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REAL-TIME ESTIMATION OF TRAVEL TIMES ON SIGNALIZED ARTERIALS

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ABSTRACT

Travel time is an important measure for assessing the operating efficiency of signalized arterials, evaluating the performance of traffic management strategies and developing real-time traveler information systems. An analytical model is proposed to estimate the travel times on arterial streets in real-time based on data commonly provided by system loop detectors (flow and occupancy) and the signal settings (cycle length, green times and offsets) at each traffic signal. The model is based on kinematic wave theory to model the spatial and temporal queuing at the traffic signals and explicitly considers the signal coordination in estimating traffic arrivals at the intersection. The application of the proposed model on two arterial sites and comparisons of the estimated travel times with simulated and field data show that the model accurately predicts travel times at the selected sites.

INTRODUCTION

Continuous traffic growth through developed areas and difficulties in building new infrastructure have caused a need for careful monitoring of operating conditions on existing transportation facilities and development of improved strategies to manage the increase in traffic demand. The analysis of existing conditions and evaluation of alternative improvements requires estimates of performance measures for both system operators and users. Travel time is an important measure to assess the existing operating conditions along signalized arterials, evaluate control and management strategies and provide information to travelers.

Conventional approaches for estimating travel times on arterial links include speed vs. volume/capacity ratio relationships, mostly used in four-step planning models (BPR, 1964, Akcelik, 1991) or procedures based on the Highway Capacity Manual (HCM) (Transportation Research Board, 2000). The HCM procedure calculates average travel time as the sum of the running time and the intersection delay. The running time is based on the time required to traverse the arterial link based on the arterial design characteristics and the signal density. The intersection delay is calculated based on a deterministic point delay model. Such approaches are not suited for real-time applications especially under congested conditions.

Existing real-time performance measurements are typically restricted to the estimation of local intersection-based measures for signal timing parameter tuning or qualitative measures of performance to provide collective traffic information, e.g., through changeable message sign (CMS) displays (Perrin et al., 2002). However, this qualitative information cannot provide to drivers with an accurate representation of the changes in actual traffic conditions, including dynamic travel time variations along urban arterial routes.

Several models have been proposed for estimating travel times from surveillance data. One of the simplest methods estimates the average speed from the loop detector flow and occupancy data, assuming an average vehicle length. These speed estimates however are inaccurate for most traffic environments because they are obtained at the detector location (usually in the middle of the link) and ignore the delay at the intersection, except for short congested links where the detector occupancies reflect the true operating conditions. Other similar approaches use double loop detectors or procedures to estimate the average vehicle length, similar to those techniques used in travel time estimation on freeways (Oda, 1990). Other approaches include statistical models based on regression analysis to estimate link travel times as a function of the site characteristics (spacing, free-flow speeds, saturation flows) and loop detector data. Both linear (Turner et al., 1996), non-linear (Zhang, 1999) and Bayesian (Frechette and Khan, 1998) model formulations have been proposed. Most of these models however, are quite site specific and cannot be readily applied to other environments.

Recently, advancements in sensor technology have produced sensors that can provide the magnetic vehicle signatures to re-identify individual vehicles (or vehicle platoons) at a downstream location for travel time estimation (Sun et al, 1998, Lucas et al, 2004). However, the effectiveness of such systems has not been thoroughly tested and their costs are much higher as compared to conventional loop detectors, which make the practical implementation prohibitively expensive especially on large signal systems. A number of approaches employ neural network, fuzzy set theory to develop arterial travel time prediction models that involve the use and fusion of multiple data sources--loop detectors, probe vehicles, license plates (Zhu and Wang, 2002, Cheu et al., 2001, Takaba, 1991). Most of such models have been developed and tested under simulated traffic conditions.

The objective of this research is to develop and test a model for real-time estimation of travel times along signalized arterials based on surveillance data (flows, occupancies) from conventional loop detectors and signal settings (cycle length, green times and offsets) from the signal control system. The research is part of a study to develop and implement a performance measurement system for arterial networks (Skabardonis et al, 2004).

The next section of the paper describes the formulation of the proposed model. The estimation of the model parameters and the implementation of the model is described in section 3. The application of the model to two arterial networks is presented in Section 4. The last section summarizes the study findings, and outlines ongoing and future work.

PROPOSED MODEL

The proposed model estimates the travel time on an arterial link as the sum of the free flow time and the delay at the traffic signal. The model formulation is designed for arterial systems with detectors placed sufficiently upstream from the intersection stopline (system detectors), so the flow and occupancy measurements are not affected by the presence of queues at the traffic signal.

The free flow time is the time a vehicle needs to travel the length of the link without interference from the presence of the signal. This is the time traveling at or close to the free-flow speed and can be readily measured from the loop detectors under low flow conditions.

