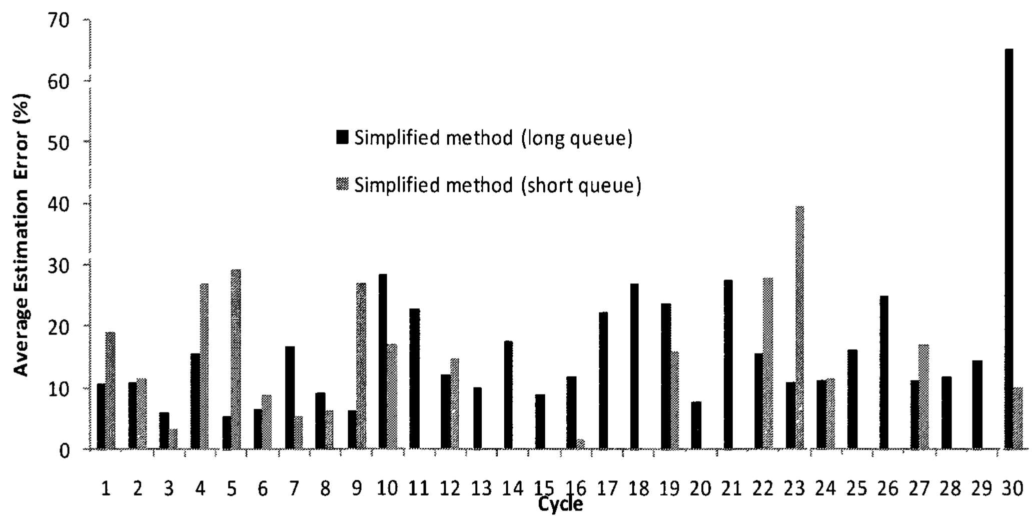
Cycle	Actual (m)	40m	60m	80m	100m	120m	140m	160m	Average (%)
1	82 L3**				75.6 (7.80)*	70.96 (13.46)	67.05 (18.25)	51.6 (37.07)	66.30 (19.14)
2	133 L2						138.85 (4.39)	108.3 (18.57)	123.57 (11.48)
3	109 L2					103.95 (4.63)	103.95 (4.63)	110.50 (1.37)	106.13 (3.54)
4	95 L2					112.7 (18.63)	119.25 (25.52)	130.15 (37)	120.7 (27.05)
5	108 L3					93.05 (13.84)	103.95 (3.95)	31.95 (70.41)	104.51 (29.4)
6	113 L2					119 (5.53)	112.7 (0.26)	136.7 (20.97)	122.8 (8.92)
7	105 L2					101.8 (3.04)	119.25 (13.57)	104.95 (.05)	108.66 (5.55)
8	133 L2						121.05 (8.98)	127.95 (3.79)	124.5 (6.38)
9	132 L3					99.6 (24.54)	86.15 (34.73)	95.25 (27.84)	93.66 (29.03)
10	104 L2					112.7 (8.36)	127.15 (22.25)	125.8 (20.96)	121.88 (17.19)

TABLE 5-8 Estimated Queue Length Using Simplified Method (Short Queue) (Continued)

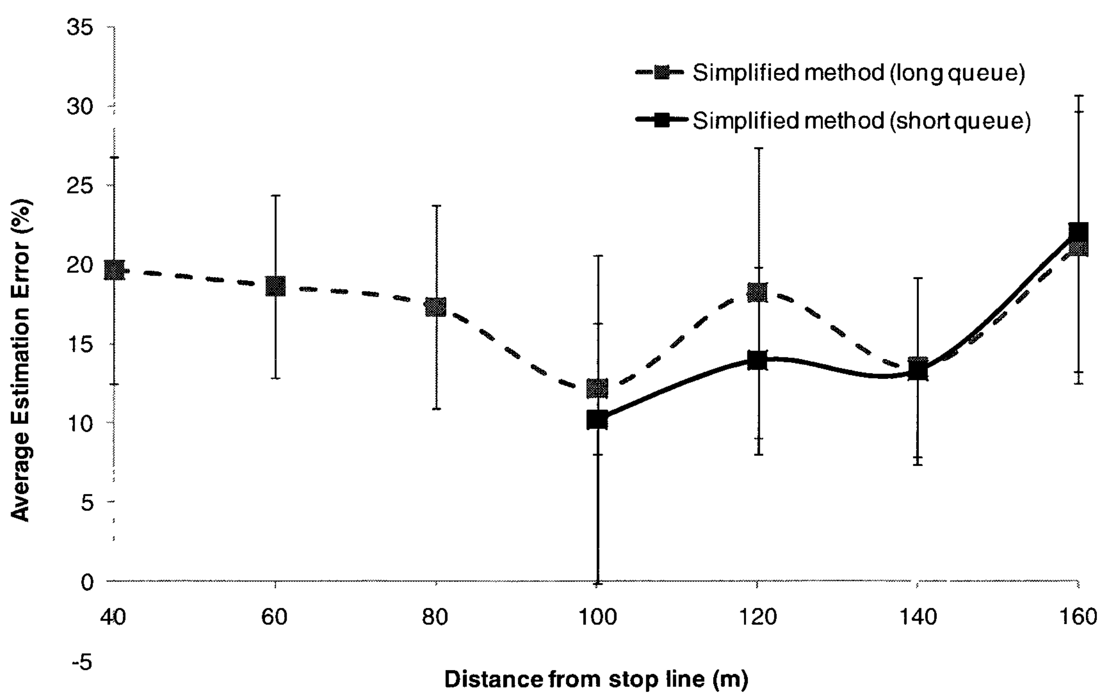
Cycle	Actual (m)	40m	60m	80m	100m	120m	140m	160m	Average (%)
11	165 L2								
12	107 L2					82.15 (23.22)	108.38 (1.26)	128 (19.62)	106.17 (14.7)
13	184 L2								
14	164 L2								
15	177 L1								
16	148 L2							145.4 (1.75)	145.4 (1.75)
17	145 L2								
18	177 L2								
19	173 L2								
20	114 L2				99.6 (12.6)	86.5 (24.12)	99.6 (12.63)	130.15 (14.16)	103.96 (15.88)
21	155 L2								
22	153 L2							110.5 (27.77)	110.5 (27.77)
23	123 L2						79.95 (35)	69.05 (43.86)	74.5 (39.43)
24	121 L2						135.7 (12.14)	134.5 (11.15)	135.1 (11.64)
25	157 L2								
26	161 L2								
27	124 L2						112.7 (9.11)	93.05 (24.95)	102.87 (17.03)
28	185 L3								
29	177 L2								
30	127 L2						134.5 (5.90)	145.4 (14.48)	139.95 (10.19)
Average Error (%)					10.21	13.93	13.29	21.98	
Standard Deviation (%)					3.39	8.37	11.05	17.58	

*: Estimation error

**: Lane with maximum queue length (analysis lane). Average error and standard deviation of errors are calculated in this lane only.



