IMPROVING ATSPM'S APPROACH DELAY CALCULATION

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ABSTRACT
There are more than 330,000 traffic signals in the United States, at least 75% of which could be significantly improved by updating their equipment or timing plans, reducing unnecessary delay to motorists. A major reason for this excess delay is poor traffic signal timings that account for nearly 300 million vehicle-hours on major roadways alone. Automated Traffic Signal Performance Measures (ATSPM) is a recent development which provides performance measures that can be used for decision making and for the selection of more effective operational strategies that could reduce delays and costs. However, the delay estimation method of ATSPM is extremely simplified and ignores some of the fundamental components of control delay experienced by motorists at a traffic signal. The team proposes three alternative methodologies for the estimation of approach delay, each of which addresses one or more of the drawbacks currently present in the ATSPM framework. The cumulative arrival/departure method uses upstream and downstream detections and incorporates stopped, deceleration, and the acceleration delays into the approach delay. The departure-only method strictly uses the actuations at the departure detectors to estimate the approach delay, comprising both stopped and initial queue delays. The shockwave method uses arrival and departure detections to estimate the approach delay. The stop component of the approach delay is fully incorporated in the estimates by this method. Results of the proposed methodologies reveal promising improvements over ATSPM's conventional delay framework.

Keywords: Approach Delay, Arrival/departure Curves, Shockwave, ATSPM, Detector

INTRODUCTION

Traffic signals, though a very critical part of today's traffic control system, can be a major cause of chronic congestion and safety hazard on roads if not operated efficiently. In the United States, there are more than 330, 000 traffic signals according to the United States Access Board (FHWA-ATSPM, 2019). Because of the aging control infrastructure and lack of proper assessment, many of these signals are not performing up to their standards. In addition, the lack of tools and frameworks to assess traffic signals based on actual phasing and detector data has been a notable limitation. Consequently changes in time variant traffic demand in terms of overall traffic volume, peaking time, and arrival patterns are not incorporated in their assessment. Hence, it is important to construct an advanced framework to evaluate the performance of signals based on measured traffic signal and detector data.

Automated Traffic Signal Performance Measures (ATSPM) is one such framework. It collects high resolution signal and detector data and generates performance metrics for approaches at signalized intersections (FHWA-ATSPM, 2019). These metrics include but are not limited to total and average delay, split failure, and yellow and red actuations. These metrics help improve decision making and the selection of more effective operational strategies to reduce congestion and crash risk. However, there are important limitations in the ATSPM methods for generating delay and other performance measures. The approach delay reported by ATSPM does not include acceleration, deceleration, and queueing delays. Moreover, the delay estimation process does not account for any initial queue and oversaturation. Hence, it is very likely that ATSPM's reported delays are underestimated, thus prompting the need for further improvements.

In this study, three algorithms have been developed to estimate approach delay at a signalized intersection using high resolution signal and detector data. Each of these algorithms addresses at least one limitation in the current ATSPM delay estimation method. High resolution traffic data from the Utah Department of Transportation were provided for two corridors for the year 2018. The signal phase and detector actuation data were fused, and fundamental traffic flow relationships were applied to develop these methods, along with some reasonable assumptions. The three methods and the existing ATSPM delay estimation methods are applied to a thru movement approach at a signalized intersection located at the 700 East Corridor in Salt Lake City (SLC), Utah. Delay estimated by all the three methods are compared and its variation by time of day are portrayed.

The remaining part of the paper is organized as follows. The next section presents a review of past studies on delay estimation methods at signalized intersections. Following that, the underlying method for each of the three algorithms is described. Then, the results from the application of these methods to a signalized intersection approach in SLC are presented. In the last section concluding remarks, limitations of the methods, and scope for future research are provided.

LITERATURE REVIEW

In the evaluation of operational performance of intersection, delay serves as an important measure of effectiveness. However, as an accurate computation of delay is challenging, it is a common practice to estimate delay based on the vehicle queue length. The majority of queue length estimation methods can be classified into two approaches: the input-output and shockwave based models. In the input-output models, vehicle accumulation is analyzed at the stop-line of the intersection. The shockwave models account for the variation of traffic stream properties during the queue formation and dissipation. They can also account for temporal and spatial information during the queuing process.

Studies of shockwave theory to estimate queue length and delay began in the early 1980s. Michalopolous (1981) firstly developed a real-time control policy minimizing the total intersection delays based on queue length constraints. The study analyzed an isolated intersection rather than a system of signalized intersections and the proposed model was tested with real world data. The analysis result showed that the suggested method can generate accurate results during the peak hours. Pisharody (1981) proposed analytical and mathematical procedures to estimate delay for system of coordinated signalized intersections. It showed, using numerical examples, that the shockwave models or macroscopic models in general can yield more accurate results than conventional input-output models.

With the advancement in technology, models that utilize detector data for delay estimation were developed. ATSPM has also gained popularity in terms of using high-resolution data at signalized intersections for performance estimation. In the Georgia DOT report, America (2016) discussed the analysis results using ATSPM and reported interpretation for all the attributes of performance measures. Liu et al. (2009) used high resolution data collected from SMART-SIGNAL in Minnesota to estimate real time queue lengths. The study suggested three models: a classical Lighthill-Whitham-Richards (LWR) model and two models with minor changes in the basic LWR model. The analysis results implied that the classical LWR model yielded the most accurate estimation results when it is used with high-resolution data. The mean absolute percentage error (MAPE) was about 7% for the classical model and 15% and 21% for two modified models, respectively. Although the suggested model yielded a minimum error, the lack of available high-resolution makes its use limited. Cheng et al. (2011) developed a methodology for cycle by cycle queue length estimation for signalized intersections. Vehicle trajectory data were used as an input and Critical Points (CP) are defined based on the trajectory. The critical points represent different states of vehicles in the queue and can be used for queue estimation. The proposed methodology was tested with real world data and the results yielded a MAPE of 20% on average. The proposed model is simple to implement and can be used in the real world but the error is considerably high. In cases of oversaturation, the error goes up to 25%. In essence, the findings of the literature review imply that there are several applicable models for delay estimation, all of which have their own advantages and limitations. Also, the analysis results were highly dependent on the type of data used and the assumptions related to it.

Incremental Queue Accumulation (IQA) Method

The performance measure of an intersection depends on several criteria such as delay, queue length and number of stops, among which delay is considered to be the most important in determining the level of service (LOS). One of the most widely used methods for delay estimation is the Queue Accumulation Polygon (QAP) method proposed in the Highway Capacity Manual (HCM). Consistent with the uniform delay model, the HCM method (Transportation Research Board, 2016) assumes that the QAP has a triangular shape (shown in Figure 1). However, the assumption does not hold true in real-world scenarios as vehicle arrivals and departures may vary within a cycle. To address the limitation, the previous studies developed several alternative methods that accounts for the non-uniform pattern of arrivals and departures of vehicles.