The delay at the traffic signal is calculated as the sum of a) the delay of a single vehicle approaching the traffic signal b) the delay because of the queues formed at the intersection, and c) the oversaturation delay, the additional delay caused when the arrival rate is greater than the service rate at the signal. Figure 1 shows the structure of the proposed model.

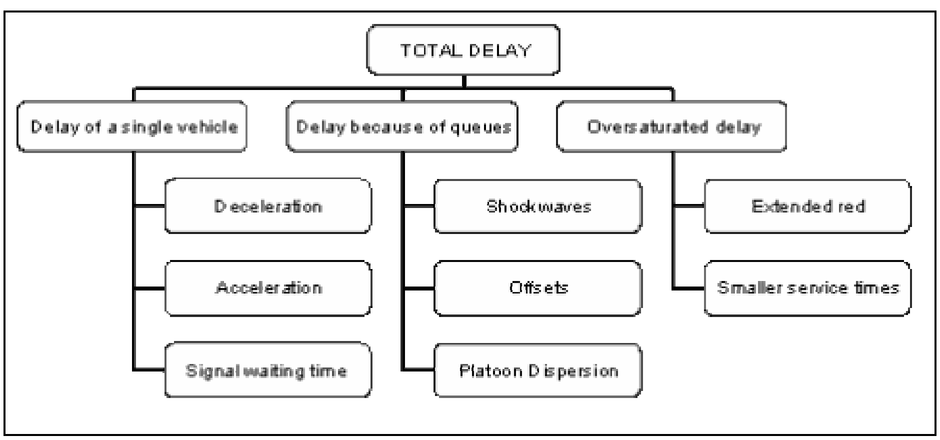

Figure 1. Proposed model for delay estimation

Delay of a single vehicle at a traffic signal

The delay of a single vehicle d_s at a traffic signal, assuming no interactions with other vehicles is defined as the additional time over "free flow speed time" and is the sum of two delays: a) the deceleration plus idle delay d_{s1}, which is the additional time required to decelerate and stop at the stopline plus the waiting time at the stopline, and b) the acceleration delay d_{s2}, which is the additional time required to reach the free flow speed when the signal turns green. Figure 2 shows typical trajectories of vehicles approaching a traffic signal, and illustrates the definition of the delay d_s and its components d_{s1} and d_{s2} for vehicle B.

The signal settings at the intersection are known, where g is the effective green time and r is the effective red time. We introduce the following two decision points (Figure 2):

1. *1st decision point*: it is located at distance L_0 upstream from the intersection stopline where the driver decides whether to decelerate with normal deceleration rate γ_d because the signal is red or to continue traveling at the same speed. If T is the driver's reaction time and u_f is the free flow speed, then the distance L_0 is:

$$L_0 = T \cdot u_f + \frac{u_f^2}{2\gamma_d} \tag{1}$$

2. *2nd decision point*: it is located at distance L_{0m} upstream from the stopline where the driver decides whether to decelerate with maximum (emergency) deceleration rate γ_{dm} because the signal is red or to continue traveling at the same speed. L_{0m} is also given by the Equation (1) by using the value of γ_{dm} instead of γ_d.

The first delayed vehicle is vehicle A in Figure 2, which reaches the 2nd decision point at t=0 (start of red time). This vehicle arrives at the 1st decision point at time t_0 before the start of red time:

$$t_0 = \frac{L_0 - L_{0m}}{u_f} = \frac{u_f}{2} \cdot \left(\frac{1}{\gamma_d} - \frac{1}{\gamma_{dm}} \right) \tag{2}$$

Consider a vehicle which reaches the 1st decision point when the signal turns red (t=0). The delay of this vehicle is the sum of the two predefined types of delays for $t=0$:

$$d_{s1}(0) = r - T - \frac{u_f^2 / 2\gamma_d}{u_f} = r - T - \frac{u_f}{2\gamma_d} \tag{3}$$

$$d_{s2}(0) = \frac{u_f}{2\gamma_\alpha} \tag{4}$$

Note that the acceleration delay d_{s2} is the same for all vehicles that stop at the intersection because each vehicle accelerates from $u=0$ to $u=u_f$ with normal acceleration rate γ_α.