(a) Estimation errors for each cycle



(b) Average estimation errors at different data collection points

FIGURE 5-6 Comparison of estimation errors between long queue and short queue using simplified method

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

This study develops a method for estimating queue length at a signalized intersection using traffic data collected from loop detectors (data collection points) upstream of the stop line. The proposed method considers the variations in length of vehicles and queue length across lanes, since queue length is affected by longer length of trucks and different number of vehicles in each lane. It can estimate the queue length when the queue spills over detectors similar to the queue length estimation method developed using shock wave theory. In particular, the method simplifies the procedure of queue length estimation without shock wave calculation. Maximum queue length among lanes was estimated using this simplified method and the shock wave method at 7 different locations of detectors (data collection points) upstream of the stop line. As a result of the analysis, the study found the following:

1. The estimation errors vary at different detector locations. This is because as detectors are closer to the stop line, lane change occurs less frequently but queue spills over detectors more frequently.

2. The estimation errors of the simplified method were generally lower than the errors of the shock wave method in each cycle. Although there exists a location of data collection point with the lowest error, the errors were not significantly different among different locations.

3. The average errors and standard errors were generally constant at all data collection point locations. This implies that the simplified method can estimate queue length at similar level of accuracy regardless of detector locations.

4. The estimation error and standard error of the shock wave method was lower as the detector location is further away from the stop line. This indicates that shock wave speeds need to be estimated at the location closer to the end of the queue for more accurate estimation of queue length. However, the accuracy of the shock wave method can be improved by installing detectors at more distant location from the stop line as long as the queue spills over detectors.

5. The simplified method can also estimate queue length even when the queue does not spill over detectors (i.e. short queue) at similar level of accuracy as long queue. Thus, the method can be applied to queue length estimation regardless of queue spillback.

6. The accuracy of queue length estimation is lower for mixed flow of cars and trucks compared to car-only flow. This is because the variation in queue length across lanes is greater for mixed flow than car-only flow and potential error with identifying the lane with maximum queue length has greater impact on the accuracy for mixed flow.

Based on the findings, it is recommended that the simplified method be applied to the queue length estimation at signalized intersections since the calculation is simpler and accuracy is more reliable at any detector location and regardless of queue spillback.

There are some limitations of the simplified method. The method cannot estimate the queue length if there is no clear distinction between queue discharge flow and new arrival flow similar to the shock wave method. Also, the method cannot capture the vehicles that pass detectors immediately before the end of the previous cycle but did not pass through the intersection. This phenomenon frequently occurs for the vehicles in the dilemma zone. This error can be reduced by starting vehicle counts a few seconds before the beginning of red interval. However, it is unclear how to determine this start time of vehicle counts in each cycle. Similarly, the method cannot be applied to the oversaturated conditions when not all queued vehicles are cleared by the end of green interval. Clearly, the length of these "residual" queues from the previous cycle should be estimated for more accurate estimation of queue length in the current cycle.

In future studies, the method should be evaluated using the actual observed data collected from the intersection. The method should also be applied to the queue length estimation for left-turn and right-turn lanes to understand how different signal timing plan (protected or permissive) affect queue length.

The proposed method can be applied to estimate queue length at any signalized intersections using second-by-second traffic data collected from upstream of the stop line. Although it was found that the optimal location of data collection point did not exist at the studied intersection, it may exist at the other intersections. Then the proposed method can help decision makers to determine suitable location of detectors for accurate estimation of queue length.

APPENDICES

APPENDIX A: Sample data sheet

Table A-1: Sample Data Sheet

Cycle-by-cycle traffic counts; Huron Church -Tecumseh Intersection (northbound)

Date: Friday, June 6, 2009 Time: 11am.-12pm.

Cycl	Total	Total		L-T Lane		Lane 3		Lane 2		Lane 1		R-T Lane	
		C	T	C	T	C	T	C	T	C	T	C	T
1 R	4			1				1		1		1	
G	22	16	6			6		3	6	3		4	
2 R	4	3	1	2					1	1			
G	23	21	2	1		8		2	2	6		4	
3 R	5	5		2						2		1	
G	28	23	5			10	2	3	1	8	1	2	1
4 R	3	3		1		1						1	
G	20	19	1			5		8	1	3		3	
5 R	8	6	2			2			2	3		1	
G	22	16	6	1		5	1		3	5	1	5	1
6 R	9	9		2				1		4		1	
G	23	21	2	1		5		3	2	5		5	
7 R	4	4		2		1		1					
G	23	15	8			5	2	1	6	5		4	
8 R	6	6								1		5	
G	33	29	4			8		3	4	11		7	
9 R	5	5		1						2		2	
G	22	20	2			4		2	2	6		8	
10 R	7	5	2	2		1				1		1	2
G	26	22	4			9		6	4	3	1	3	
11 R	8	7	1	1		2		1	1	2		1	
G	22	17	5			6		2	4	5	1	3	1
12 R	9	8	1	2		1				1		4	1
G	30	25	5			8		1	5	6		10	
13 R	4	4				1		1		1		1	
G	29	26	3			10		3	3	8		5	
14 R	3	3		1						1		1	
G	26	24	2			6		4	1	5	1	9	
15 R	5	5		3		1						1	
G	26	23	3	2		8		6	2	3		4	1
16 R	10	10		2		1		1		4		2	
G	24	21	3	1		7		1	3	7		5	

Table A-1: Sample Data Sheet (continued)

Cycl	Total	Total		L-T Lane		Lane 3		Lane 2		Lane 1		R-T Lane	
		C	T	C	T	C	T	C	T	C	T	C	T
17 R	5	5		1		1				3			
G	19	11	8			3		1	7	4		3	1
18 R	4	4								1		3	
G	31	27	4			11		3	4	5		8	
19 R	8	8						1		2		5	
G	15	11	4						4	7		4	
20 R	16	11	5			1			5	7		3	
G	11	8	3	2		4		2	3				
21 R	6	5	1	2		3			1				
G	19	15	4			5	4	3		6			1
22 R	10	8	2	1		2			2	3		2	
G	16	13	3	1		2		2	3	3		5	
23 R	2	2		1		1							
G	29	24	5			9		5	5	4		6	
24 R	13	13		1		2		2		3		5	
G	21	15	6				5		2	5	3	7	1
25 R	5	4	1	1	1	1						2	
G	26	20	6			7		3	5	5	1	5	
26 R	9	9		2		1				3		3	
G	18	14	4			6	1		3	3		5	
27 R	5	5		1		1				3			
G	29	24	5			5	1	4	3	9	1	6	
28 R	7	7		1		1		1		3		1	
G	16	16				4		3		6		3	
29 R	4	4						1		1		2	
G	22	16	6			7		2	4	5	2	2	
30 R	10	9	1	2	1			1		4		2	
G	20	12	8	0	0	8	1	1	4	2	2	1	1

C = Car; T = Truck; R = Red interval; G = Green interval; L-T = Left-turn;

R-T = Right-Turn

Table A-2: Sample Data Sheet

Cycle-by-cycle traffic counts; Huron Church -Tecumseh Intersection (northbound)

Date: Friday, June 6, 2009 Time: Time: 3:30 pm. - 4-30 pm.

