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Real-Time Identification and Tracking of Traffic Queues Based on Average Link Speed

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ABSTRACT

Within the scope of Intelligent Transportation Systems (ITS), automatic identification and tracking of queues on a freeway corridor are important in monitoring traffic condition and determining the impact of incidents on traffic flow as these activities can improve motorist safety and traffic flow efficiency.

Existing models for identifying and tracking queue generally suffer from the following limitations. First, they typically rely on the spot speeds obtained from loop detectors. Second, they subjectively select threshold speed values to distinguish queued condition from non-queued condition. Third, the use of a single speed threshold value to distinguish between congested and uncongested traffic conditions ignores the uncertainty that often exists in distinguishing congested from uncongested traffic. Lastly, some existing queue tracking models are not capable of detecting and tracking a queue on a real-time basis.

This study proposes a method for the real-time identification and tracking of freeway traffic queues. The method consists of the three algorithms: (1) Average Link Speed Algorithm; (2) Traffic Zone Identification Algorithm; and (3) Queue Type Identification Algorithm.

The proposed method overcomes some of the limitation of existing queue tracking method by using average link speeds rather than spot speeds, and by using "objectively" calibrated threshold speeds as a means of identifying queued condition based on field data. The proposed method is also suitable for real-time implementation. The method is evaluated using loop detector data from the Gardiner Expressway in Toronto, Canada. The accuracy of the method was generally high in spite of a relatively simple model structure.

KEYWORDS: Traffic, Queue, Real-time, Freeway, Link speed

INTRODUCTION

Traffic queues are formed as a result of either recurring congestion or incidents. The accurate and timely detection and tracking of queues are important to maximize motorist safety and traffic flow efficiency within the scope of Intelligent Transportation System (ITS). Safety is a significant concern since at the tail of the queue vehicles must decelerate quickly. Accident risk is increased in locations where sight lines restrict drivers' ability to see the end of the queue, reducing the time available for deceleration. Predicting when the end of the queue will reach or pass the selected on-ramps/off-ramps can help prevent the growth of ramp blockage and minimize the delay caused by queue spill back.

For these reasons, queue identification and tracking have drawn attention from researchers and practitioners over the past few years. A number of methods have been proposed, but as described in the next section, they suffer from a number of limitations. To avoid these limitations, this study develops a method that estimates the location of a queue in real-time based on the average speed on the road section(s). In the proposed method, the threshold speeds are determined from the calibration of the model using real traffic data rather than analysts' arbitrary assumption. The study also evaluates the accuracy of the method by comparing the method estimates with the observed queue transition using the re-scaled cumulative arrival curves.

This paper consists of six sections. The second section describes existing queue-tracking methods and identifies their limitations. The third section describes the proposed method. The fourth section illustrates the calibration of threshold speeds in the method using loop detector data and re-scaled cumulative arrival curves. The fifth section discusses the calibration results and the estimation of errors. The sixth section discusses the findings, and makes conclusions and recommendations for future studies.

LITERATURE REVIEW

Some work has been done in the area of queue detection and queue tracking on freeways. Queue detection consists of determining whether or not traffic is congested at the point along the roadway whereas queue tracking attempts to estimate the location of the extent of the queue at any point in time.

Many studies have suggested the methods of queue detection based on real-time traffic flow data. For example, Lin and Daganzo (1997) used cumulative differences of occupancy over time to detect a traffic queue between two neighboring loop detectors. They suggested that significant disruption of cumulative difference of occupancy indicate the occurrence of incidents. Coifman (2000) developed the method of detecting the onset of congestion based on individual vehicles' travel time. His model detects the onset of congestion whenever link travel time dramatically increases.

Some studies developed queue-tracking methodologies that provide a more refined estimation of queue location using real-time traffic flow data. Cassidy and Windover (1995) have attempted to track a queue evolution using a "re-scaled cumulative arrival curve" (*N*-curve). The *N*-curve is used as a tool to identify the transitions between free flow traffic and queued conditions, and to identify some important traffic features on freeways. The transition of traffic queue was manually identified using this methodology in their few other studies (Cassidy and

Bertini, 1996; Cassidy and Mauch, 2001). However, to minimize human intervention, queue identification and tracking need to be automated.

In this regard, the City of Toronto (1992) developed a queue-tracking algorithm for the Gardiner Expressway to determine the location of a queue between a pair of loop detector stations in their corridor traffic management system (CTMS). The required inputs of this algorithm are volume at upstream and downstream detectors, five-minute average vehicle lengths, and the separation distance between vehicles in a queue. Jasperse and Van Toorenburg (1999) developed a queue-tracking model to estimate the queue length on a long freeway segment in the Netherlands. The model calculates the queue location from the difference in densities between the congested and the uncongested regimes. However, these models were not validated extensively by comparing with true queue position.

Payne and Thompson (1998) attempted to identify traffic conditions between loop detector stations on the basis of the boundary speeds between queued and non-queued condition using only spot speeds at loop detector stations. However, spots speeds only reflect traffic conditions at particular fixed locations but cannot account for the variation of traffic condition within the road section. Thus, the average link speed needs to be estimated using spot speeds. They applied the model to the Twin Cities freeway system for two weeks and tested the performance. Despite the application of the model to real traffic conditions, the authors did not clearly provide guidance as to the basis of the selection of threshold speeds.

THE PROPOSED METHOD

To overcome the limitations from the past studies, this study proposes a method for identifying and tracking freeway queues in real-time based on average link speed. Link speed is estimated using speeds and volumes at fixed locations along the roadway. These speed and volume data are usually obtained from road sensors such as loop detectors installed on freeways. The proposed method is composed of three main parts as shown in Figure 1 and they are explained in detail in the following subsections.