Vehicle C reaches the stopline when the signal turns green. This is last vehicle to stop at the signal (Figure 2). It arrives at the 1st decision point at time t_c before the start of green:

$$t_c = T + \frac{u_f}{\gamma_d} \tag{5}$$

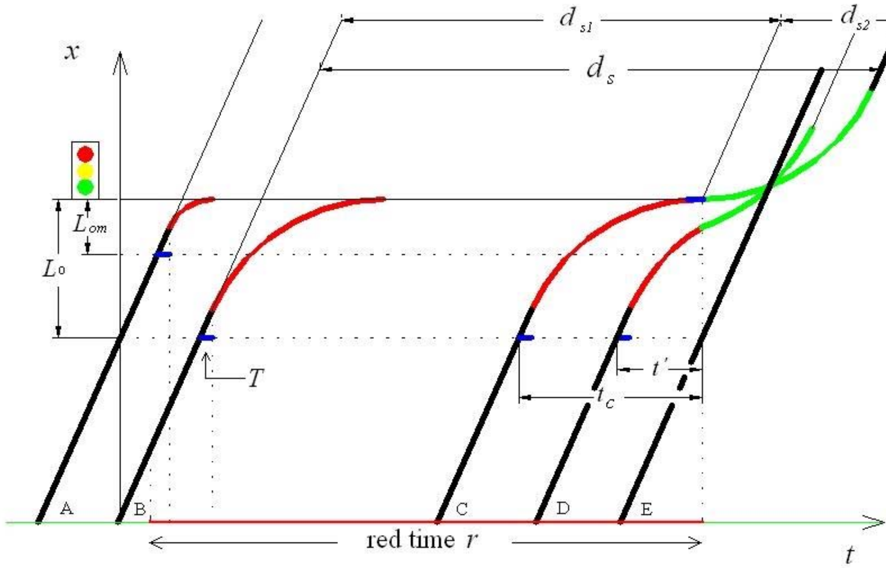

Figure 2. Trajectories of vehicles approaching a traffic signal

For the time interval $-t_0 < t < r-t_c$ where t is the time after the signal turns red that a vehicle arrives at the 1st decision point, the delay d_{s1} is a linear function of time t, and the delay d_{s2} is constant:

$$d_{s1}(t) = d_{s1}(0) - t \quad for \quad -t_0 < t < r - t_c \tag{6}$$

$$d_{s2}(t) = d_{s2}(0) \quad for \quad -t_0 < t < r - t_c \tag{7}$$

If a vehicle arrives at the first decision point at time $t>r-t_c$ (vehicle D in Figure 2) it decelerates but the signal turns green before it stops so it accelerates from a speed $u \neq 0$. Let t′ be the remaining time from the beginning of green when the vehicle reaches the 1st decision point $(t' = r - t)$. The vehicle decelerates from u_f to u in t'-T time units by traveling with deceleration rate γ_d. Then, it accelerates from u to u_f by traveling distance s with acceleration rate γ_α. After some manipulations to estimate the distances traveled in the deceleration and acceleration modes, the delays d_{s1} and d_{s2} are given by the formulae:

$$d_{s1}(t') = \frac{s}{u_f} - (t' - T) \Rightarrow d_{s1}(t') = 0.5\gamma_d \frac{(t'-T)^2}{u_f} \tag{8}$$

$$d_{s2}(t') = \frac{((t'-T)\gamma_d)^2}{2u_f\gamma_a} \Rightarrow d_{s2}(t') = \frac{\gamma_d}{\gamma_a} d_1(t') \tag{9}$$

The delay d_s over the signal cycle as a function of arrival time t at the 1st decision point is given by Equation (10):

$$d_s(t) = \begin{cases} -t + \left(r - T - \dfrac{u_f}{2\gamma_d} + \dfrac{u_f}{2\gamma_\alpha} \right) & , \ -t_0 < t < r - t_c \\ 0.5\left(\dfrac{\gamma_d \cdot \gamma_\alpha + \gamma_d^2}{\gamma_\alpha} \right) \dfrac{(r-t-T)^2}{u_f} & , \ r - t_c < t < r \\ 0 & , \ otherwise \end{cases} \tag{10}$$

Figure 3 shows the delay of a single vehicle as a function of time t, where t_0 is the time required to decelerate and stop from the first decision point. This function is partly linear and partly quadratic, but continuous and differentiable in its domain.