Cycle	Total	Total		L-T Lane		Lane 3		Lane 2		Lane 1		R-T Lane	
		C	T	C	T	C	T	C	T	C	T	C	T
1 R	7			1				2		2		2	
G	21	17	4	1		6		3	4	4		3	
2 R	6	6		2		2				1		1	
G	23	19	4	1		6		2	4	6		5	
3 R	5	5		2						2		1	
G	18	15	3	1		3		1	3	4		6	
4 R	7	7				1		2		3		1	
G	21	19	2			6		5	2	4		4	
5 R	6	5	1			1			1	3		1	
G	22	19	5			6		4	5	2		7	
6 R	7	2	5						5	1		1	
G	23	14	4			6		1	3	4	1	3	
7 R	5	5		2		2						1	
G	23	17	3	1		7		3	2	4		2	1
8 R	9	9				2		2		2		3	
G	33	17	4			7	1	3	3	3		4	
9 R	8	6	2	1		1			2	3		1	
G	27	21	6			7			6	8		6	
10 R	9	9				1		1		3		4	
G	24	21	3			9		3	3	4		5	
11 R	9	8	1			1		2		1	1	4	
G	22	20	2			10		2	2	5		3	
12 R	10	10		2		2				3		3	
G	30	28	24	4		6	1	6	3	4		7	
13 R	8	6	2	1		1			2	1		3	
G	21	16	5			7	1	4	3	2		3	1
14 R	15	9	6			3			4	2	1	4	1
G	19	17	2			7		2	2	3		5	
15 R	5	4	1	1		1			1	1		1	
G	24	21	3			8		3	3	5		5	
16 R	7	7		1		3		2				1	
G	20	18	2	1		6		1	1	2	1	8	
17 R	3	3				1		1				1	
G	19	10	10			3		1		4		2	
18 R	15	14	1	1		6				2	1	5	
G	22	19	3			9		2	3	7		1	

Table A-2: Sample Data Sheet (Continued)

Cycle	Total	Total		L-T Lane		Lane 3		Lane 2		Lane 1		R-T Lane	
		C	T	C	T	C	T	C	T	C	T	C	T
19 R	8	8				1		1		4		2	
G	17	11	6			4		2	5	3		2	1
20 R	7	7		1		1		1		3		1	
G	25	21	4			8		4	4	4		5	
21 R	9	9		1				2		2		4	
G	21	19	2			6		5	2	4		4	
22 R	15	15		3		2		2		3		5	
G	21	17	4			6		4	3	2	1	5	
23 R	3	3		1								2	
G	21	17	4			5		5	4	3		4	
24 R	11	11		3				2		2		4	
G	24	22	2			6		6	1	6	1	4	
25 R	6	6		1				1		2		2	
G	15	15				6		3		3		3	
26 R	8	8				3		2		2		1	
G	32	29	3			12		8	1	3	1	6	1
27 R	10	10		3		2		2		1		2	
G	25	20	5			9		1	4	8		2	1
28 R	3	3				1				2			
G	22	17	5			7			4	5		5	1
29 R	4	3				1		1		1			
G	25	25		2		9		6		5		3	
30 R	6	5	1	1	1					1	1	2	
G	31	26	5			9		7	5	5		4	1

C = Car; T = Truck; R = Red interval; G = Green interval; L-T = Left-turn;

R-T = Right-Turn

APPENDIX B: Effective green and effective red calculation

A. Effective green and effective red calculation for northbound Huron Church-Tecumseh intersection based on Mannering et al.(2009)

Car length = 4.55 m (average); Truck length = 22 m (average)

Width of Tecumseh Road (cross road) at the intersection = 18.29 m

Cross road width + car width = 18.29 + 4.55 = 22.84 m

Cross road width + truck width = 18.29 + 22 = 40.29 m

North bound speed [50 - 70 km / hour] = [13.88 - 19.44 m / sec] =16.66m / sec (average)

Minimum All Red interval = AR= (Cross road width + car width) / speed

= 22.84 m / 16.66 m / sec = 1.37 sec = 1.5 sec

Or, Maximum All Red interval AR = (Cross road width + truck width) / speed

= 40.29 m / 16.66 m / sec = 2.42 sec = 2.5 sec

Displayed Red = Red end – start end = 71sec - 0 sec = 71 sec

Displayed Green = G = Green end – Red end = 113 sec - 71 sec = 42 sec

Yellow=Y= Start at 114 sec - end at 117 sec = 4 sec

AR= All Red interval = 2.5 sec (estimated) = start at 118 sec - end at 120 sec

Start-up lost time = TsL = Reaction-Perception time (1 sec) and acceleration time (1 sec) = 2 sec

Clearance lost time, TcL = Last second of yellow interval (1 sec) + entire All-Red interval (3 sec) = 1 sec + 2.5 sec = 3.5sec

Effective green, $g = G+Y+AR-(TsL + TcL) = (42+4+2.5) - (2+3.5) = 48.5$ sec $- 5.5$ sec $= 43$ sec

Effective Red =120 sec - 43 sec = 77 seconds

They considers the typical value of the start-up lost time as around 2 seconds and the entire all-red interval including the last second of the yellow interval as the clearance lost time (Mannering et al., 2009).

B. Effective green and effective red calculation for northbound Huron Church at Tecumseh intersection based on CCG method

According to the Canadian Capacity Guide (CCG) effective green time is 1 sec longer than the displayed green interval for signalized intersections (Canadian Capacity Guide for Signalized Intersections, 2008). We can follow the CCG method for calculating effective green time since it should better reflect the Canadian traffic conditions. Thus if the green time for northbound through traffic of Huron Church Road is 42 seconds, the effective green time will be 43 seconds and the effective red time for this approach will be 77 seconds.

APPENDIX C: Estimation of values of shockwave parameters

Assuming the average speed (u) of vehicles is 40 km / hour (38 ft /sec) in saturation flow state (before break point C) and 60 km (53 ft /sec) in arrival state (after break point C) at upstream of a signalized intersection. The minimum safe time headways (h_{MIN}) between the vehicles can be computed by using the following Pipes' theory (May, 1990):

$$h_{MIN} = 1.36 + L_n / u$$

where, L_n = length of vehicle and u = speed of vehicle

Case 1: Car 100% + truck 0%

Assuming car length = 20 ft.