Algorithm 1—Average Link Speed Algorithm

This algorithm calculates the average space mean link speed using the speed and volume collected by road sensors such as loop detector. This study assumes that the 20-second average loop detector data are used to calculate average link speed. However, this algorithm can also be used with the data collected by other types of road sensors. Due to the large variation associated with the 20-second detector data, the one-minute moving average speed for each individual lane is used. The "station speed" (S_s), calculated as the volume-weighted average of the individual lane one-minute average speed, is determined for both upstream and downstream loop detector stations. For each road section, the upstream and downstream station speeds are denoted as S_{s1} and S_{s2} respectively, as shown in Figure 2.

The space mean speed associated with travel over the entire road segment is computed using Equation 1. However, travel time (t_i) over the road segment length cannot be measured using loop detectors.

$$S_{sms} = \frac{N}{\sum_{i=1}^{N} t_i} \tag{1}$$

where:

S_{sms}	=	space mean speed (km/h);
t_i	=	travel time of vehicle *i* over unit road segment (h/km);
N	=	total number of vehicles.

The average volume-weighted space mean link speed can be approximated using the one-minute volume-weighted average speeds at both upstream and downstream detectors. The two station speeds are converted to travel time using Equation 2a and the average volume-weighted link travel time (t_L) between the two stations is calculated using Equation 2b:

$$t_1 = \frac{d}{S_{S1}},\ t_2 = \frac{d}{S_{S2}} \tag{2a}$$

where:

t_1, t_2	=	estimated travel time at upstream and downstream loop detector station respectively;
d	=	distance between the two successive loop detector stations;
S_{S1}, S_{S2}	=	one-minute volume weighted average speeds (station speed) at upstream and downstream loop detector station respectively.

$$t_L = \frac{V_1 * t_1 + V_2 * t_2}{V_1 + V_2} \tag{2b}$$

where:

t_L	=	average volume-weighted link travel time for the road section between the two successive loop detector stations;
V_1	=	volumes at upstream loop detector station;
V_2	=	volumes at downstream loop detector station.

Finally, average volume-weighted link speed (S_L) for road segment between the two loop detector stations is calculated as shown in Equation 2c.

$$S_L = \frac{d}{t_L} \tag{2c}$$

The average link speed can be calculated from the fundamental relationship between speed, distance, and time by substituting (2b) into (2c):

$$S_L = \frac{d}{\left[\dfrac{V_1 * t_1 + V_2 * t_2}{V_1 + V_2}\right]}$$

$$S_L = \frac{V_1 + V_2}{\left[\dfrac{V_1}{S_{s1}} + \dfrac{V_2}{S_{s2}}\right]} \tag{3}$$

Algorithm 2—Traffic Zone Identification Algorithm

The proposed method modifies the methodology of identifying traffic queues suggested by Payne and Thompson (1998). Unlike their original study, this study uses the above-mentioned average link speed and compares it with two threshold speeds to identify the existence of queues.

Most existing queue tracking methods state that traffic conditions are divided into two categories: congested (traffic jam) and uncongested (free flow) traffic condition; and they are divided with only one threshold speed. However, this classification is over-simplified as the boundary between congested and uncongested traffic often cannot be defined clearly. Kerner and Rehborn (1996) found that there exists a "gray area" between the two traffic conditions define this as "synchronized traffic flow."

The traffic zone identification algorithm in this study utilizes three categories of traffic conditions as defined by Kerner and Rehborn; 1) "Free Flow" (FF), where vehicles are able to change lanes and to pass; 2) "Synchronized Traffic Flow" (SF), where vehicles are not able to pass as the density is higher than in free flow; and 3) "Traffic Jam" (TJ), where vehicles are moving very slow or at a high density condition.

Peculiarities of "synchronized" traffic flow and traffic jams that are responsible for a complex behavior of traffic were found and discussed in Kerner and Rehborn's previous work. Under the synchronized flow, vehicles move at almost the same speed, on different lanes of the highway. On the other hand, under free flow condition, vehicles in the left (median) lane typically travel at higher speeds than vehicles in the right (shoulder) lane. Under synchronized traffic flow, the average speed of vehicles is noticeably lower and the density is noticeably higher than under free flow. It can be assumed that in synchronized traffic flow, vehicles have limited opportunities to pass.

To account for the uncertainty of this "gray area," the proposed method uses two threshold speeds, namely S_r (FREE threshold speed: boundary between free traffic flow and synchronized traffic flow) and S_b (BREAK threshold speed: boundary between synchronized traffic flow and traffic jam), to define the traffic zone categories. The traffic zones are categorized as follows:

Traffic Zone Category	**Criteria**
Free Flow (FF)	$S_L > S_r$
Traffic Jam (TJ)	$S_L < S_b$
Synchronized Traffic Flow (SF)	$S_b \leq S_L \leq S_r$

Figure 3 illustrates an example of "synchronized traffic flow" in a speed-volume relationship of a sample of speed data; where congested regime is denoted as "traffic jam" and uncongested regime is denoted as "free flow."

Algorithm 3—Queue Type Identification Algorithm

This algorithm determines whether or not a queue exists and if a queue exists, identifies the type of queue. The basic process in this algorithm consists of 3 steps: (1) Initialize all road sections to non-queued sections; (2) identify the existence of queue on each road section; and (3) if a queue exists, identify the queue type on each road section.

Identification of Queue Existence

The existence of a queue is determined on the basis of the traffic condition identified in Algorithm 2 (e.g. free flow (FF), synchronized traffic flow (SF), and traffic jam (TJ)). When a road section is identified as FF, no queue exists. When a road section is identified as TJ, a queue exists. When a road section is identified as SF, a queue exists only if the road sections upstream and downstream are queued, as illustrated in Figure 4(a). In addition, if the road section is adjacent to one or two road sections classified as SF either upstream or downstream within the boundary, it is also classified as "queued." Otherwise, it is classified as "no-queue." The reason for this restriction is the uncertainty of queue existence on a long stretch of SF road sections (i.e. three SF road sections with approximately 1.5 km in length).