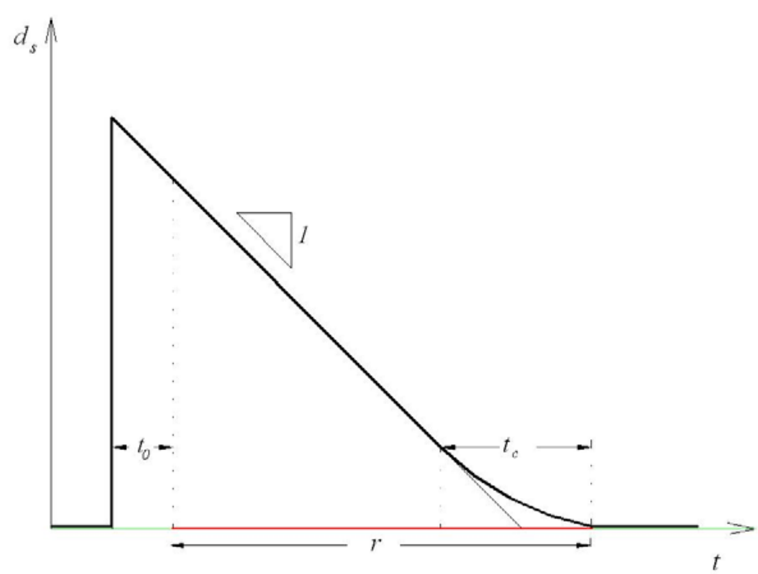

Figure 3. Delay of a single vehicle arriving at 1st decision point

Delay at a single signal because of queue

This is defined as the delay because of the queues present at the traffic signal. In the absence of queues all the vehicles would depart by following the trajectory BC in Figure 4, so the queuing delay is the area of the triangle BCD. As it is shown in Figure 4, the delay increases from 0 to a maximum delay at point C, and then decreases to zero for the vehicle that passes through point D in the *x-t* diagram. The distance L_{qm} is the distance at which the queuing delay is maximum, and the distance L_q is the maximum back of the queue (the farthest point the queue extends throughout the cycle).

We estimate the queuing delay according to the kinematic wave LWR theory (Lighthill and Whitham, 1955 and Richards, 1956), so to explicitly consider the temporal and spatial formation of queues. We assume a piecewise linear flow-density relationship (fundamental diagram) with parameters u_f (free-flow speed), c (capacity), k_j (jam density), and w (congested wave speed) shown in Figure 5.

The queuing delay d_q of the n-th vehicle arriving at the signal from the beginning of the red time is estimated by considering the following three types of delays (Figure 4):

1. The positive delay d_{q1} which is measured from the vertical axis crossing point B for $0 < x < L_q$, and reaches the maximum at point C.
2. The negative delay d_{q2} which is measured from the vertical axis crossing point C for $L_{qm} < x < L_q$
3. The positive delay d_{q3} which is measured from the vertical axis crossing point B for $0 < x < L_q$