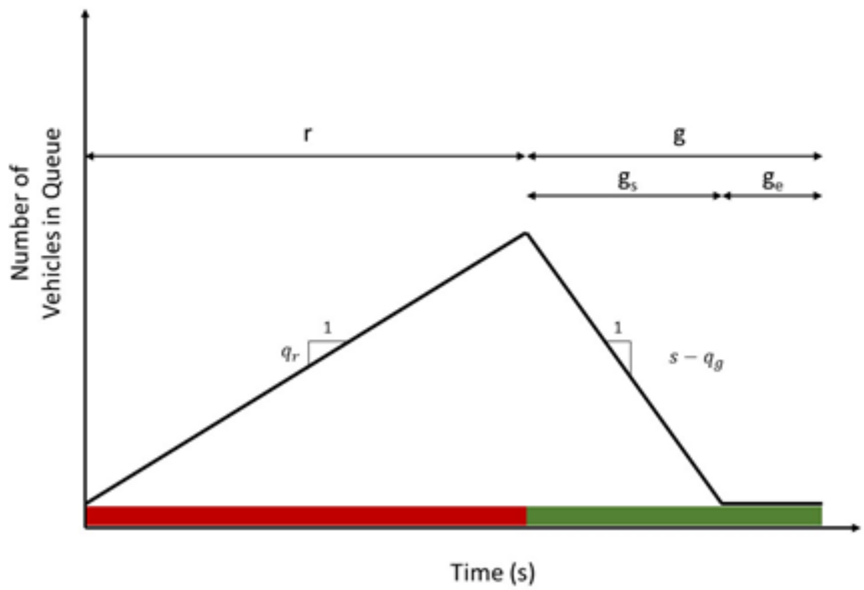


FIGURE 1 Queue Accumulation Polygon (QAP) in HCM

Robertson (1969) first proposed an analytical framework to account for the pattern of arrivals and departures at a signalized intersection. The suggested method uses a flow profile diagram which can be estimated by the arrivals and departures in short time steps of 2-4 seconds.

Strong et al. (2005) modified the flow profile diagram and proposed the Incremental Queue Accumulation (IQA) method, which is an alternative calculation method for HCM first-term delay model. IQA calculates the areas under short time slices of the QAP rather than using the uniform delay formula to calculate the area under QAP. Thus, the IQA method removes a limiting assumption of the QAP formula. The study demonstrated that the IQA method yields more understandable, accurate, and applicable results for delay calculation over a broader range of conditions. Strong and Rouphail (2006) proposed an extended approach of IQA method that can account for non-uniform arrival rates affected by platooned arrivals. The proposed method yielded higher accuracy and flexibility compared to the progression factor method suggested in the HCM.

Kyte et al. (2008) validated the IQA method using NGSIM trajectory data and compared its delay estimates with field measurements and the HCM method. Field delays were estimated using travel time between the arrival and departure points at the intersection. IQA method estimates delay using a queue accumulation polygon by calculating the number of queued vehicles in each time period. The method incorporates non-uniform arrival rates by matching the vehicles upstream (at an upstream detector) and downstream (at the stop bar detector), yielding more accurate results compared to the HCM delay method.

METHODOLOGY

Data Preparation

The dataset provided as part of the ATSPM project includes high-resolution vehicle detection data. Those data are provided by the Utah DOT for two corridors: corridor-1: 300W (US-89) and corridor-2: 700E (UT-71). Data were prepared for an intersection in corridor 2 located at 700 East and 500 South using both detector and signal timing (phase) data. Data have event-based time stamps at 0.1 second resolution, consisting of two variables namely an event parameter and an event code. Event parameter represents a detector or phase ID and Event code represents the condition of detector or a phase whether it is "on" or "off". Using these two variables, an R script was used to extract cycle by cycle data of vehicle arrivals and departures.

Method 1: Cumulative Curves Based Approach

Basic Principles of IQA

The IQA method estimates the length of queue using arrivals and departures information in short time steps. The method adds and subtracts vehicle arrivals and departures during each time step to the queue at the start of the time step. Figure 2 illustrates the queue length computed at time step i, which is the difference between accumulated arrivals and departures. The time steps are defined whenever a vehicle arrives or departs, thus designating a change of status in the queue size.

Subsequently, the total delay experienced in a cycle can be calculated by summing all the products of queue size and time steps. The corresponding average vehicle delay per cycle can be computed by dividing the total delay by the number of vehicles departing in a cycle. As such, the IQA method computes the QAP area of trapezoids representing the total time during the cycle where the inflow and outflow rates are not equal.

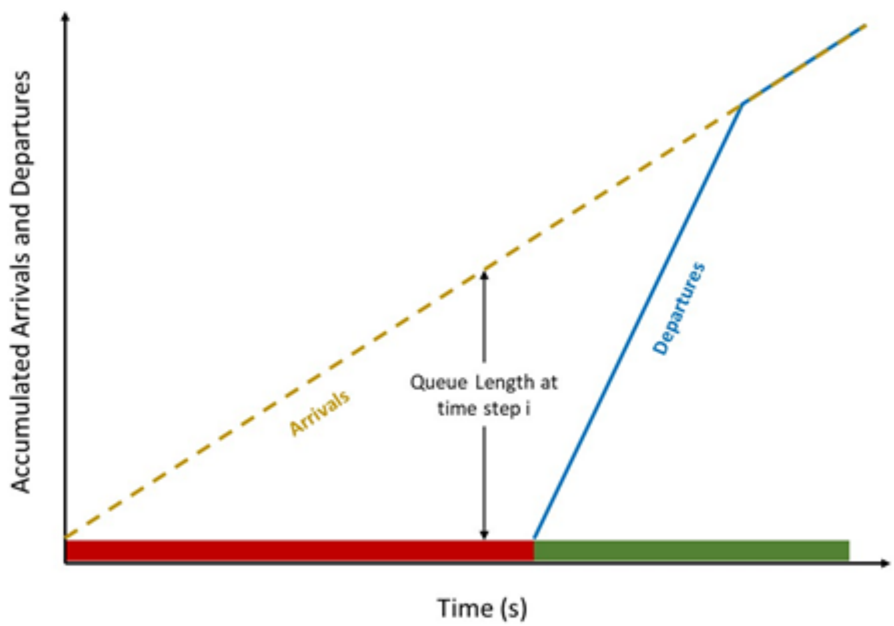


FIGURE 2 Queue Length during Time Step (i)

Application with ATSPM Data

Prior to the application of the IQA method with ATSPM data, it is necessary to identify what types of data are available to use. Figure 3 depicts a time-space diagram that shows vehicle trajectories approaching an intersection. The figure can be drawn only when every vehicle trajectory is recorded. ATSPM data, however, includes only vehicle observation records that are detected at the upstream and stop-bar detectors--- which are shown as dotted lines in the figure. As such, for the IQA method, it is essential to tie the observations at both the upstream and stop-bar detectors.

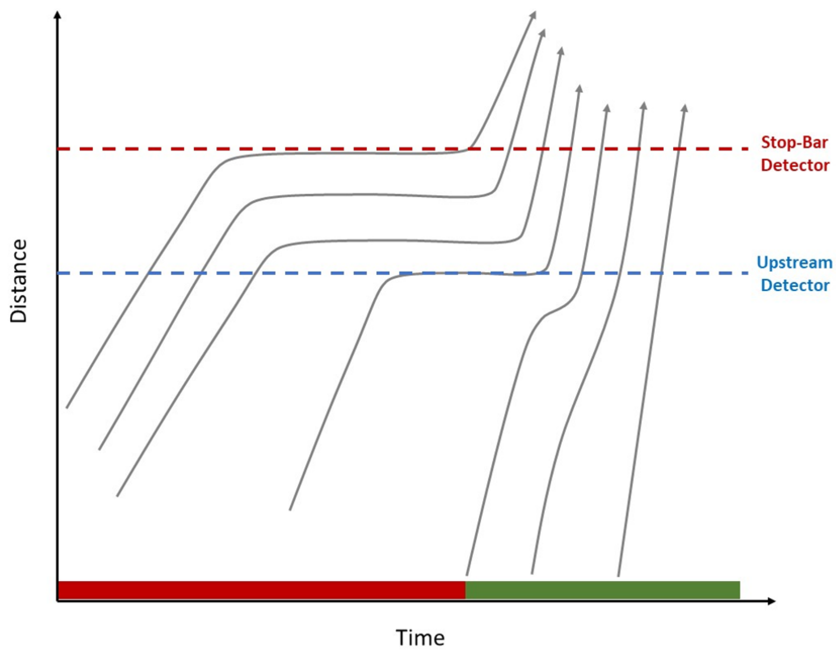


FIGURE 3 Time-Space Diagram for Vehicle Trajectories

This methodology assumes stable operations of the traffic signal (i.e. departure curve catches up with the arrival curve before the end of a cycle). Hence, there will be no queued vehicles present at the end of the green phase. It further assumes that arrivals and departures obey the FIFO (first-in-first-out) principle. This means that vehicles depart from the stop bar in the same order as they have arrived at the upstream detector. These assumptions enable the analyst to tie the arrivals at the upstream to the departures from the stop bar.