Headway time in saturation flow state, h = 1.36 + 20 / 38 = 1.88 sec

Flow in saturation state, q_m = 3600 /1.88 = 1914 vehicles / hour

Density in saturation state, k_m = flow / speed = [1914 v / hr] / [40 km / hr] = 48 vehicle / km

Assuming that the headway time in arrival state is longer than that of saturation state (For example, 1.25 times longer).

Therefore, headway time in arrival state is 1.88 x 1.25 = 2.35 sec.

Flow in arrival state, q_a = 3600 / 2.35 = 1532 vehicles / hour

Density in arrival state, k_a = flow / speed = [1532 v / hr] / [60 km / hr] = 26 vehicle / km

Assume effective length = length of vehicle + gap between the two consecutive vehicles in queue

= 20 ft +10 ft = 30 ft = 9 m

Jam density, kj = 1000 m / 9 m = 111 vehicles / km

Queuing shockwave, $v_1 = (0 - q_a) / (k_j - k_a)$ = (0 -1532) / (111 - 26) = - 1532 / 85

= - 18.02 km/ hour = - 5.0 m / sec

Discharge shockwave, $v_2 = (q_m - 0) / k_m - k_j) = (1914 - 0) / (48-111) = 1914 / - 63$

= - 30.38 km/hour = - 8.4 m /sec

Departure shockwave, $v_3 = (q_m - q_a) / (k_m - k_a) = (1914 - 1532) / (48 - 26) = 382 / 22$

= + 17.37 km/hour v_3= + 4.8 m/sec

Case 2: Car 50% and truck 50%

Assuming car length = 20 ft

Assuming truck length = 72ft.

Headway time in saturation flow state, h = 50% (1.36 + 20 / 38) + 50% (1.36 + 72 / 38)

= 50% x 1.88 + 50% x 3.25 = 2.565 sec

Headway time in saturation flow state, h = = 2.565 sec

Flow in saturation state, q_m = 3600 / 2.565 = 1404 vehicles / hour

Density in saturation state, k_m = flow / speed = [1404 v / hr] / [40 km / hr] = 35 vehicle / km

Assuming that the headway time in arrival state is longer than that of saturation state (For example, 1.25 times longer).

Therefore, headway time in arrival state is 2.565 x 1.25 = 3.2 sec.

Flow in arrival state, q_a = 3600 / 3.2 = 1125 vehicles / hour

Density in arrival state, k_a = flow / speed = [1125 v / hr] / [60 km / hr] = 18.75 vehicle / km

Assume effective length = length of vehicle + gap between the two consecutive vehicles in queue

= 50% x 30 ft +50% x 90 ft = 60 ft = 18.29 m

Jam density, k_j = 1000 m /18.29 m = 56 vehicles / km

Queuing shockwave, $v_1 = (0 - q_a) / (k_j - k_a) = (0 -1125) / (56 - 18.75) = - 1125 / 37.25$

= - 30.20 km / hour = - 8.38 m / sec

Discharge shockwave, $v_2 = (q_m - 0) / k_m - k_j) = (1404 - 0) / (35 - 56) = 1404 / - 21$

$= - 66.85$ km / hour $= - 18.57$ m /sec

Departure shockwave, $v_3 = (q_m - q_a) / (k_m - k_a) = (1404 - 1125) / (35 - 18.75) = 279 / 16.25$

$= + 17.1$ km / hour $= + 4.76$ m/sec

Case 3: Car 0% and truck 100%

Assuming truck length = 72ft.

Headway time in saturation flow state, h = 1.36 + 72 / 38 = 3.25 sec

Flow in saturation state, q_m = 3600 /3.25 = 1107 vehicles / hour

Density in saturation state, k_m = flow / speed = [1107 v / hr] / [40 km / hr] = 28 vehicle / km

Assuming that the headway time in arrival state is longer than that of saturation state (For example, 1.25 times longer).

Therefore, headway time in arrival state is 3.25 x 1.25 = 4.06 sec.

Flow in arrival state, q_a = 3600 / 4.06 = 887 vehicles / hour

Density in arrival state, k_a = flow / speed = [887 v / hr] / [60 km / hr] = 15 vehicle / km

Assume effective length = length of vehicle + gap between the two consecutive vehicles in queue

= 72 ft +18 ft = 90 ft = 27 m

Jam density, k_j = 1000 m / 27 m = 37 vehicles / km

Queuing shockwave, $v_1 = (0 - q_a) / (k_j - k_a) = (0 - 887) / (37 - 15) = - 887 / 22$

$= - 40.31$ km/ hour $= - 11.2$ m / sec

Discharge shockwave, $v_2 = (q_m - 0) / (k_m - k_j) = (1107 - 0) / (28-37) = 1107 / - 9$

$= - 123$ km/hour $= - 34.2$ m /sec

Departure shockwave, $v_3 = (q_m - q_a) / (k_m - k_a) = (1107 - 887) / (28 - 15) = 220 / 13$

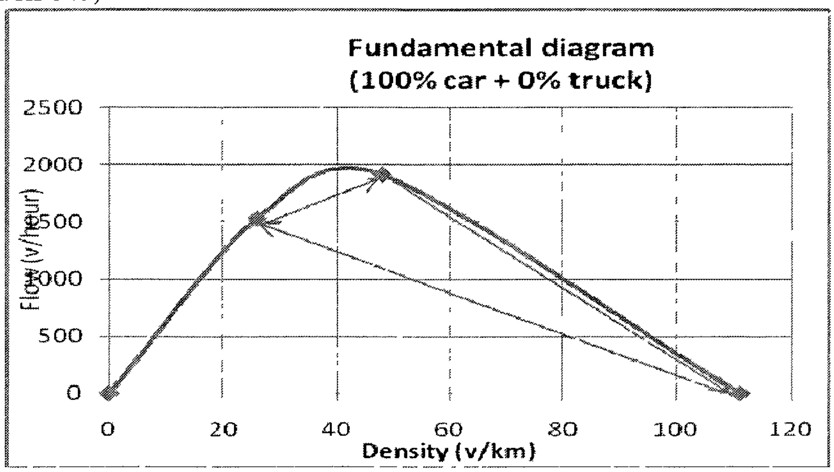
$= + 16.92$ km/hour $v_3 = + 4.70$ m/sec