Identification of Queue Type

Queue can be classified into the following four categories: (1) head-of-queue, (2) tail-of-queue, (3) in-queue, and (4) inclusive queue. A head-of-queue zone represents the road section immediately downstream of the queue body in which traffic transitions from congested conditions to uncongested conditions. Similarly, a tail-of-queue zone represents the road section immediately upstream of the queue body in which traffic transitions from uncongested to congested traffic conditions. All zones within the boundary of the head-of-queue and tail-of-queue zones are categorized as in-queue zones. If tail-of-queue is not identified, then the current head-of-queue section becomes "inclusive queue" (i.e. queue length is shorter than the length of one road section). The identification process compares a road section with both upstream and downstream road sections as shown in Figure 4(b) and determines the queue type of the road section.

It should be noted that the proposed method is unable to estimate the exact location of a head-of-queue or a tail-of-queue within the section. The level of resolution of location estimates from the proposed method is limited to the spacing between consecutive detector stations.

CALIBRATION METHOD

The proposed method requires the calibration of two threshold speeds, S_r and S_b. The following sections describe the calibration methods employed and the results obtained. The data used to calibrate the parameters were obtained from loop detectors on a 10-km section of the Gardiner Expressway in Toronto, Canada. The expressway is instrumented with dual-loop detectors in all lanes located approximately every 500-meter; the detectors are polled every 20 seconds. Figure 5 illustrates the location of loop detector stations on the Gardiner Expressway.

A sequential calibration process was adapted in which S_b is first calibrated assuming S_r = 90 km/h—the posted speed limit of the freeway. Then S_r is calibrated using the value of S_b obtained in the first step.

This calibration process uses a method called "re-scaled cumulative arrival curves" (*N*-Curves) proposed by Cassidy and Windover (1995). *N*-curves were constructed for several days to avoid the effect of variation of daily traffic. The curve represents the cumulative count of vehicles which pass a given detector station as a function of time. Other than the curve for the most upstream station, the curves of all the downstream stations are shifted by the average free-flow travel time of vehicles traversing from the upstream to downstream station. Average free-flow trip time is determined based on the free-flow speed of the freeway and the length of the road section(s). Thus, when a queue does not exist on a roadway, the two curves at two successive detector stations will overlap each other. The vertical displacement of cumulative count curves between two successive detector stations can be interpreted as the excess vehicle accumulations (i.e. the number of vehicles in a queue) due to traffic delays. However, the accumulations are not easily noticeable as they are much smaller than the cumulative number of vehicles that have passed the detector since the start of the analysis period. A critical element of *N*-curves is the re-scaling of the cumulative curves to accentuate queue formation. Curves are re-scaled by subtracting the number of vehicles in which time interval equal to a constant flow rate times the time interval (called “background flow”). At the time where excessive vehicle accumulation is found, the time and the location of the queue formation can be identified. However, the constraint with *N*-curves is the road section of any two consecutive stations must not contain any on-ramps or off-ramps if the volumes on these ramps are not known (Cassidy and Windover, 1995; Cassidy and Bertini, 1999).

To evaluate the accuracy of the proposed queue tracking method, the time of queue formation and the queue type were identified using the *N*-curves method and compared to those predicted from the proposed method. The performance of the proposed method was evaluated by comparing the estimated results with the *N*-curves results within the studied period on a minute-by-minute basis. In order to check if the proposed method correctly detects the transition of queue types, the time when observed queue type changes were compared (e.g. from no-queue to head-of-queue). However, it should be noted that the queue types at the most upstream and the most downstream road sections within the boundary of the *N*-curves are not known with certainty. Thus, for the most upstream road section, if either a tail-of-queue or in-queue condition is identified using *N*-curves, it is also considered correct if either a tail-of-queue or in-queue condition is estimated by the proposed method. Similarly, for the most downstream road section, if either a head-of-queue or in-queue is identified using *N*-curves, it is also considered correct if either a head-of-queue or in-queue is estimated by the proposed method.

CALIBRATION RESULTS AND DISCUSSIONS

Example of the Proposed Method Estimation (Calibration of S_b)

The *N*-curves were constructed for 6 hours (2:00 p.m. – 8:00 p.m.) on Tuesday, January 20, 1998 using loop detector data at the stations between Bathurst Street and Dufferin Street (from Station dw0050dwg [Sta. 50] to dw0080dwg [Sta. 80]) on the westbound Gardiner Expressway as shown in Figure 5. The *N*-curves results and the queue status estimated by the proposed method are shown in Figure 6 and 7, respectively. In Figure 6, the *N*-curves were plotted with the background flow of 5300 vehicle/hour. Figure 7 shows the time and location of the queue formation and dissipation estimated by the proposed method. This figure is referred to as the "queue status" diagram. From the queue status diagram, one can easily identify the queue location and queue type at any location and time specified within the study scope.

In comparing Figure 6 and 7, the similarity in the pattern of queue formation and dissipation between *N*-curves and the proposed method was observed. From 2:00 to 2:10 p.m., the *N*-curves at the four detectors are almost overlapped with each other and this indicates that there was no queue during this period. Similarly, the proposed method also indicates that no queue existed between 2:00 and 2:13 p.m.

In Figure 6, at 2:10 p.m., the *N*-curves at the most downstream detectors of the section, Sta. 70 and 80, start to diverge from the curves at the other two upstream detectors. This indicates that a tail-of-queue exists between Sta. 60 and 70. At approximately the same time, the proposed method indicates a queue formed between Sta. 50 and 80, where a tail-of-queue is located between Sta. 50 and 60.

At 2:40 p.m., the curve at Sta. 60 is separated from the curve at Sta. 50. This indicates that a tail-of-queue had spilled over Sta. 60. After 3:00 p.m., the spacing between curve 50 and 60 becomes constant and this implies that the volume on the section between Sta. 50 and 60 reached capacity - i.e. in-queue. This in-queue condition lasted for approximately four hours. The proposed method also shows similar traffic queue conditions during the same time period.