With the refined data, vehicle arrivals-departures curve can be drawn as in Figure 4, which illustrates the accumulated vehicle arrivals to time. In the figure, the orange and blue lines are vehicle arrivals detected at the upstream and stop-bar detectors, respectively. The horizontal time gap between two lines is the travel time of a vehicle between the two detectors. To compute the delay for vehicle using the arrival and departure time, the appropriate free-flow travel time needs to be determined.

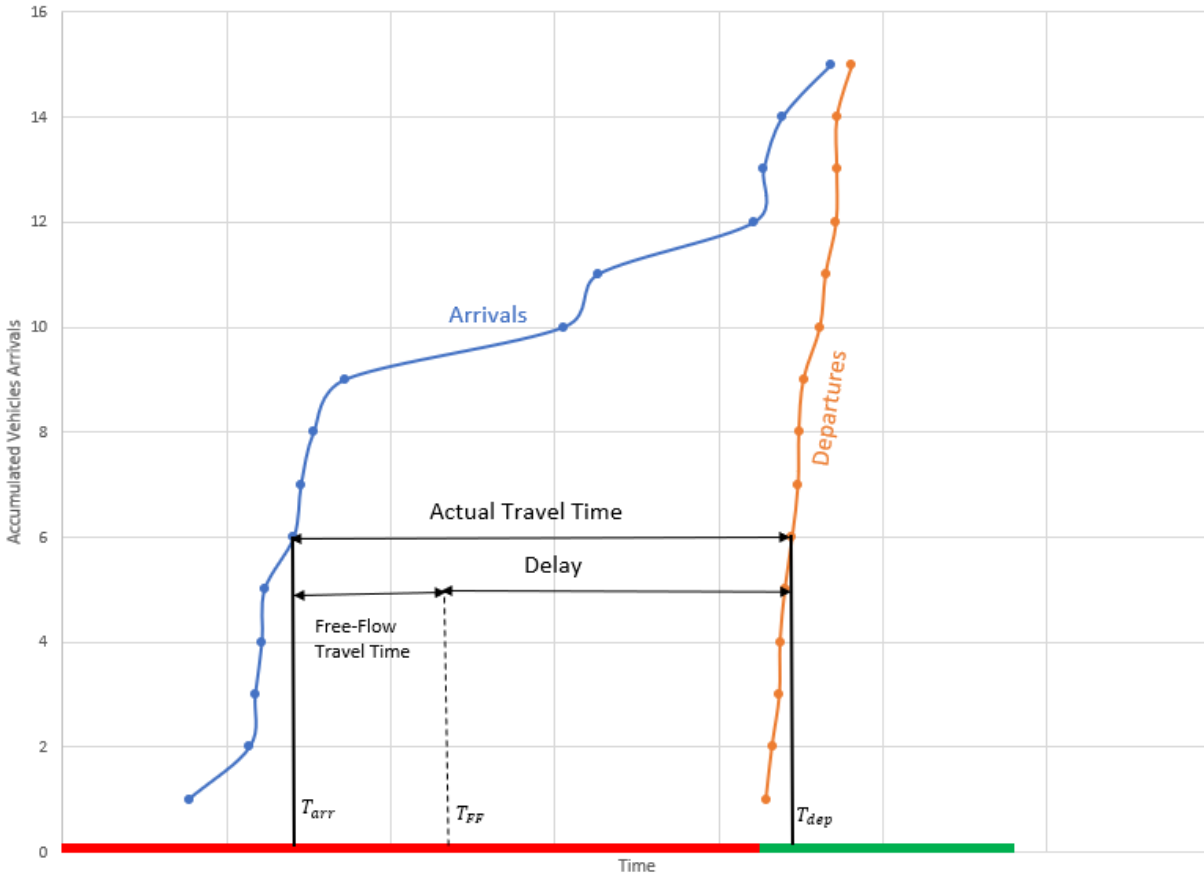


FIGURE 4 Accumulated Vehicle Arrival and Departure Time

To estimate the delays for individual vehicles, the expected departure time under free-flow conditions is calculated by adding the free-flow travel time to the observed arrival time (see Figure 4). Free flow travel time is estimated for each vehicle as described below:

$$T_{FF} = \frac{L}{Speed\ Limit} \qquad \text{Eq. (1)}$$

Where,
T_{FF} = Free flow travel time (in seconds)
L = length between upstream and downstream detectors (ft)
Speed Limit = Posted Speed Limit (in ft./sec)

Thus, delay can be computed by subtracting the expected free-flow departure time from the observed departure time as shown in the figure. The total experienced delay can be computed by adding up all the delays and the average experienced delay per vehicle can be calculated by dividing the total delay by the number of vehicle departures.

Method 2: Delay Based on Exit Sensor Data Only

This method employs actuation data only from the lane-by-lane detectors located at or just downstream of the stop line of an intersection approach. It leverages the changes in headway of departing vehicles when all queued vehicles are served and the un-queued vehicles start arriving at the intersection. Two thresholds are implemented to classify queued and un-queued vehicles, which enables the estimation total delay per cycle and average delay per vehicle based on reasonable estimates of arrival volumes

<u>Assumptions:</u>

1. Vehicles departing from a queue are likely to have inter-departure times or headways that are close to the saturation headway.
2. A large gap between two successive departures indicates that the queue may have cleared and the remaining vehicles will not experience any delay.
3. The arrival rate is assumed to be uniform during the green and red periods, except for some special cases that are described separately.
4. Multiple lanes of a movement group operate independently and their delay can be aggregated at the movement level.
5. No spillback queues occur from any downstream intersection

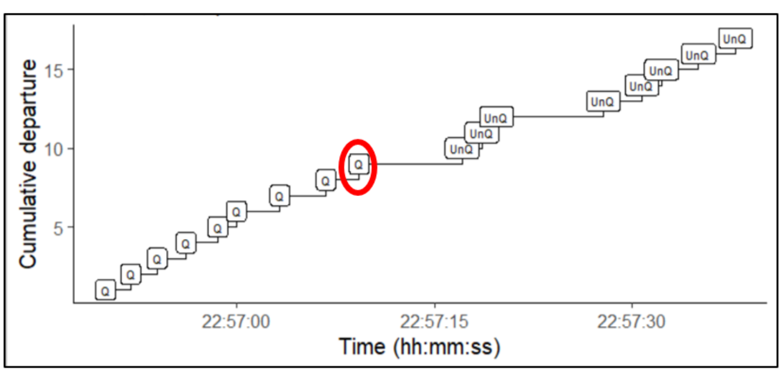
Inputs

At the end of each cycle (end of green time) the phase time information and actuation (detector "on") events from the stop bar detectors is archived.