APPENDIX D: Fundamental diagrams

FIGURE D-1: Fundamental Diagram

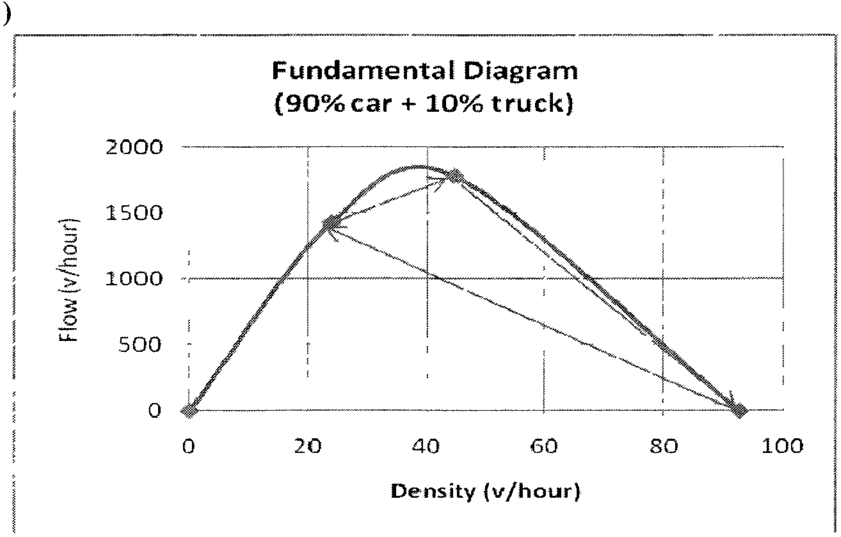
Case 1 **(Car 100%+truck 0%)**

Density	Flow
0	0
26	1532
48	1914
111	0



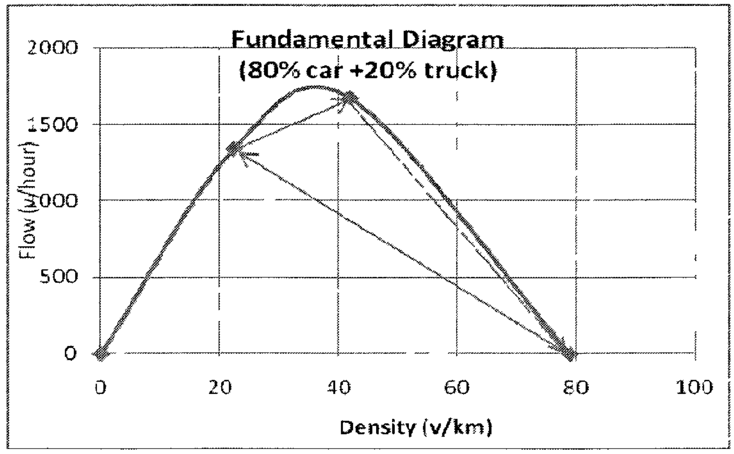
Case 2 **(Car 90%+truck 10%)**

Density	Flow
0	0
23.8	1428
44.6	1784
92.6	0



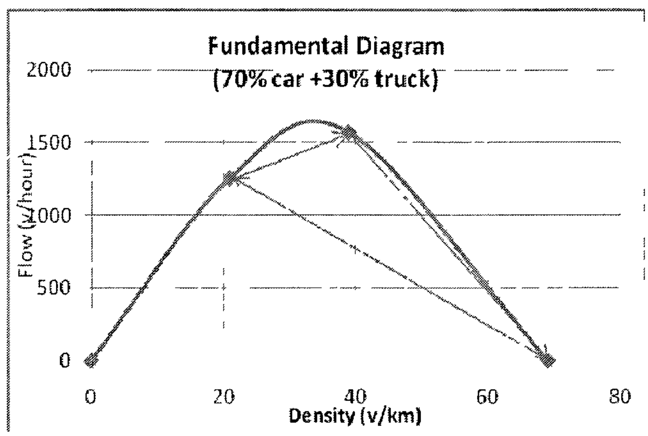
Case 3 (Car 80%+truck 20%)

Density	Flow
0	0
22.33	1340
41.85	1674
79	0



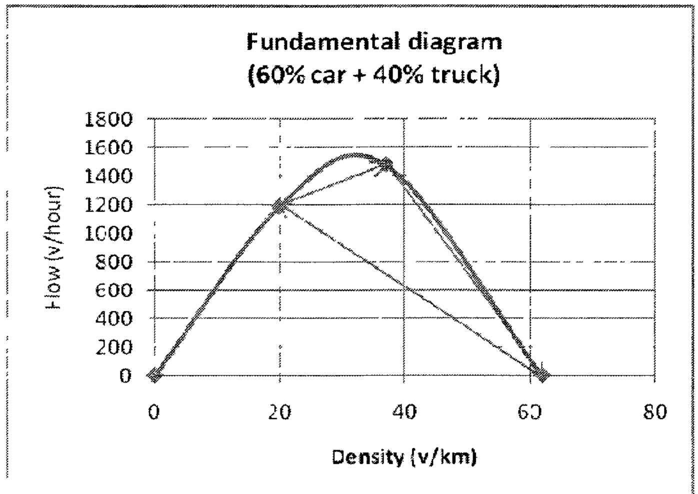
Case 4 (Car 70%+truck 30%)

Density	Flow
0	0
21	1258
39	1572
69	0



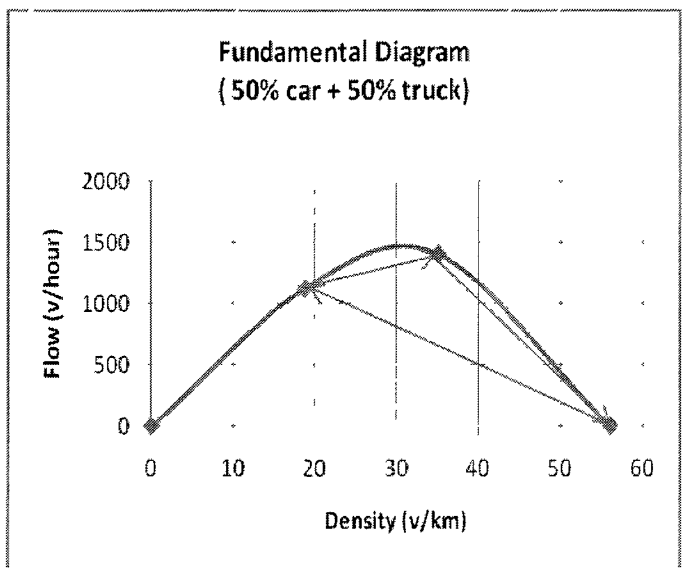
Case 5 (Car 60%+truck 40%)