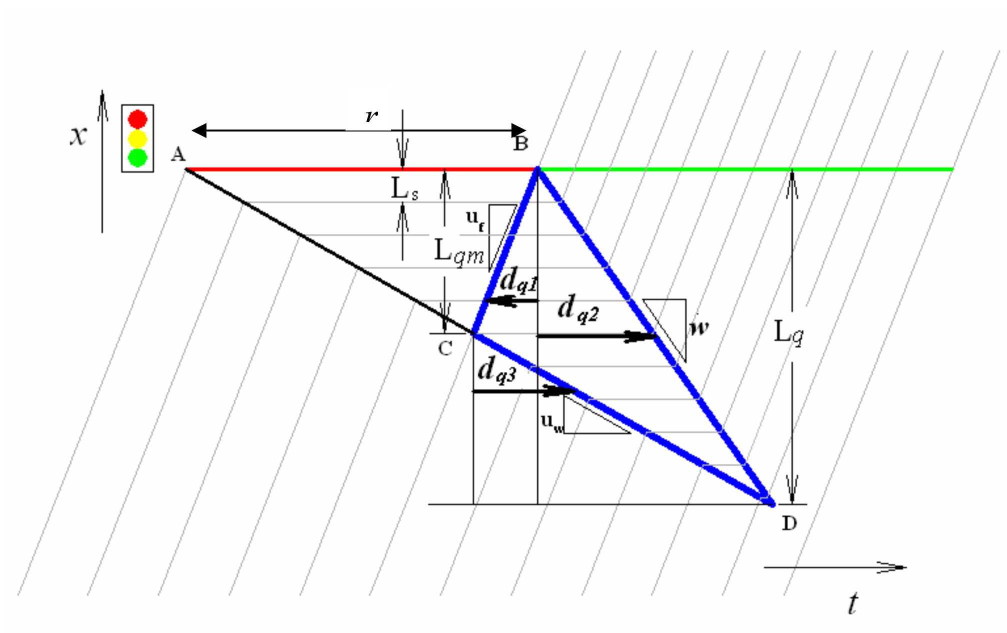

Figure 4. x-t diagram at a single signal

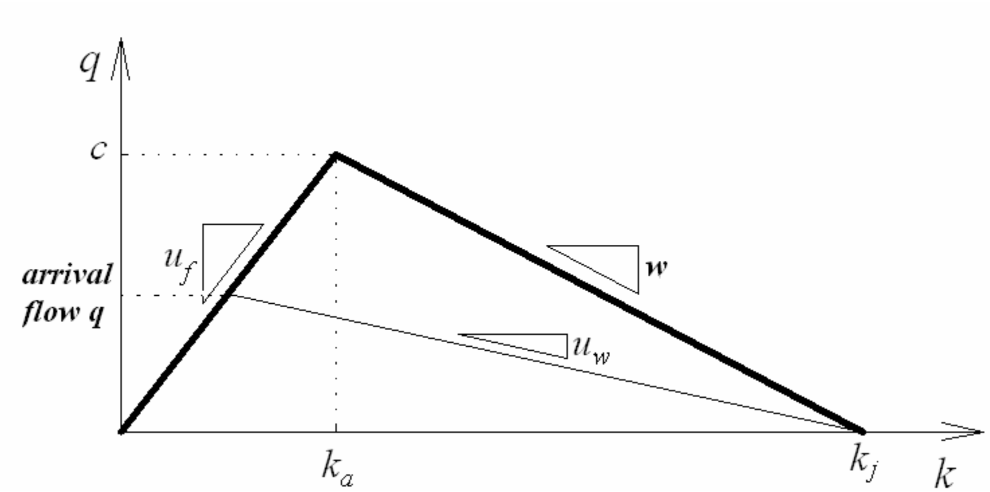

Figure 5. Assumed flow density diagram

From the geometry of triangles ABC and ABD we obtain:

$$r = \frac{L_{qm}}{u_w} + \frac{L_{qm}}{u_f} \quad \Rightarrow L_{qm} = r \cdot \frac{u_f \cdot u_w}{u_f + u_w} \tag{11}$$

$$r = \frac{L_q}{u_w} - \frac{L_{qm}}{w} \quad \Rightarrow L_q = r \cdot \frac{w \cdot u_w}{w - u_w} \tag{12}$$

where u_w is the speed of the shockwave shown in Figure 5.

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The number of stopped vehicles in the distances L_{qm} *and* L_q are:

$$N_{qm} = \left[\frac{L_{qm}}{L_s}\right] \quad \text{and} \quad N_q = \left[\frac{L_q}{L_s}\right] \tag{13}$$

where:

[x] : integer part of number x

L_s : the effective length of a stopped vehicle, i.e., the reciprocal of the jam density k_j

The delay of the n-th vehicle for $n<N_q$ arriving at the signal from the beginning of the red time is the sum of three types of delays given by the formulae:

$$d_{q1} = \left(\min(n, N_{qm}) - 1\right) \cdot \frac{L_S}{u_f} \tag{14}$$

$$d_{q2} = -\left(\min\left(\max(n,\ N_{qm}),\ N_q\right) - N_{qm}\right) \cdot \frac{L_S}{u_w} \tag{15}$$

$$d_{q3} = \left(\min(n, N_q) - 1\right) \cdot \frac{L_S}{w} \tag{16}$$

The total delay of the first k vehicles for $n<N_q$ which arrive at the signal is given by the formula:

$$d_{qk}=L_S\sum_{n=1}^{k}\left(\frac{\min(n,N_{qm})-1}{u_f}-\frac{\min\left(\max(n,\ N_{qm}),\ N_q\right)-N_{qm}}{u_w}+\frac{\min(n,N_q)-1}{w}\right)\tag{17}$$

The effect of signal offsets in the delay estimation

The estimation of intersection delays along an arterial or a network with coordinated signals, must explicitly consider the offsets between adjacent signals. The offsets determine the arrival pattern of vehicle platoons at successive intersections and greatly affect the vehicle delays. Under favorable progression most of the vehicles travel without stops and delays between successive intersections ("green wave"). On the other hand, "bad" offsets cause high delays and may result in spillovers especially for short signal spacing.

The green time that can be utilized by the vehicle platoon at the downstream signal depends on the offset o, the distance L between the adjacent signals and the speed of vehicles. Therefore in order to apply the proposed base model for delay estimation we need to modify the phase durations at the downstream intersection. Assuming that the first vehicle departs from the upstream intersection at the beginning of the green with free flow speed u_f, then we have the following two cases as shown in Figure 6.

1. the first vehicle departing from the upstream intersection arrives during the green time at the downstream signal (Figure 6a):

$$\begin{aligned} g' &= g-\left(L/u_f-o\right) \\ r' &= r \end{aligned}\tag{18}$$

2. the first vehicle departing from the upstream intersection arrives during the red time at the downstream signal (Figure 6b):

$$\begin{aligned} g' &= g \\ r' &= r-\left(o-L/u_f\right)+t_q \end{aligned}\tag{19}$$

where t_q is the portion of the green time that the vehicles have to stop because of the queue presence. This time interval is calculated by considering the speeds of the shockwaves (Figure 4), where N_q is the number of delayed vehicles:

$$t_q=\min\left(\frac{N_q\cdot L_s}{w}+\frac{N_q\cdot L_s}{u_f},g\right)\tag{20}$$