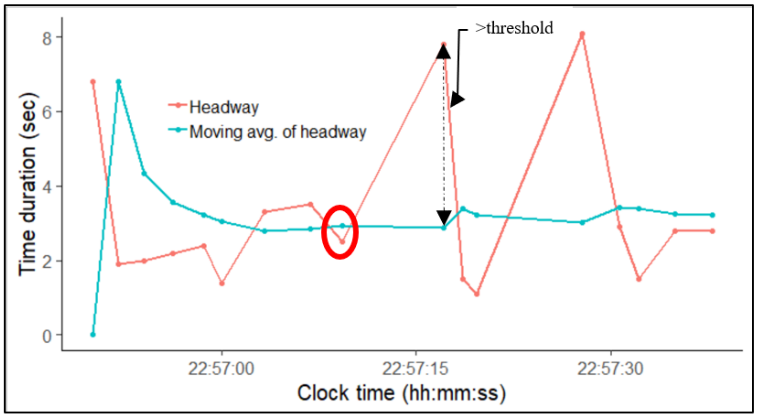
Process

In the subsections below, the steps required to apply this method to each lane of a movement group are described. It assumes that there is no residual queue, and that both queued and un-queued vehicles are served by the lane during its green time. Following this, the applicability of this method to some special cases are discussed when any of these conditions are violated.

Step 1: Queued and Un-queued Vehicle Identification: Figure 5 (a) shows a typical cumulative departure plot for a lane within a cycle. It is evident that, had the first vehicle departed from a queue, the queue cleared with the departure event circled red. However, this classification of queued and un-queued vehicles is not trivial because the saturation headway for queued vehicles is a stochastic parameter. To address this issue, a moving average of headways of the departing vehicles are calculated as shown by the cyan line in Figure 5 (b). Queue clearance is declared when the difference between this moving average and an observed headway (red line in Figure 5(b)) exceeds a pre-specified numerical threshold.



(a)



(b)

FIGURE 5 (a) Cumulative departure of queued and unqueued vehicles from a lane during green time for that movement (b) Variation of individual and moving average headways during green time

If Nq is the number of queued vehicles, then the time to clear the queue from the start of the green (g_q) is estimated using the following equation.

$$g_q = h_{av} + \sum_{i=1}^{N_q} h_i \qquad \text{Eq. (2)}$$

Where,
h_{av} is the average headway of the queued vehicles,
$i=1, 2, 3, \ldots N_q$ is the count of departing vehicles from a queue.

Here, the headway of the first vehicle is calculated as the time difference between the start of the green time and its departure time.

In a cycle where there are no queued vehicles present at the beginning of green, the total delay should be zero. There are several ways to identify such a scenario. One way is to use the headway for the first vehicle – a large value for this headway would indicate that this vehicle arrived during green and all the vehicles in the lane did not experience any delay in that cycle. However, the major issue with this threshold is that it is sensitive to the latency in the detector activation. Moreover, the presence of a queue from a closely spaced downstream intersection may affect the selection of this threshold (see Prosser and Dunn (1994)). Another way is to estimate the speed of the first and subsequent vehicles from the stop-bar detector or by other feasible means. Based on those indicators, it can be decided whether the first vehicle departed from a queued or an un-queued condition.

Step 2: Arrival Rate Estimation: Figure 6 shows the estimated cumulative arrival of vehicles during red and green (dashed lines) and the departure (solid line) from a lane within a cycle. Since only departure events are known, the solid lines are measured from those events while the dashed lines need to be estimated. The saturation flow rate (s) can be calculated as the inverse of the average headways for all the queued vehicles.

The arrival rate during green (q_g) is estimated using the following equation.

$$q_g = N_f / g_u \qquad \text{Eq. (3)}$$

Where,
N_f = count of vehicles departing from an unqueued condition,
g_u = remaining green time after the last queued vehicle departs, or g-g_q.

The arrival rate during red (q_r) is estimated using the following equation:

$$q_r = \max\left(\frac{N - Q_i - q_g * g}{r}, 0\right) \qquad \text{Eq. (4)}$$

Where,
N = total vehicle departing in a cycle, and g and r are the green and red time for that cycle, respectively,
Q_i= initial queue or the residuals from the preceding cycle. Note that Q_i = 0 for the first cycle in the analysis period.

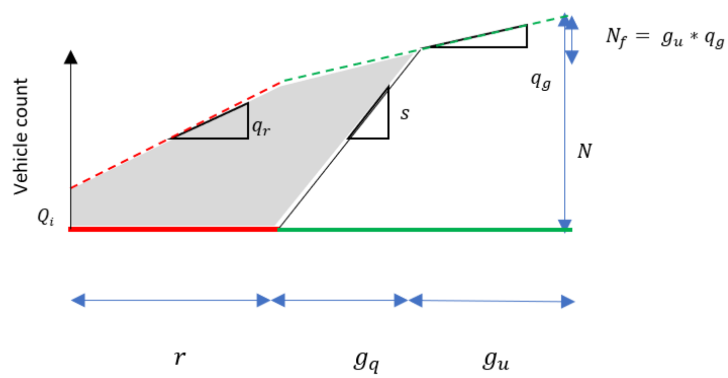


FIGURE 6 Estimation of arrival rates during red and green for a lane within a cycle

Step 3: Performance Measure Estimation: The shaded area in Figure 6 represents the total delay within that cycle, which is estimated using the following equation

$$Delay\ per\ cycle, D = \left(\frac{1}{2}\right) * r * (r * q_r + Q_i) + \left(\frac{1}{2}\right) * q_r * r * g_q \qquad \text{Eq. (5)}$$

Delay per vehicle and percentage arrivals on red can be estimated as:

$$Delay\ per\ vehicle, d = \frac{D}{N} \qquad \text{Eq. (6)}$$

$$AR\% = (q_r * r)/N * 100 \qquad \text{Eq. (7)}$$

The above performance measures are calculated for each cycle and each lane of a movement, then averaged over multiple lanes, and weighted by the estimated arrival volumes served in each lane.

Special Cases
The method described above – specifically the ability to estimate variable arrival rates in the red and green phases, will not work if the cycle is oversaturated or if there are no un-queued vehicles.

However, with some additional, rational assumptions, the analyst can provide sensible estimates of delays and overflow queues for such cases as described below.

Oversaturated Cycle: Figure 7 below shows the arrival-departure plot for successive oversaturated cycles followed by an under-saturated one. The arrival rate (q) in this case is assumed to be uniform throughout all such cycles and estimated as the slope of the dashed line.

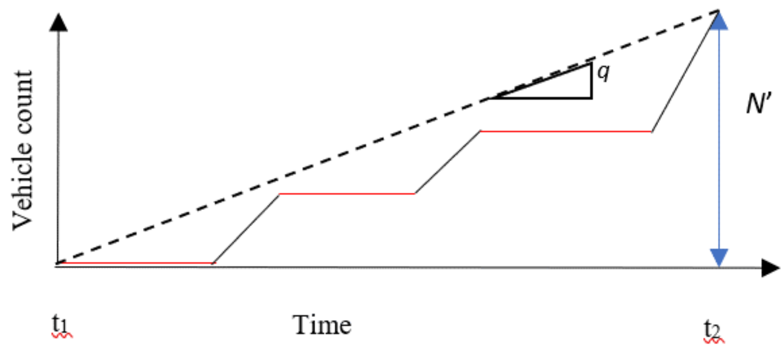


FIGURE 7 Estimation of average arrival rate for successive oversaturated cycles

$$Average\ arrival\ flow\ rate, q = N'/(t_2 - t_1) \qquad \text{Eq. (8)}$$

Where,
N' is the total vehicle departing between the start of the first oversaturated cycle and the end of the first under-saturated cycle that follows. The times t_1 and t_2 mark these start and end times.

For each oversaturated cycle, the overflow queue size, Q_o is estimated as:

$$Q_o = Q_i + qC - sG \qquad \text{Eq. (9)}$$

Where,
Q_i is the size of the initial queue at the start of the cycle, or red time. The delay for each cycle is then estimated as:

$$D = 0.5 * q * r * C + 0.5 * (Q_i * r + Q_o * g) \qquad \text{Eq. (10)}$$

Undersaturated Cycle with No Unqueued Vehicles: Figure 8 shows the arrival-departure plot for an undersaturated cycle where no unqueued vehicles departed after the queue was cleared.