Density	Flow
0	0
19.8	1186
37	1482
62	0



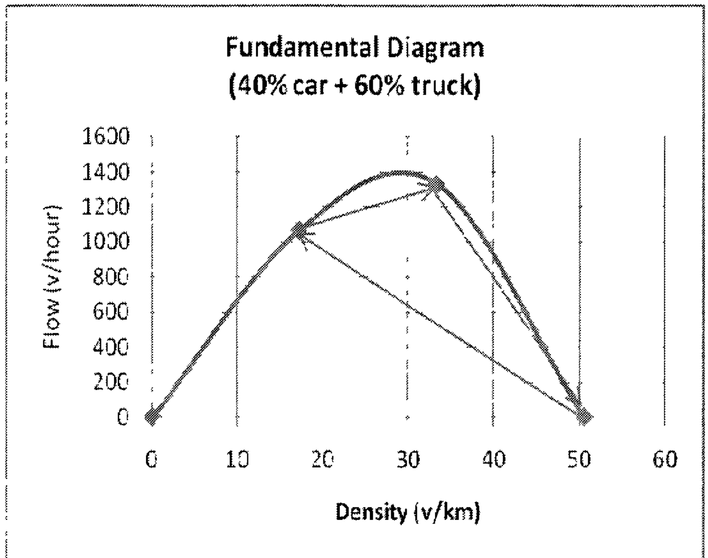
Case 6 (Car 50%+truck 50%)

Density	Flow
0	0
18.75	1125
35	1404
56	0



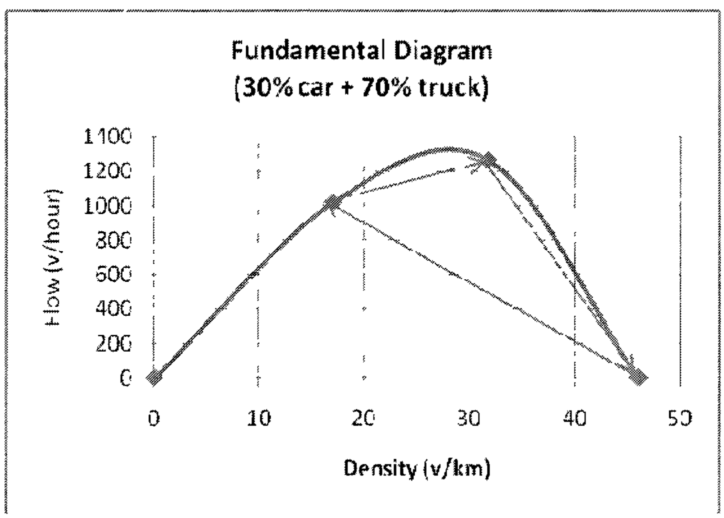
Case 7 **(Car 40%+ truck 60%)**

Density	Flow
0	0
17.17	1066
33.3	1332
50.5	0

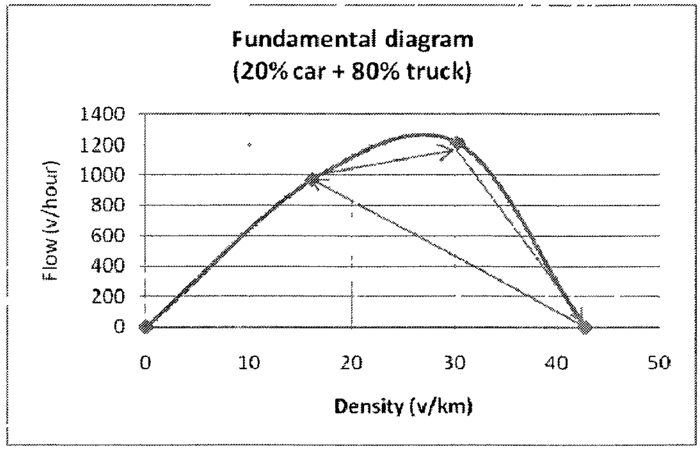


Case 8 **(Car 30%+truck 70%)**

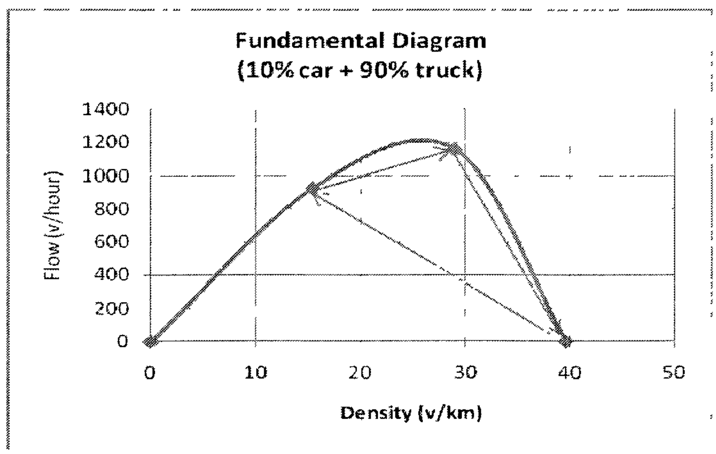
Density	Flow
0	0
16.94	1017
31.7	1268
46	0