The on-ramp, approximately 1.15 km downstream of Sta. 80, is closed daily from 3:00 to 6:00 p.m. When the ramp is opened after 6:00 p.m., severe congestion normally occurs due to heavy traffic volume entering from the on-ramp. At 5:55 p.m., the proposed method clearly reflects this situation by showing that another head-of-queue formed downstream of Sta. 80 and it merged with the existing head-of-queue upstream of Sta. 80.

Between 6:52 and 7:03 p.m., curve 60, 70, and 80 are suddenly overlapped with each other. This indicates that there was queue dissipation for a short time period. Curve 50 is separated by a constant vertical displacement with the other three curves. This indicates that a head-of-queue formation between Sta. 50 and 60. Around 6:56 p.m., the proposed method indicated a head-of-queue was located between Sta. 60 and 70. The reason for this change in traffic queue condition is an accident that occurred at 6:48 p.m. downstream of Sta. 60. This accident reduced the capacity and restricted the flow of traffic upstream of Sta. 60. Consequently, the queue pattern upstream and downstream of Sta. 60 changed. At this time, a head-of-queue was formed immediately downstream of the accident location. This was clearly reflected in the results from both the *N*-curves method and the proposed method.

After 7:03 p.m., curve 60 is separated from curve 70 by a relatively constant number of vehicles. The other curves are also separated by a constant number of vehicles. This represents the re-formation of a queue after the clearance of the accident. The proposed method displays the similar traffic pattern at 7:08 p.m.

At 7:38 p.m., curve 50 and 60 overlap again. This represents that the queue dissipated on the section between Sta. 50 and 60 (a forward recovery shockwave). At 7:40 p.m., curve 60 and 70 also overlap. This represents queue dissipation on the section between Sta. 60 and 70. The proposed method also estimated queue dissipation on these two road sections at 7:32 p.m. and 7:35 p.m., respectively.

To calculate the accuracy of the estimation, queue types (over 1,000 one-minute queue types) were compared between *N*-curves and the proposed method on a minute-by-minute basis for the three selected road sections between 2 p.m. and 8 p.m. As a result of the comparison, when S_b is 83 km/h, the accuracy of the proposed method is maximized at 90%.

Sensitivity Analysis

Similar analysis using *N*-curves was performed on three other days on the same road section. The results are summarized in Table 1. At accuracy of 85% or more, S_b varies within the range of 64-90 km/h range and average accuracy is 88.4%. The mean S_b over four days is 78 km/h and this speed is assumed to be the fixed S_b for this freeway.

To validate the suggested S_b of 78 km/h, it should be compared with the "true" results. The sensitivity of the accuracy of the proposed method to the selection of S_b was examined by comparing the results using the daily optimal S_b (i.e. S_b with the highest accuracy) and the suggested S_b of 78 km/h. The difference in the accuracy between the results using different mean S_b for different days and the results using the one-fixed S_b of 78 km/h for all days are summarized in Table 1. The difference in accuracy (i.e. average queue identification error) is only 0.65%. Due to this small difference, it is reasonable to assume that the same S_b can be applied to all dates. In facts this is more preferable for practical application than applying different S_b on different days.

S_r is then calibrated with S_b set to equal at 78km/h. The same calibration process is used and S_r is determined to be 86km/h. The results are summarized in Table 2. Coincidentally, S_r with highest accuracy is the same on the four days and the average accuracy is 85.4%.

To understand the implication of S_b and S_r in the context of traffic flow fundamentals, the relationship on the speed and volume relationship on the four days was plotted and fitted with a traffic flow model. This study uses Van Aerde Model (Van Aerde and Rakha, 1995) due to its high accuracy of estimation. The Van Aerde Model is a single regime 4-parameter non-linear model of speed-flow-density relationships. All of the road sections tested for the four days show the same traffic characteristics with the same S_c (speed at capacity), S_f (free-speed; not S_r), V_c (flow at capacity), and D_j (jam density). S_c for these four dates is estimated to be 78km/h and it is always same as the calibrated S_b, as shown in Figure 8.

This analysis suggests two important findings: (1) S_b of the proposed method essentially represents the speed at capacity (S_c); (2) Synchronized traffic flow, or the "gray area", falls within the uncongested regime of the speed-volume relationship, rather than being partially overlapped with both congested and uncongested regime.

CONCLUSIONS AND RECOMMENDATIONS

The method presented in this paper has the following advantages over the existing queue-tracking models. First, the proposed method uses average link speed as a basis of queue identification because they reflect traffic condition within road sections better than spot speeds. Second, unlike existing models, threshold speeds are determined based on experimental results rather than analysts' subjective assumptions. Third, the proposed method avoids manual interpretation and therefore can be used in real-time; this is the key feature of the proposed method and it provides the convenience in computerization.

The accuracy of the estimates from the proposed method was generally high in spite of a relatively simple model structure. From the comparison of estimation errors, it was found that the proposed method can identify traffic queues on the selected freeway at an accuracy of 85%. Furthermore, this method can be integrated with other ITS applications, including ATMS (Automated Traffic Management Systems) and ATIS (Automated Traveler Information Systems).

In future studies, the following tasks will be performed:

- In order to check the transferability of the proposed method, other urban freeways where recurrent traffic congestion occurs will be examined with the proposed model.
- It appears that on-ramp and off-ramp on freeways do not significantly affect the queue identification process in the proposed method because the average link speed reflects the effect of traffic delay caused by the traffic entering from/exiting to ramps; this must be confirmed in future studies.
- The effect of detector spacing on the performance of the proposed method will be considered as detectors are sparsely located in inter-urban freeways;
- The traffic data, other than average link speed, (such as occupancy data) will be considered as a basis of determining queue types.