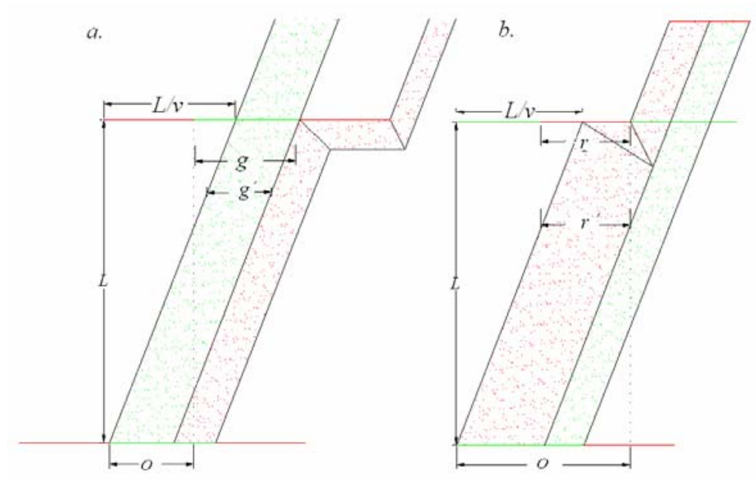

Figure 6. Offsets and platoon arrivals at adjacent intersections

Oversaturation delay

Oversaturation delay is defined as the additional delay caused when the arrival rate exceeds the service rate at the traffic signal. If the queue discharge time t_q calculated in Equation (20) is equal to the available green time g, then a number of arriving vehicles cannot be served in this green phase, but they will be served in the next cycle (Figure 7). So, the oversaturated delay is estimated as follows:

1. The delay of the vehicles that are not served in the cycle they arrive, originally estimated from Equations (10) and (17), must be increased by the amount of the red interval *(r)*.

2. The green time for the vehicles arriving in the next cycle must be reduced by the time interval required to serve the residual queue from the previous cycle. Under heavy congested conditions this time interval may be larger than the green time, which means that in this cycle only vehicles that arrived in the previous cycle(s) will be served, and some vehicles will be delayed by more than "red time" interval.

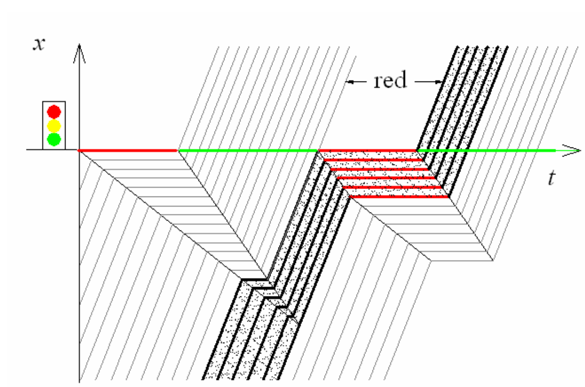

Figure 7. Illustration of oversaturation delay

Platoon dispersion

Traffic departing a traffic signal initially moves as a tight platoon with short vehicle headways. The platoon spreads out the farther downstream it travels because of different vehicle speeds, lane changes, merging or weaving, and other interferences (parking, pedestrians, and other frictional effects). Thus, the arrival pattern at the downstream signal (and the delays) are affected by the platoon dispersion. Our delay estimation model described above, does not consider platoon dispersion because according to the assumed piecewise linear q-k diagram all vehicles depart from the stopline at the free flow speed and all the flows are at capacity.

The platoon dispersion is modeled by applying the kinematic wave theory and assuming a non-linear (concave) *q-k* relationship (Figure 8). The parameters and form of the non-linear part of the diagram is estimated using data from the 2000 Highway Capacity Manual for undersaturated conditions. The first vehicle departs from the intersection stopline at free flow speed, but because of interfaces between different points of *q-k* diagram, the following vehicles cannot depart at free flow speed but at a lower speed. The rate of platoon dispersion depends on the curvature of the increasing part of the *q-k* diagram. We estimate the average platoon dispersion ratio at a distance L_i from the intersection stopline using a numerical approach with finite differences. The mathematical formulation and detailed description of the methodology are described elsewhere (Geroliminis and Skabardonis, 2005). Figure 9 shows the average platoon dispersion as a function of distance.