Case 9	(Car 20%+truck 80%)
Density	Flow
0	0
16.13	968
30.25	1210
42.73	0



Case 10	(Car 10%+truck 90%)
Density	Flow
0	0
15.42	925
28.9	1156
39.68	0



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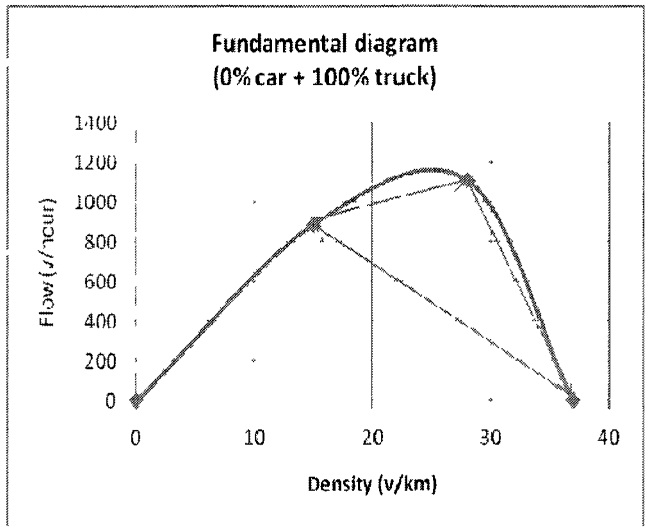
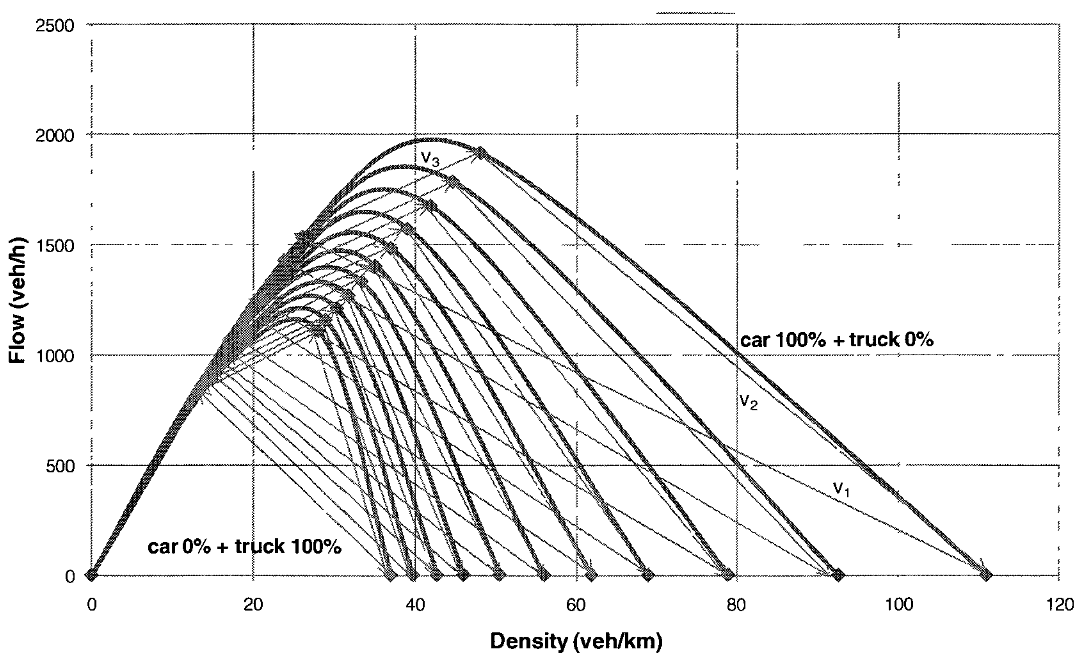
Case 11 **(Car 0%+truck 100%)**

Density	Flow
0	0
15	887
28	1107
37	0





APPENDIX E: Second -by-second VISSIM Simulated data

TABLE E-1: Second -by-second VISSIM Simulated Data (Lane 1 at 60 m Upstream of Stop Line)

Time (sec)	Car	Truck	Occupancy (%)	Break Points
1	0	0	0	No queue
2	0	0	0	
3	0	0	0	
4	0	0	0	
5	0	0	0	
6	0	0	0	
7	0	0	0	
8	0	0	0	
9	0	0	0	
10	0	0	0	
11	0	0	0	
12	0	0	0	
13	0	0	0	
14	0	0	0	
15	1	0	26.8	
16	0	0	0	
17	1	0	31.8	
18	0	0	0	
19	0	0	0	
20	0	0	0	
21	0	0	0	
22	0	0	0	
23	0	0	0	
24	1	0	36.1	
25	0	0	0	
26	1	0	44.6	
27	0	0	18.9	
28	0	0	0	
29	0	0	0	
30	0	0	0	
31	0	0	0	
32	0	0	0	
33	0	0	0	
34	0	0	0	
35	0	0	0	
36	1	0	42.7	
37	0	0	0	

TABLE E-1: Second -by-second VISSIM Simulated Data (Lane 1 at 60 m Upstream of Stop Line) (Continued)

Time (sec)	Car	Truck	Occupancy (%)	Break Points
38	0	0	0	
39	0	0	0	
40	0	0	0	
41	0	0	0	
42	1	0	1.1	
43	0	0	0	
44	0	0	0	
45	0	0	0	
46	0	0	0	
47	0	0	0	
48	0	0	0	
49	1	0	18.7	
50	0	0	48.1	
51	0	0	0	
52	0	0	0	
53	0	0	0	
54	0	0	0	
55	0	0	0	
56	0	0	0	
57	0	0	0	
58	0	0	0	
59	1	0	5.5	
60	0	0	0	
61	0	0	0	
62	0	0	0	
63	0	0	0	
64	0	0	0	
65	0	0	0	
66	0	0	0	
67	0	0	0	
68	0	0	0	
69	0	0	0	
70	0	0	0	
71	0	0	0	
72	0	0	0	
73	0	0	0	
74	0	0	0	
75	0	0	0	
76	0	0	0	
77	0	0	0	

TABLE E-1: Second -by-second VISSIM Simulated Data (Lane 1 at 60 m Upstream of Stop Line) (Continued)

Time (sec)	Car	Truck	Occupancy (%)	Break Points
78	**0**	**0**	**0**	
79	0	0	0	
80	0	0	0	
81	0	0	0	
82	0	0	0	
83	0	0	0	
84	1	0	88.3	
85	0	0	15.4	
86	1	0	60.5	
77	0	0	0	
88	1	0	35.5	
89	0	0	19.5	
90	1	0	50.7	
91	0	0	0	
92	1	0	48.1	
93	0	0	0	
94	0	0	0	
95	0	0	0	
96	0	0	0	
97	0	0	0	
98	0	0	0	
99	0	0	0	
100	0	0	0	
101	0	0	0	
102	0	0	0	
103	0	0	0	
104	0	0	0	
105	0	0	0	
106	1	0	20.5	
107	0	0	6.6	
108	0	0	0	
109	0	0	0	
110	0	0	0	
111	0	0	0	
112	1	0	26.1	
113	0	0	0	
114	0	0	0	
115	0	0	0	
116	0	0	0	
117	0	0	0	
118	0	0	0	
119	0	0	0	
120	0	0	0	

Note: No queue formed in L1.

TABLE E-2: Second -by-second VISSIM Simulated Data (Lane 2 at 60 m Upstream of Stop Line)

Time (sec)	Car	Truck	Speed	Occupancy (%)	Break Point
1	0	0	0	0	
2	0	1	42.4	50.4	
3	0	0	0	100	
4	0	0	0	58	
5	0	0	0	0	
6	0	0	0	0	
7	0	0	0	0	
8	0	0	0	0	
9	0	0	0	0	
10	0	0	0	0	
11	0	0	0	0	
12	0	0	0	0	
13	0	0	0	0	
14	0	0	0	0	
15	0	0	0	0	
16	0	0	0	0	
17	0	0	0	0	
18	0	0	0	0	
19	0	0	0	0	
20	0	0	0	0	
21	0	0	0	0	
22	1	0	52.8	30.8	
23	0	0	0	0	
24	0	0	0	0	
25	1	0	49	16.2	
26	0	0	0	18	
27	0	0	0	0	
28	0	0	0	0	
29	0	0	0	0	
30	0	0	0	0	
31	**0**	**1**	**27**	**66.3**	**A**
32	0	0	0	100	
33	0	0	0	100	
34	0	0	0	90.8	
35	0	0	0	0	
36	0	0	0	0	
37	0	0	0	0	
38	0	0	0	0	
39	0	0	0	0	