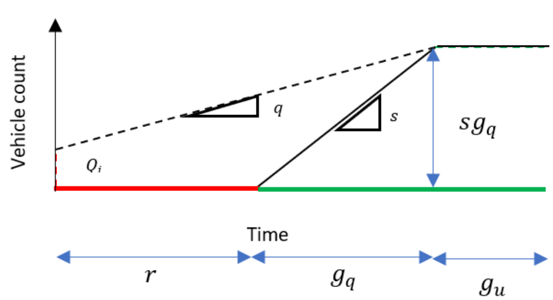


FIGURE 8 Arrival rate estimation for a lane within a cycle with no unqueued vehicle

Here, we assume that the arrival rate is uniform from the start of the cycle to the end of g_q. The average arrival flow rate is estimated as:

$$q = (s * g_q - Q_i) / (r + g_q) \quad \text{Eq. (11)}$$

Delay per cycle is estimated using the following equation.

$$Total\ delay, D = 0.5 * (Q_i * r + q * r * (r + g_q)) \quad \text{Eq. (12)}$$

Note that the formula for delay per vehicle and percentage arrival on red remain same for these special cases. In all cases, the per vehicle delay is reported per *departing vehicle* in the cycle.

Limitations

1. Platoon of vehicles arriving right after the queue clearance may cause erroneous classification of queued and un-queued vehicles
2. In the case of closely spaced intersections, queues from a downstream intersection may affect the identification of queued and un-queued vehicles, causing excessive delay for the first vehicle (queue blockage) and an increase in the departure headways for subsequent vehicles.
3. The method does not account for lane changes induced by lane imbalance at an intersection

Method 3: Shockwave Approach

Shockwave analysis dates back to original theories and observations by Lighthill and Whitham (1955). It utilizes a basic flow density relationship to track flow state boundaries for various flow conditions. This work has been extended for traffic signals to include additional conditions for deceleration and acceleration which began with Michalpopoulos et al (1981) and has seen methodological improvements over the decades. Such methods rely on accurate traffic state measurements which are difficult to obtain under traditional traffic signal detection layouts. Intersections with inductive loop or other point sensors must be instrumented upstream and downstream of the stop bar in positions which can capture arriving and departing traffic. However these systems are typically not used to track individual vehicles and lose fidelity when lane by lane data are aggregated for an approach.

The application of this method for improved delay estimation for ATSPM adopts a set of assumptions in order to develop cycle-by-cycle average arrival and departure flow states including uniform arrivals and equally distributed traffic across lanes. Additionally, the method assumes a triangular flow-density relationship as shown in Figure 9 and does not consider acceleration or deceleration or initial queue delays. Shockwave speeds are estimated by measuring the slope of lines connecting two transition points on the flow-density plot. Key parameters to calibrate are the density and flow at capacity and the jam density. Then, ATSPM data can be used to measure the traffic states of interest and track the red phase for the signal. It is important to note that this method assumes that there are no split failures as no information is tracked across multiple cycles.

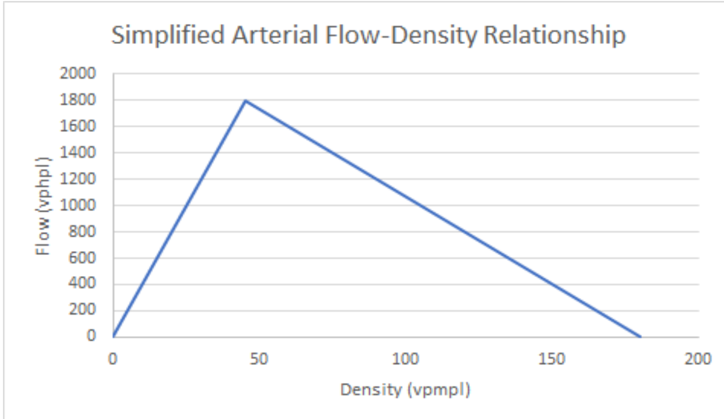


FIGURE 9 Simplified Arterial Flow-Density Relationship

Figure 10 shows the expected flow states and shockwaves. Shockwave (a) is the transition from the free-flowing arrivals (I) to the queued state (II). Shockwave (b) is the transition from queue (II) to departures (III). Shockwave (c) is the transition between departures at saturation flow and the arrivals which continue through the intersection without delay. Shockwaves (a) and (b) always begin at the start and end of the red phase respectively, while shockwave (c) continues from the point where (a) and (b) meet downstream at the free flow speed. In this example graphic, the following data were used:

- Free Flow Speed: 40 mph
- Arrival Flow Rate: 800 vphpl
- Saturation Flow Rate: 1800 vphpl
- Jam Density: 180 vpmpl
- Green Phase: 30 s
- Cycle length: 60 s

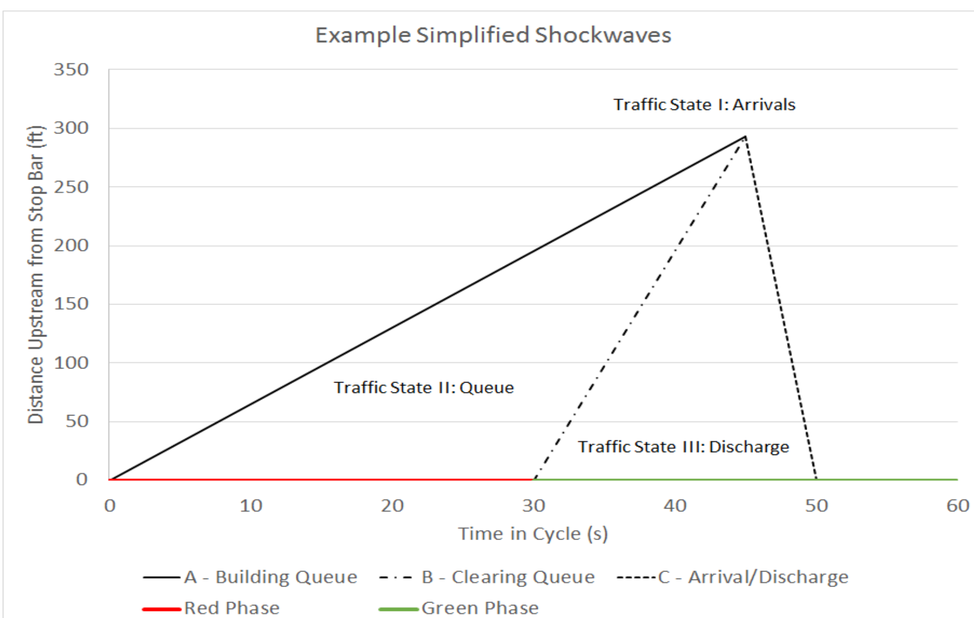


FIGURE 10 Example Simplified Shockwaves

Once all traffic states and shockwaves are estimated, average delay per vehicle can be estimated under the assumption of uniform arrivals. However with ATSPM actuations from actual arrivals are available, which can be used to estimate trajectories and therefore estimate delay. In this case, there is detection at a distance upstream where the advanced detector is located. That detection can be projected downstream at the free flow speed until it either crosses shockwave (a) or passes the stop bar un-delayed. Trajectories which cross shockwave (a) experience stop delay equal to the horizontal time between (a) and (b) at this distance. An example is shown in Figure 11 where a vehicle is detected 350 ft upstream at t=0 in the cycle. This results in a stop at 35 ft upstream and 26.4 seconds of stop delay. This method is repeated for each actuation and can be aggregated to per cycle or time period delay by direct averaging of all individual vehicle delays.