ACKNOWLEDGEMENTS

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TABLE 1 Comparison of S_b for Different Days

Date	Range of S_b (km/h) with Accuracy > 85%	S_b (km/h) with Highest Accuracy	Highest Accuracy (%)	Accuracy (%) of Fixed S_b (78 km/h)	Difference in Accuracy (%)
January 20, 1998	70 – 90	83	90.2%	89.2%	1.03%
December 7, 1998	74 – 86	77	87.1%	86.8%	0.32%
December 10, 1998	64 – 90	71	92.2%	91.5%	0.76%
May 14, 2001	77 – 79	79	86.5%	86.1%	0.49%
Average		**78**	**89.0%**	**88.4%**	**0.65%**

TABLE 2 Comparison of S_r for Different Days

Date	Range of S_r (km/h) with Accuracy > 85%	S_r (km/h) with Highest Accuracy	Highest Accuracy (%)
January 20, 1998	79 – 89	86	86.7%
December 7, 1998	78 – 95	86	86.8%
December 10, 1998*	78 – 95	86	81.5%
May 14, 2001	81 – 94	86	86.6%
Average		**86**	**85.4%**

* The highest accuracy for December 10, 1998 is under 85%; hence, the range of S_r at the highest accuracy (81.5%) is used to calculate the average.

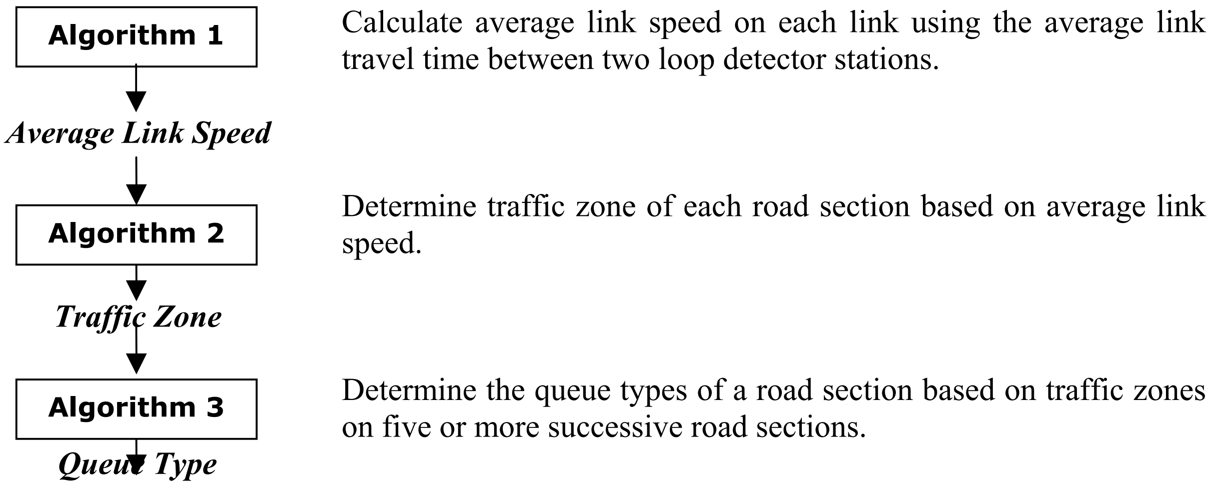

FIGURE 1 Structure of the proposed method.

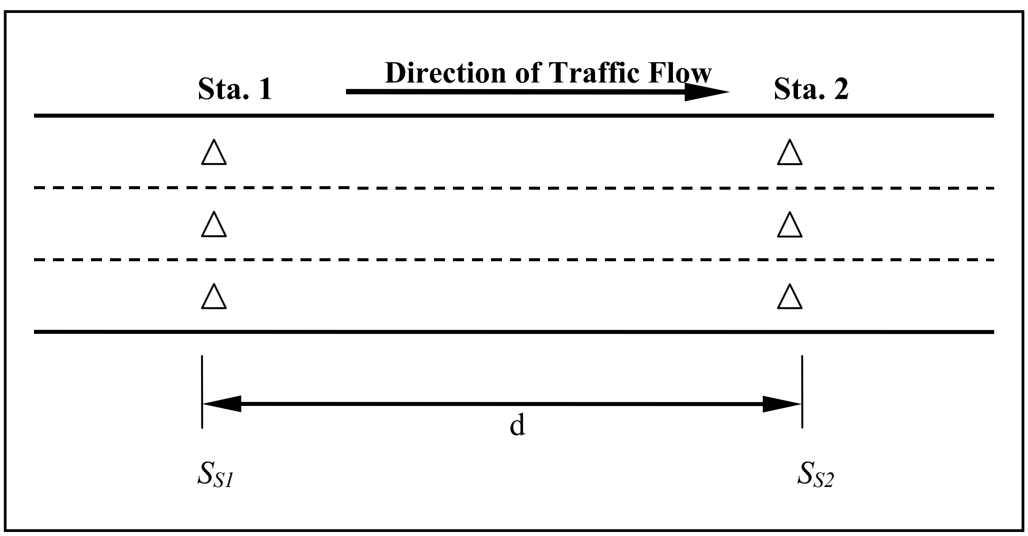

FIGURE 2 Schematic drawing of a roadway segment.