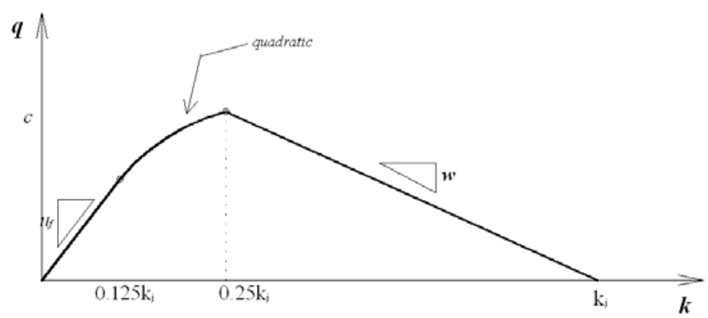

Figure 8. Assumed q-k diagram for modeling platoon dispersion

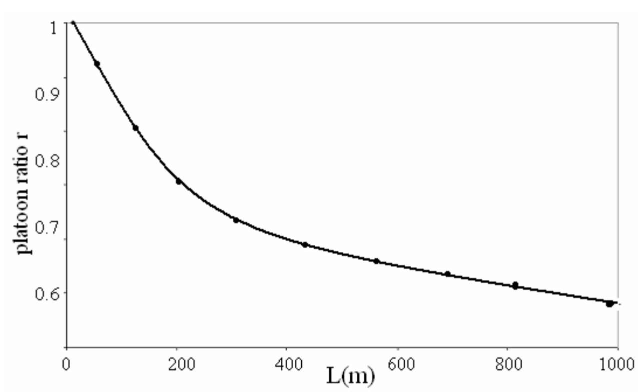

Figure 9. Mean Platoon ratio as a function of distance L from the intersection stopline

MODEL IMPLEMENTATION

Estimation of the model parameters

The default values of the parameters needed for the estimation of delays for a single vehicle (Equation (10)) T, γ_α, γ_d (γ_{dm}) are shown in Table 1. These values were obtained from published sources (ITE, 1999). These values also are in close agreement with measured values obtained from the trajectories of floating cars equipped with GPS system. In general, the values of those parameters vary among drivers and depend on vehicle characteristics and roadway conditions. However, the results from sensitivity analysis indicate that the delay d_s is fairly insensitive for a wide range of values of these parameters (Table 1). The maximum difference in delay estimates is 5 seconds.

Model Parameter	Default Value	Range	Max Difference in Delay (sec)
T (sec)	1.2	0.7 - 2.5	5 sec
γ_d (ft/sec^2)	10	5.0 – 14	
γ_α (ft/sec^2)	3.6	2.4 – 9.2	

Table 1: Model parameters for estimating delay for a single vehicle

The parameters for the fundamental diagram u_f (free-flow speed), c (capacity), k_j (jam density), and w (congested wave speed) are obtained from detector measurements along the links of the arterial network. The loop detectors provide flow and occupancy measurements. The average speed is estimated assuming a vehicle effective length (i.e., the average length of the vehicle plus the detector length), typically 20-24 ft. Direct measurement of vehicle speeds can be obtained from double loop detectors.

Figure 10 shows a plot of flow vs. occupancy detector data from the Los Angeles ATSAC (Automated Traffic Surveillance and Control) central traffic control system. The detectors are located on each lane approximately 300 ft upstream of the intersection stopline. Vehicle counts and occupancy from each loop detector are collected every 30 seconds and stored in the ATSAC database. The detector data shown in Figure 10 represent one minute measurements for the through traffic aggregated across all travel lanes, averaged over multiple link locations. We obtain the model parameters based on the best statistical fit to the field measurements. The typical values obtained from the Los Angeles Lincoln Avenue arterial are c = 1,800 vph/lane, u_f = 35 mph, k_j = 180 veh/lane-mile and w = 14 mph. These values are in agreement with values for these parameters reported elsewhere.