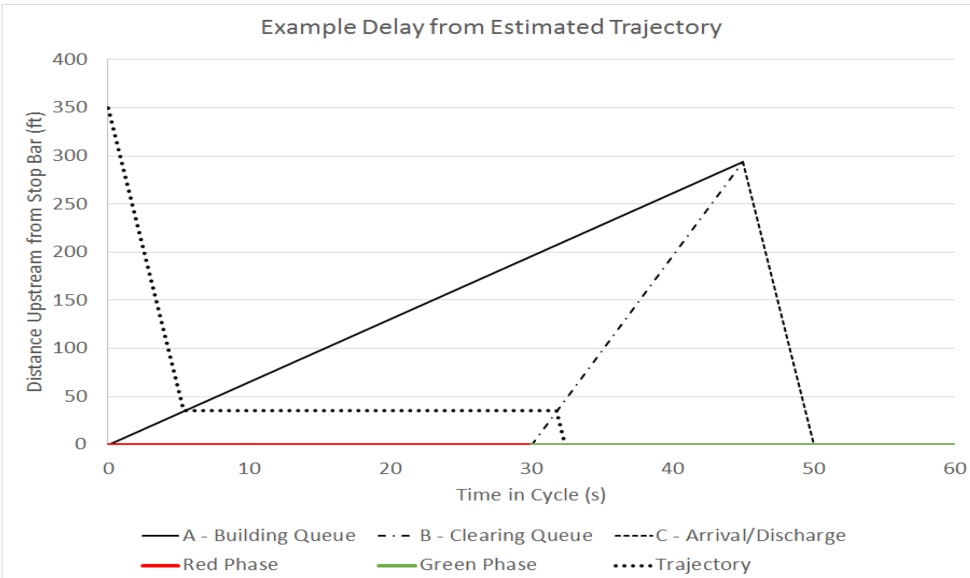


FIGURE 11 Example Delay from Estimated Trajectory

Method 4: ATSPM's Approach

ATSPM provides a simplified approach delay experienced by vehicles approaching and entering the intersection. Both, average delay per vehicle and total delay are calculated as part of this process. Individual delay per each actuation of the upstream detector is calculated and the reported approach delay is aggregated in bins of 5 minutes or 15 minutes. This method uses traffic signal timing data along with upstream detector actuation to obtain the approach delay.

The ATSPM method calculates the approach delay as the time between a detector activation during the red portion of the phase, and when that phase turns green. Figure 12 shows the delay for one arrival during red. The total approach delay for a typical cycle can be obtained as follows:

$$Delay\ per\ cycle\ =\ \sum_{i}^{n}(T_{\mathrm{G}}\ - T_{FF_i} - T_{arr_i}) \qquad \text{Eq. (13)}$$

Where T_G is the start of green for the cycle, T_{FF_i} is the free flow travel time for vehicle i estimated using equation 1, and T_{arr_i} is the time when vehicle i arrives at the upstream detector.

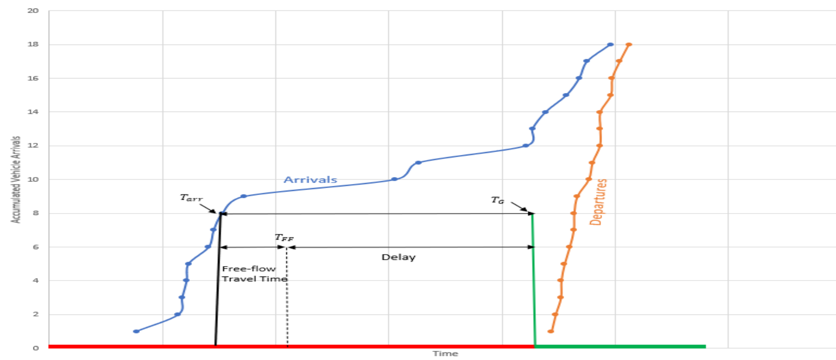


FIGURE 12 ATSPM's estimation of approach delay for a typical vehicle arriving in red

The approach delay calculated by this method fails to account for start-up delay, acceleration delay, deceleration delay and queue length exceeding the detection zone.

CASE STUDY

In order to adequately test the four delay methods in this report, the team identified three critical data elements needed at a candidate approach for conducting a case study. First, the signal timing data must be available in the ATSPM dataset, which is true for all of the sites provided. Secondly, the approach must have upstream detection in order to count arrivals. Finally, a downstream detector is needed for methods 1 and 3 in order to ensure arrival and departures in a cycle are balanced. All Signal IDs were entered in the UDOT ATSPM website in order to determine which performance measures and associated detector data were available. The team mandated that the following reports should be available for the analysis: 1) Purdue Split Failure, 2) Turning Movement Counts, 3) Approach Delay, 4)Approach Speed. Table 1 below documents the data availability.

TABLE 1 Data Availability

Signal ID	Purdue Split Failure	Turning Movt. Counts	Approach Delay	Approach Speed
7122	Y	Y		
7123	Y		Y	Y
7124	Y		Y	Y
7342				
7125	Y	Y		
7126	Y			
7127	Y			
7128				
7129	Y			
7241	Y	Y		
7180	Y			
7181	Y	Y	Y	Y
7182	Y	Y	Y	
7183	Y	Y	Y	
7184	Y	Y	Y	
7185	Y	Y	Y	Y*
7186	Y	Y	Y	Y*
7187	Y	Y	Y	
7076			Y	Y
7188	Y		Y	Y
7189	Y		Y	Y
7190	Y	Y		

**Not available for all major street approaches*

The team selected Signal ID 7181 based on data availability, and established the event enumerations of interest in the raw data file for the Southbound approach and turning movement count detectors as well as the corresponding Phase 6 signal data. During data exploration, the team discovered that the turning movement count detectors were all active beginning in October 2018, so only data beyond this point was reviewed. In order to reduce data cleaning activities, a single day on Tuesday November 6, 2018 was selected for use in all four methods. In addition to the selection of events of interest in the log, an additional field was added for advanced processing of data use for each method to establish a common cycle ID based on the start of the red for Phase 6 in each cycle.

Figure 13 shows the layout of detectors for the selected signal. Detector 4 is an advanced count detector located 390ft from the stop bar and used for keeping track of arrivals. Detectors 43, 42, and 41 are lane-by-lane stop bar count detectors used for keeping track of departures from the intersection. Methods 1 (arrival/departure curve) and 3 (shockwave) use actuations of detectors 4, 41, 42, and 43 for calculation of approach delay, while method 2 (departure-only) uses only detectors 41, 42, and 43 for this purpose.