TABLE E-2: Second -by-second VISSIM Simulated Data (Lane 2 at 60 m Upstream of Stop Line) (Continued)

Time (sec)	Car	Truck	Speed	Occupancy (%)	Break Point
40	0	1	12	2.2	
41	0	0	0	100	
42	0	0	0	100	
43	0	0	0	100	
44	0	0	0	100	
45	0	0	0	100	
46	0	0	0	100	
47	0	0	0	100	
48	0	0	0	100	
49	0	0	0	100	
50	0	0	0	100	
51	0	0	0	100	
52	0	0	0	100	
53	0	0	0	100	
54	0	0	0	100	
55	0	0	0	100	
56	0	0	0	100	
57	0	0	0	100	
58	0	0	0	100	
59	0	0	0	100	
60	0	0	0	100	
61	0	0	0	100	
62	0	0	0	100	
63	0	0	0	100	
64	0	0	0	100	
65	0	0	0	100	
66	0	0	0	100	
67	0	0	0	100	
68	0	0	0	100	
69	0	0	0	100	
70	0	0	0	100	
71	0	0	0	100	
72	0	0	0	100	
73	0	0	0	100	
74	0	0	0	100	
75	0	0	0	100	
76	0	0	0	100	
77	0	0	0	100	
78	0	0	0	100	
79	0	0	0	100	
80	0	0	0	63.7	
81	0	0	0	0	

TABLE E-2: Second -by-second VISSIM Simulated Data (Lane 2 at 60 m Upstream of Stop Line) (Continued)

Time (sec)	Car	Truck	Speed	Occupancy (%)	Break Point
82	**0**	**1**	**27.1**	**34**	**B**
83	0	0	0	100	
84	0	0	0	100	
85	0	0	0	20.8	
86	1	0	39.4	19.1	
77	0	0	0	22.2	
88	0	0	0	0	
89	1	0	42.1	29.9	
90	0	0	0	7.8	
91	**1**	**0**	**44**	**15.2**	**C**
92	0	0	0	21	
93	0	0	0	**0**	
94	0	0	0	**0**	
95	0	0	0	**0**	
96	1	0	52.5	31.4	
97	0	0	0	0	
98	0	0	0	0	
99	0	0	0	0	
100	0	0	0	0	
101	0	0	0	0	
102	1	0	50.7	31.7	
103	0	0	0	0	
104	0	0	0	0	
105	0	0	0	0	
106	0	0	0	0	
107	1	0	61.9	26.4	
108	0	0	0	0	
109	0	0	0	0	
110	0	0	0	0	
111	0	0	0	0	
112	0	0	0	0	
113	0	0	0	0	
114	0	0	0	0	
115	1	0	59.5	18.7	
116	0	0	0	8.9	
117	0	0	0	0	
118	0	0	0	0	
119	0	0	0	0	
120	0	0	0	0	

Since there are 5 cars and 4 trucks from the beginning of red interval to Break Point C (0-91 seconds) queue length in L2 = 1.2 + 5 (4.55) + 4 (22) + (9-1) (2) = 127.95 m.

TABLE E-3: Second -by-second VISSIM Simulated Data (Lane 3 at 60 m Upstream of Stop Line)

Time (sec)	Car	Truck	Occupancy (%)	Break Point
1	0	0	0	
2	0	0	0	
3	0	0	0	
4	0	0	0	
5	0	0	0	
6	0	0	0	
7	0	0	0	
8	0	0	0	
9	0	0	0	
10	0	0	0	
11	0	0	0	
12	0	0	0	
13	0	0	0	
14	1	0	26.9	
15	0	0	1.1	
16	1	0	28.9	
17	0	0	0	
18	0	0	0	
19	1	0	0.3	
20	0	0	45.1	
21	0	0	0	
22	0	0	0	
23	0	0	0	
24	1	0	55.8	
25	0	0	0	
26	0	0	0	
27	1	0	67.7	
28	0	0	0	
29	0	0	0	
30	0	0	0	
31	0	0	0	
32	0	0	0	
33	0	0	0	
34	0	0	0	
35	1	0	7.2	
36	0	0	42.3	
37	0	0	0	
38	0	0	0	
39	0	0	0	
40	**1**	**0**	**3**	**A**
41	0	0	57.3	

TABLE E-3: Second -by-second VISSIM Simulated Data (Lane 3 at 60 m Upstream of Stop Line) (Continued)

Time (sec)	Car	Truck	Occupancy (%)	Break Point
42	0	0	0	
43	0	0	0	
44	0	0	0	
45	0	0	0	
46	0	0	0	
47	0	0	0	
48	0	1	68.6	
49	0	0	100	
50	0	0	100	
51	0	0	100	
52	0	0	100	
53	0	0	100	
54	0	0	100	
55	0	0	100	
56	0	0	100	
57	0	0	100	
58	0	0	100	
59	0	0	100	
60	0	0	100	
61	0	0	100	
62	0	0	100	
63	0	0	100	
64	0	0	100	
65	0	0	100	
66	0	0	100	
67	0	0	100	
68	0	0	100	
69	0	0	100	
70	0	0	100	
71	0	0	100	
72	0	0	100	
73	0	0	100	
74	0	0	100	
75	0	0	100	
76	0	0	100	
77	0	0	100	
78	0	0	100	
79	0	0	100	
80	0	0	100	
81	0	0	100	
82	0	0	56.9	
83	0	0	0	
84	**1**	**0**	**53.2**	**B**

TABLE E-3: Second -by-second VISSIM Simulated Data (Lane 3 at 60 m Upstream of Stop Line) (Continued)

Time (sec)	Car	Truck	Occupancy (%)	Break Point
85	0	0	0	
86	1	0	49.8	
77	1	0	9.9	
88	0	0	33.9	
89	**1**	**0**	**20.9**	**C**
90	0	0	23.2	
91	0	0	**0**	
92	0	0	**0**	
93	1	0	14	
94	0	0	23.3	
95	0	0	0	
96	0	0	0	
97	0	0	0	
98	0	0	0	
99	1	0	31.5	
100	0	0	0	
101	0	0	0	
102	0	0	0	
103	0	0	0	
104	0	0	0	
105	0	0	0	
106	0	0	0	
107	1	0	27.7	
108	0	0	0	
109	0	0	0	
110	0	0	0	
111	0	0	0	
112	0	0	0	
113	0	0	0	
114	0	0	0	
115	0	0	0	
116	0	0	0	
117	0	0	0	
118	0	0	0	
119	0	0	0	
120	0	0	0	

Since there are 11 cars and 1 trucks from the beginning of red interval to Break Point C (0-89 seconds) queue length in L3 = 1.2 + 11 (4.55) + 1 (22) + (12-1) (2) = 95.25 m.

Comparing the queue formed in the above three lanes we can conclude that maximum queue length occurred in L2.

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