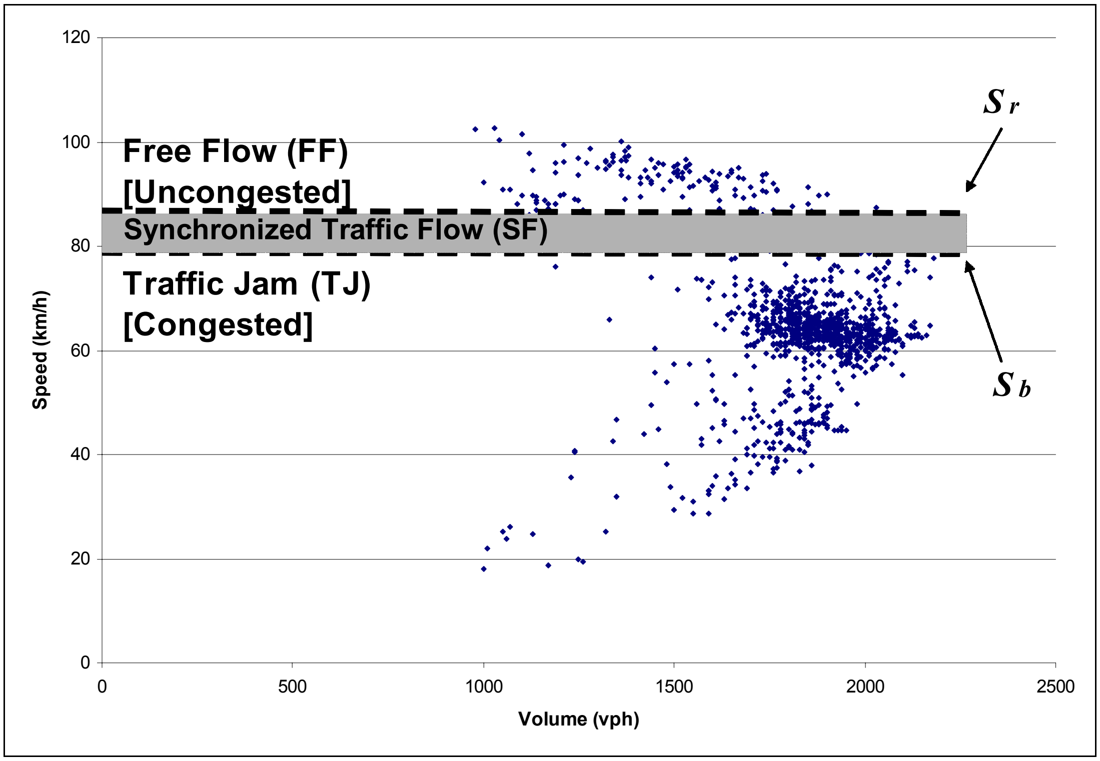

FIGURE 3 Example of Synchronized Traffic Flow in a speed-volume relationship.

Note:

1. Speed data in this figure is the S_L (Equation 3) computed for the road segment between Station dw0070dwg and dw0080dwg on the Gardiner Expressway on January 20, 1998 between 2 p.m. and 8 p.m.

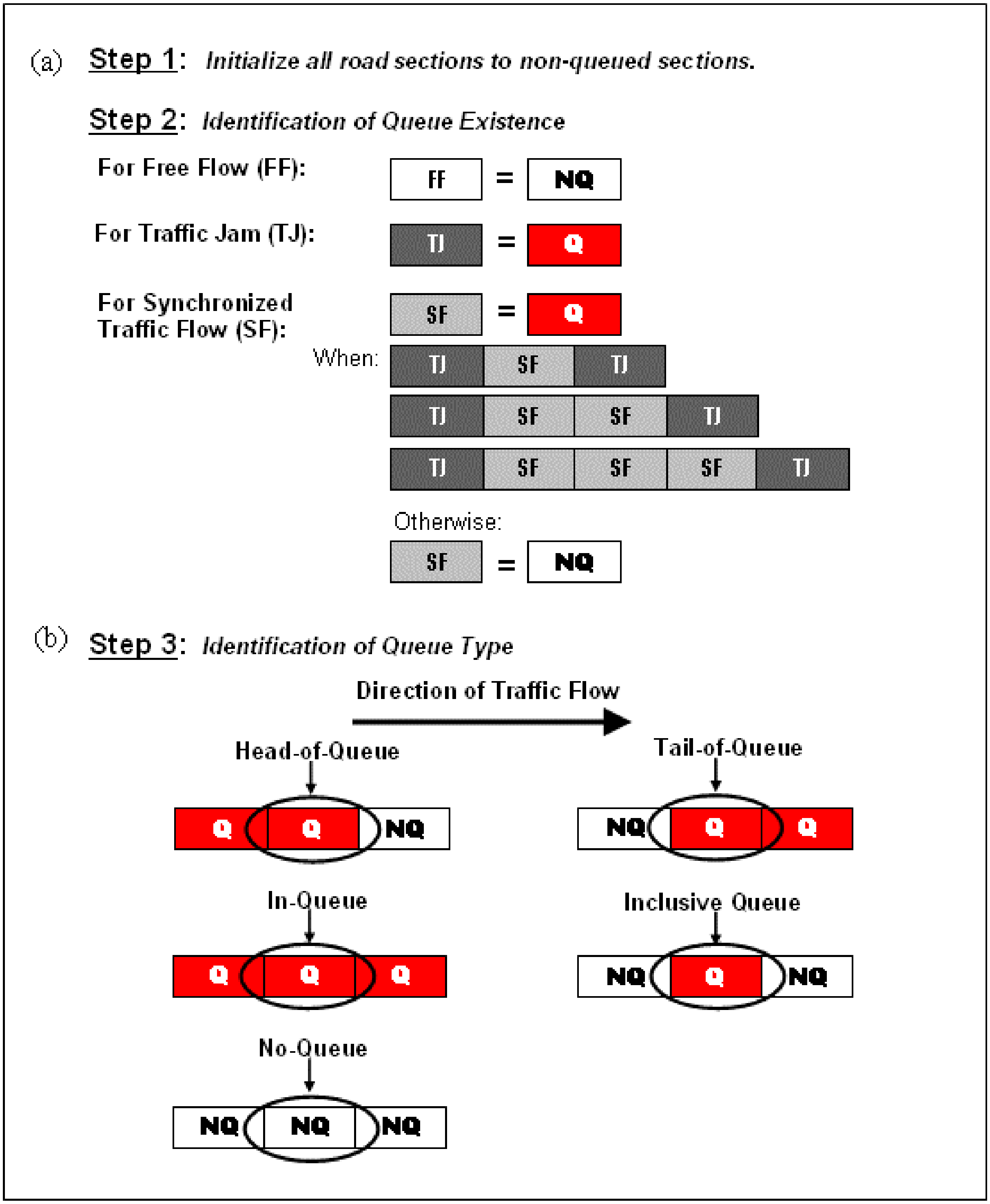

FIGURE 4 Stepwise procedures of queue type identification: (a) Step 1 & 2: Queue existence identification; (b) Step 3: Queue type identification.