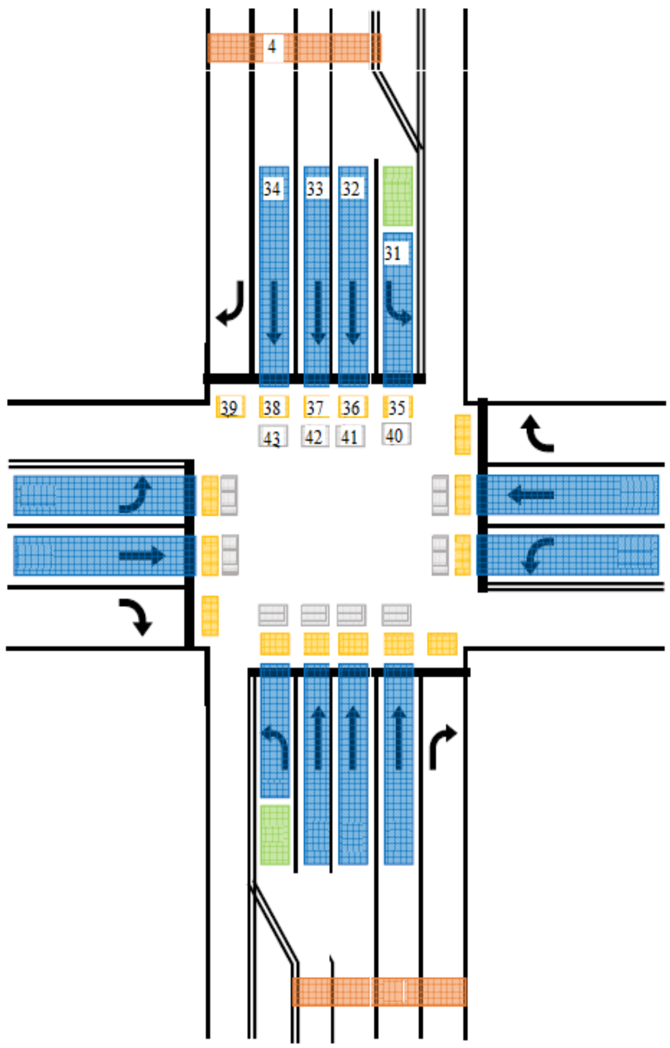


FIGURE 13 Detection setup for signal ID 7181

ANALYSIS AND RESULTS

This section provides the analysis and results pertaining to all four methods for estimating the delay on a cycle-by-cycle, 5-minutes and, 15-minutes aggregated basis for November 6, 2018. Figure 14 shows the results of average approach delay per vehicle by all four methods, aggregated at a cycle-by-cycle level. A two hour period is highlighted in Figure 15 to allow for direct comparison during the peak period. Comparison of results show that the delay estimated by Method 1 is always greater than that estimated by ATSPM. This is expected because Method 1, in addition to incorporating stop delay, includes deceleration delay and a portion of the full acceleration delay profile of vehicles. Method 2 shows high variability with higher maximum cycle average delays due to the method including different green and red arrival rates. Methods 3 and 4 using both raw or adjusted arrivals show similar trends for each cycle. This is due to the assumptions being very similar in the two methods. Aggregation to five minutes (Figure 16) or 15 minutes (Figure 17) show that these trends also occur in aggregate with two unique patterns emerging. First, Method 1 has the highest maximum delay values which are overestimated under very low flow conditions. This is most likely due to long cycles where "rebalanced" arrivals create very long travel times before the observed departure happens. Secondly, Method 3 seems to be the only method which shows a delay trend that tracks with the overall approach volume. This southbound approach has a higher PM peak than AM peak volume with every peak hour cycle showing split failure for the Southbound Phase 6 however the other methods show fairly flat delay trends over time. This delay pattern is promising as one major limitation to the current Method 4 ATSPM approach delay tends to not track the overall congestion time of day trend in many other locations the authors have studied. Finally, Method 2 seems to track well with higher delay expectations during the peak periods (7-9 am and 4-6 pm) in Figures 16 and 17.