Note:
1. Queue Existence Identification: Q = Queued; NQ = No-Queue

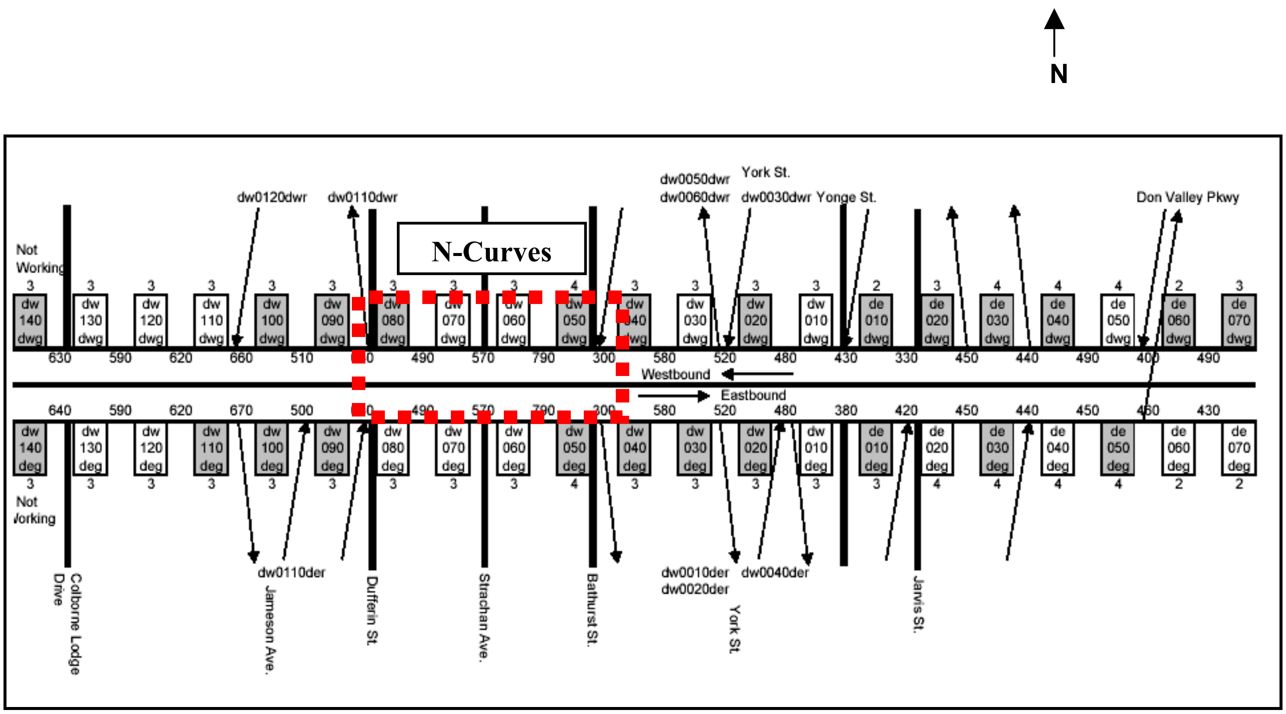

FIGURE 5 Schematic drawing of Gardiner Expressway, Toronto, Canada.

Note:
1. The arrows pointing outward indicate off-ramps and the arrows pointing inward indicate on-ramps.
2. The letters inside the squares denote the station ID.
3. The numbers shown above or below the station ID are the number of lanes.
4. The numbers shown between two successive detectors are the distance in meter.

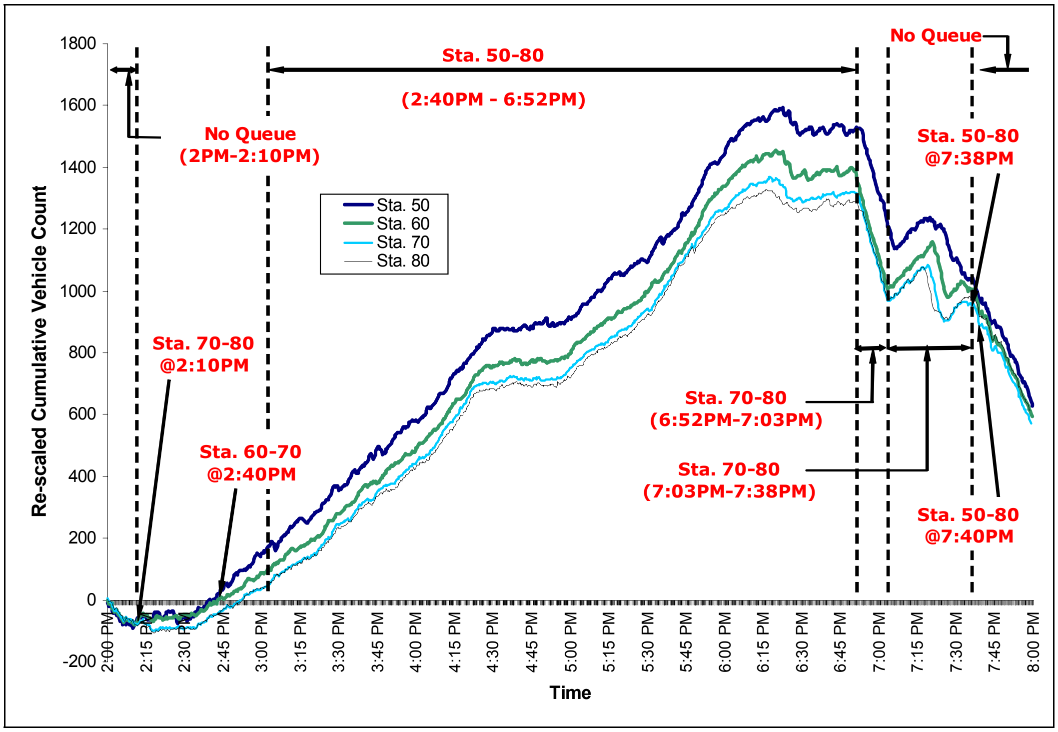

FIGURE 6 ***N*-Curves for afternoon peak period at Sta. 50 to Sta. 80 (January 20, 1998).**