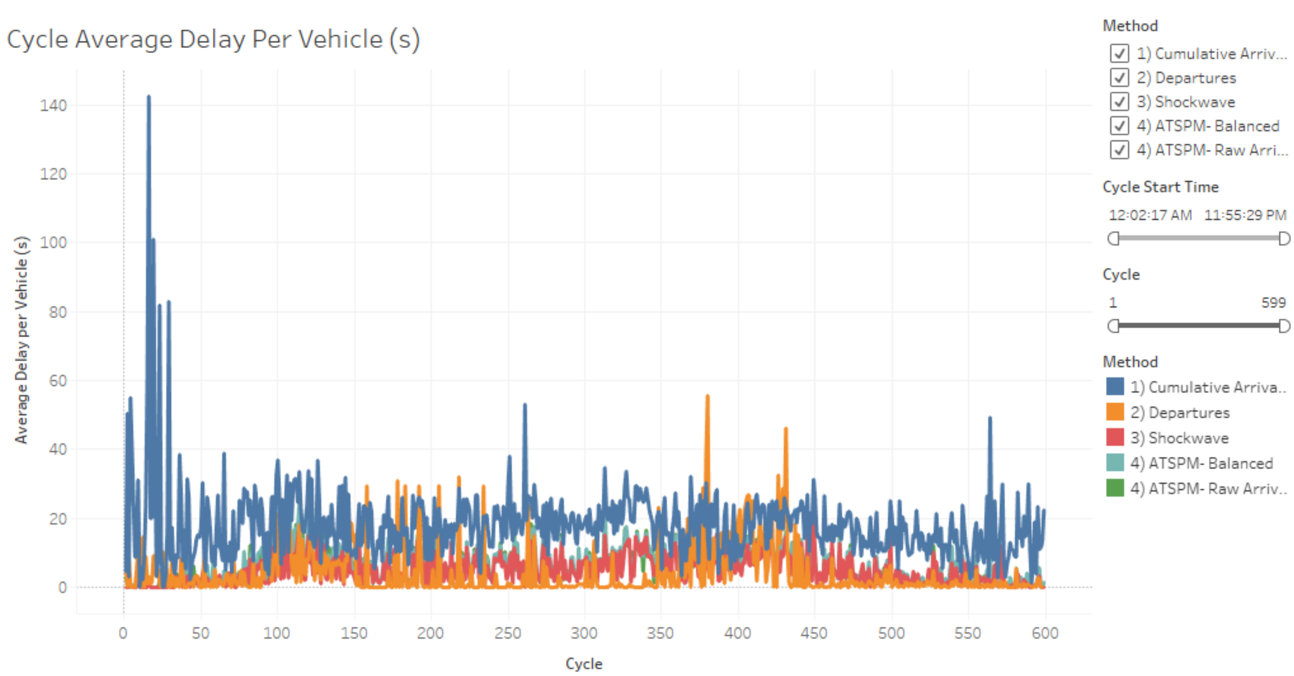


FIGURE 14 Cycle Average Delay per Vehicle

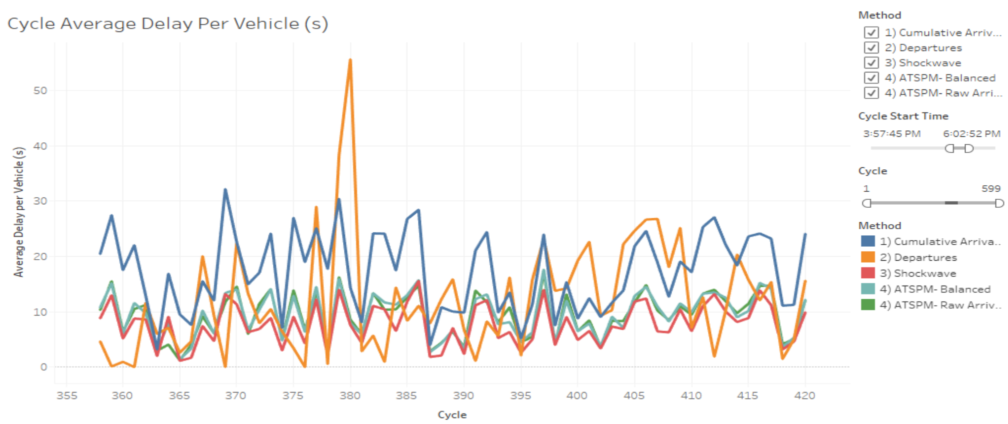


FIGURE 15 Cycle Average Delay per Vehicle - 4PM to 6PM

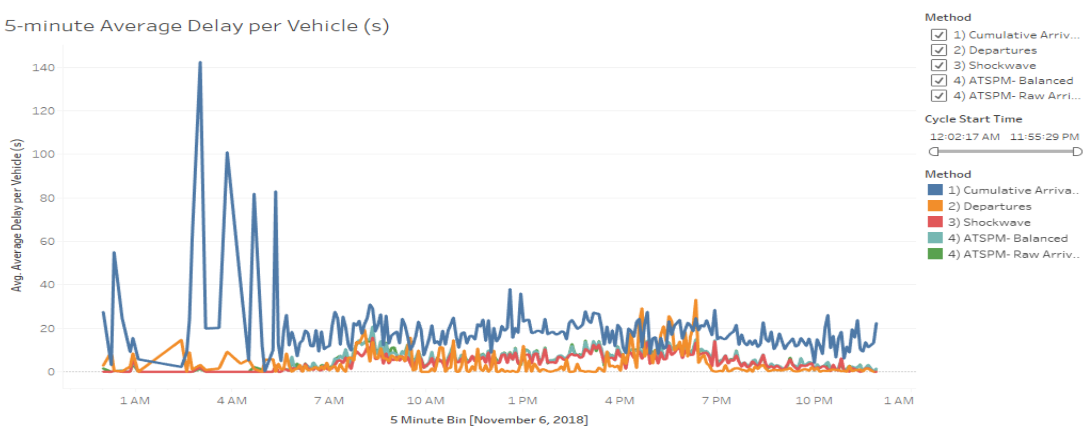


FIGURE 16 5-minute Average Delay per Vehicle

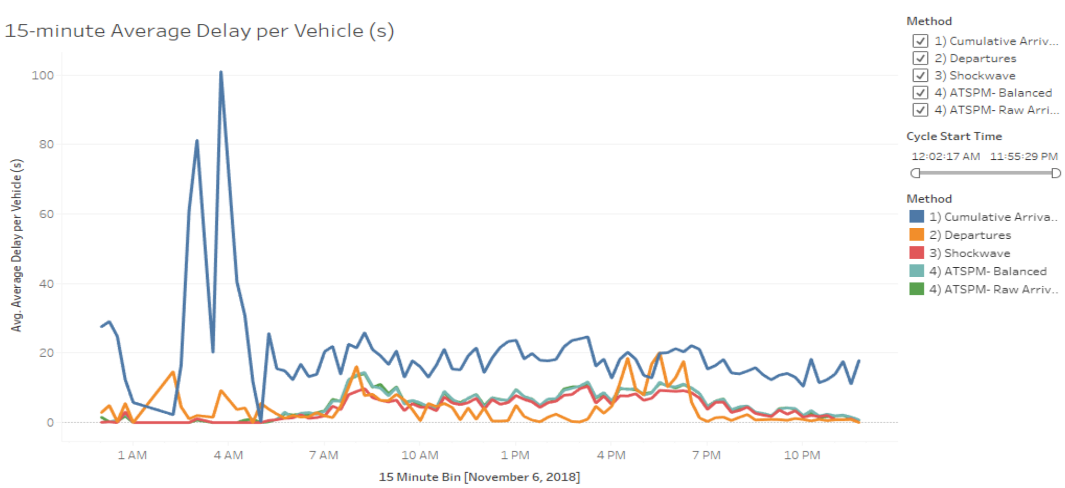


FIGURE 17 15-minute Average Delay per Vehicle

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

More than 330,000 traffic signals are being used for traffic management in the United States and 75% + of these could be significantly improved with updated equipment or timing plans (FHWA-HRT-05-002). Unnecessary delay to motorists and pedestrians is a significant side effect of these control devices. A major reason for unnecessary delays is poor traffic signal timings responsible for nearly 300 million vehicle-hours on major roadways alone (Schrank et al. (2015)). ATSPM is a recent development aimed at improved management of traffic signals by providing performance metrics based on high resolution data. These performance metrics improve decision making and selection of more effective operational strategies to reduce delay and cost.

Although ATSPM provides a measure for assessing delay experienced at an approach, it is a highly simplified method that ignores some fundamental components of delay. This report proposes three alternative methods for estimating approach delay at an intersection, each of which addresses one or more drawbacks currently present in the ATSPM framework.

The first alternative method, the "cumulative arrival/departure method" uses arrival, departure, and traffic signal timing data to estimate the approach delay. Out of the four components of delay, this method fully incorporates the stop and deceleration delays, but only partially includes acceleration delay (only that incurred prior to the downstream detector). However, the delay estimated by this method does not include any initial queue delay from previous cycles.

The second method, the "departure-only method" uses departure actuations and the traffic signal timing data to estimate the approach delay. This method fully incorporates stop component of the delay. Furthermore, it includes any initial queue delays – estimated with the assumption of uniform arrival rate that spans through all consecutive cycles with split failure. However, acceleration and deceleration components of the approach delay are not considered in this method.

The third method, "shockwave method", uses arrival, departure, and traffic signal timing data to estimate the approach delay. The stop component of the approach delay is fully incorporated in the estimates by this method. This method, however, does not include the initial queue, acceleration and deceleration components of the approach delay.

Table 2 lists the assumptions and/or limitations for each methodology. Method 1 assumes, in addition a first-in-first-out (FIFO) principle for vehicles that arrive at the upstream detector and depart from the downstream detector. Table 3 provides a graphical depiction of delay components covered by each methodology.

TABLE 2 Model Limitations or Assumptions

Limitation or Assumption	Method 1	Method 2	Method 3	Method 4
Requires Arrivals	Yes	No	Yes	Yes
Requires Departures	Yes	Yes	Yes	No
Adjust Arrivals	Yes	No	Yes	No
Assumes Fixed Speed	Yes	Yes	Yes	Yes
Assumes Under-saturated Cycles	Yes	No	Yes	Yes
Assumes Uniform Arrival Rate	No	Yes	No	No
Processing time (sec)	14.5	20	15	-

*Processing time for data preparation and cycle division: 225 seconds.

TABLE 3 Aspects of Control Delay Included in Estimate

Method	Stop	Deceleration	Acceleration	Initial
1:Cumulative Arrival/Departure			i	
2: Departure-Only				ii
3: Shockwave				
4: Current ATSPM	iii			

i) Acceleration delays incurred prior to the downstream detector are included
ii) Initial queues are estimated using uniform arrival rate which spans consecutive cycles with split failure
iii) Stopped delay is minimally estimated assuming a "vertical queue"

Overall, all four methods showed unique trends for certain times of day and traffic conditions. Table 5 shows the distribution of cycle average delay per vehicle estimated across the study day. Methods 3 and 4 followed very similar trends, however the unique time of day pattern seen in Method 2 is especially promising for future study.

TABLE 4 Cycle Average Delay per Vehicle Descriptive Statistics

Method	Minimum	Average	Maximum	Standard Deviation
1. Cumulative Arrival/Departure	0	18.06	142.55	10.81
2. Departures Only	0	4.08	55.66	7.14
3. Shockwave method	0	5.02	19.20	4.26
4. ATSPM- Balanced	0	6.25	24.70	5.05
4. ATSPM- Raw Arrivals	0	6.19	26.04	5.02

Each method includes some critical assumptions or limitations, many of which may be addressed through future research. The authors note that Method 1 should always produce higher delays than the base ATSPM Method 4 due to the inclusion of all deceleration and some acceleration delay. However this method relies heavily on consistent actuations upstream and downstream which cannot be achieved when the upstream detector includes additional turning movements. Thus, future work on Method 1 should include more defensible and realistic addition or subtraction of upstream actuations as well as consider split failure conditions where departures are lower than arrivals for one or more cycles.

Method 2 is the only method which includes initial queue delay, though the current method assumes a constant arrival rate for consecutive cycles with split failure. This provides a minimal estimate of initial queue delay that may be better calibrated using upstream arrival distributions. This is also the only method which requires solely downstream detection. Other candidate improvements identified include detection of platooned vs discharging flows, handling downstream spillback, and initial headway thresholds during low flow conditions.

Finally, Method 3 has the largest combination of data needs and assumptions resulting in many limitations to the methodology. Full additions of upstream arrival rates and more realistic discharge rates will improve the delay estimate, however this may lead to the method becoming a generalized case of Method 2. Due to the ease of estimation, this method may be best left as a comparison method for future work on one of the other two proposed methods.

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