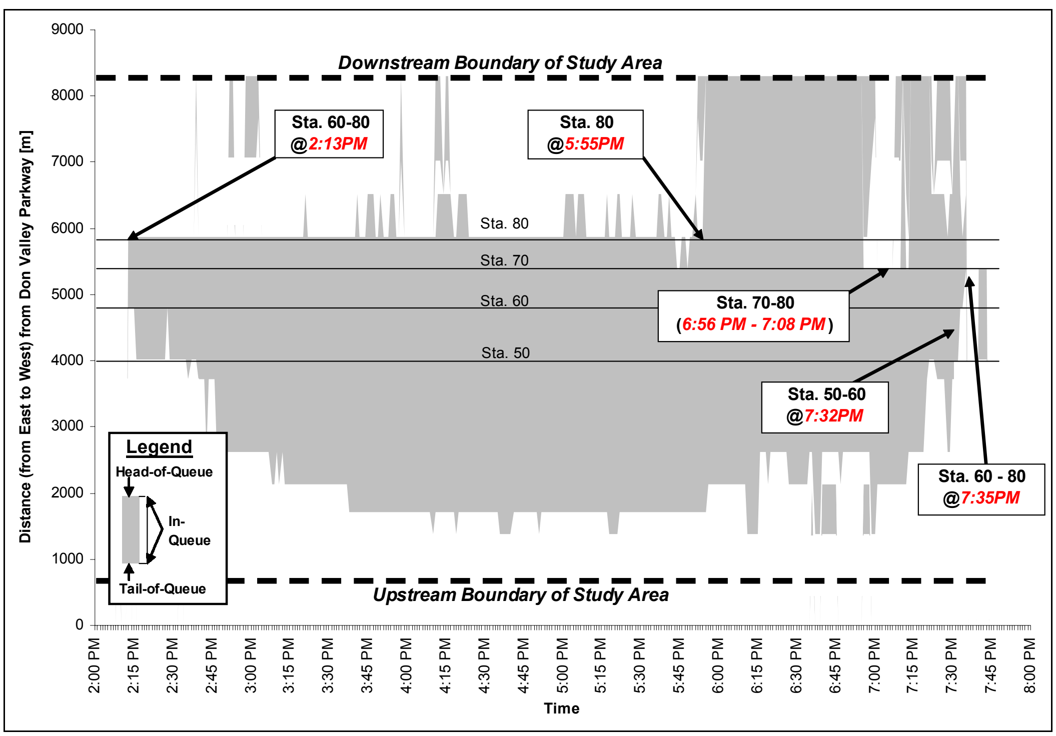

FIGURE 7 Queue Status Diagram for afternoon peak period (January 20, 1998).

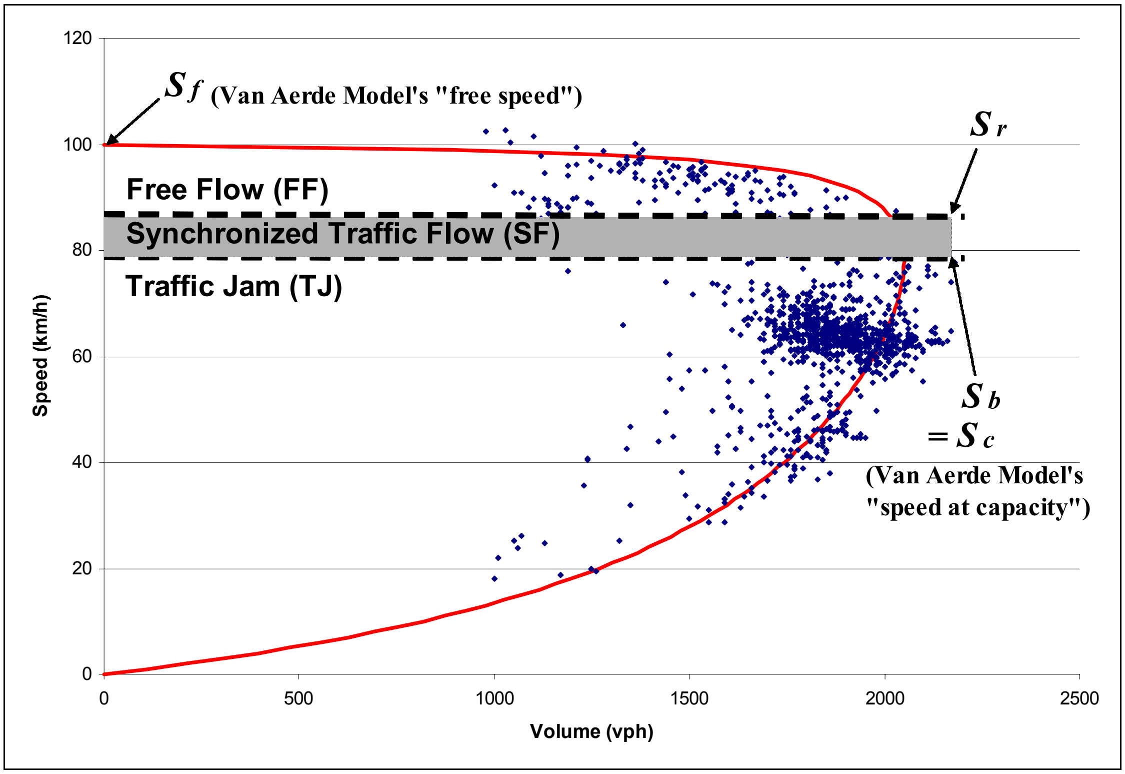

FIGURE 8 Consensus of parameters between Van Aerde Model and the proposed method.

Note:
1. The data in this figure is the same as those in Figure 3.