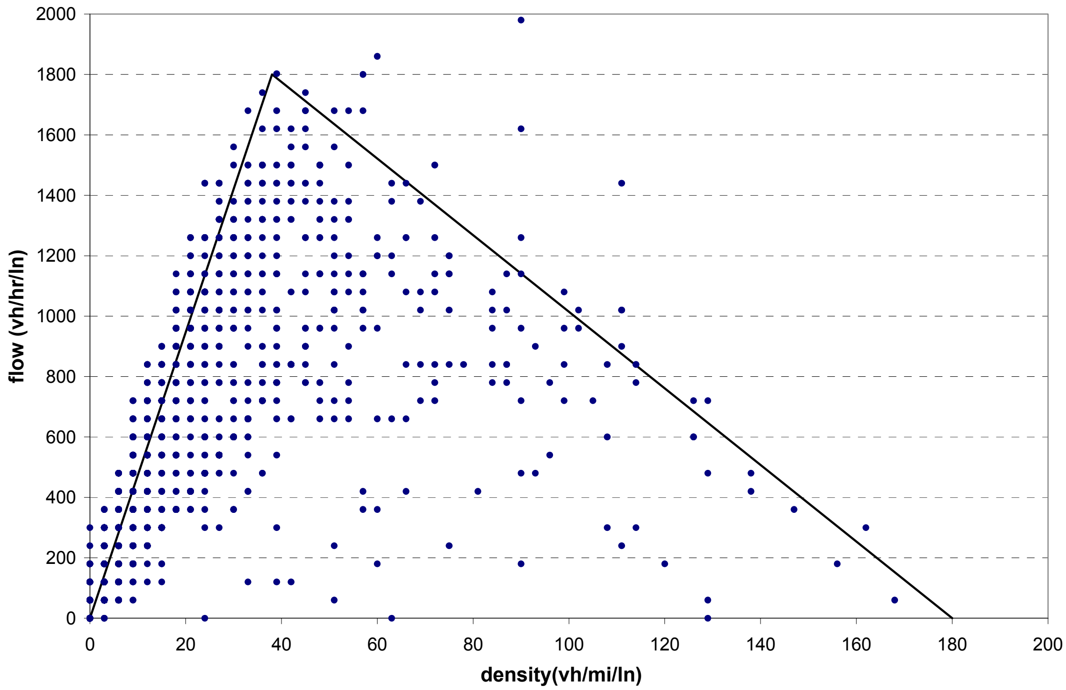

Figure 10. Flow-density diagram: loop detector data

Implementing the model

The implementation of the analytical model to estimate travel times from real-time surveillance data first involves the development of a database with a) signal timing plans at each intersection (cycle length, green times and offsets) for each time period of the day, and b) data from each loop detector in the arterial system. Next, the model parameters should be calibrated from the detector measurements as previously described.

Input to the model are the flow q and occupancy data O from detectors at 30 second intervals. The data are stored and verified for accuracy and then processed as follows:

1) The time of vehicle arrivals at the downstream signal is estimated using the speeds estimated at the detector location.
   The platoon dispersion analysis is engaged for distances longer than 500 ft (platoon ratio smaller than 0.9).

2) The model adjusts the phase times at the downstream signal considering the signal offsets and the queue discharge time t_q from the previous cycle, and determines whether the first vehicle arrives during the green time based on the arrival times from Step 2.

   The queue discharge time $t_q = 0$ in the first signal cycle. It is also assumed that are no residual queues at the beginning of the analysis period.

a. If the first vehicle arrives during the green time, then the number of vehicles crossing the intersection without delay is $g' \cdot q$, where g' is the adjusted effective green time and q is the measured flow in this time interval. If $g' > 30$ sec then all vehicles are undelayed.

   If $g' < 30$ sec, the remaining platoon of vehicles $(30 - g') \cdot q$ is delayed:
   i. The delay is estimated from Equations (10) and (17)
   ii. The red time of the next cycle is extended by the amount of queue discharge time t_q using equation (20) for N_q equal to the number of delayed vehicles
   iii. The average delay of all vehicles is the weighted average delay of delayed and uninterrupted vehicles (sec/veh)

   .

b. If the first vehicle arrives during the adjusted red phase, the delay is calculated from Equations (10) and (17) for all vehicles.
   If the number of delayed vehicles is greater than the capacity at the signal $g' \cdot c$, the delay of the unserved vehicles (residual queue) increases by one red time interval, as described in the estimation of oversaturation delay.

   Under heavy congestion the queue length may exceed the available link length and blocks the outflow from the upstream intersection. This condition is detected in the model using the flow and occupancy measurements. If the occupancy exceeds 40 percent and the flow (service rate) is less than the capacity, then the service rate at the signal is affected by spillback. The model accounts for the blocking effect by calculating the green time available as the ratio of the measured outflow over the capacity rate.

MODEL APPLICATION

The proposed model was applied to two arterial test sites: The M street arterial in Washington DC, and Lincoln Avenue in Los Angeles. The first site is typical of urban arterials in downtown areas; the second test site is typical of multilane major arterials on urban/suburban areas.

M Street, Washington DC

The study section consists of seven closely spaced signalized intersections. The average signal spacing is 460 ft. There are two through lanes on each direction of the arterial and the free-flow speed is 30 mph. Most of the signals are two phase fixed-time operating on a common cycle length of 60 seconds. Signal coordination favors the eastbound direction on the arterial. Data on the study network, geometrics and traffic volumes were available from previous research studies on traffic control. The study site has also been coded into the

CORSIM microscopic simulation model (FHWA, 2003) and the model was calibrated in the previous studies.

To apply the model, first loop detectors were coded in each link to obtain the loop count and occupancy for input to the analytical model. Also, a time-varying traffic demand pattern was input to the CORSIM model starting with low flows increasing to higher flows and then decreasing back to lower flows to emulate the growth and decay of traffic demand. Figure 11 shows the model estimated travel times vs. CORSIM predicted values for each signal cycle.

The results show that the model predicts travel times per cycle with an error smaller than 5% in most signal cycles. The differences are mostly due to the variability in the driver/vehicle characteristics generated by the CORSIM model. Note also that under high flows the travel time in the westbound direction increases dramatically as opposed to the eastbound direction. This is because the offsets favor signal progression in the eastbound direction which result in high delays and long queues westbound. The proposed model predictions correctly track the pattern of demand variations and delays over the analysis period.

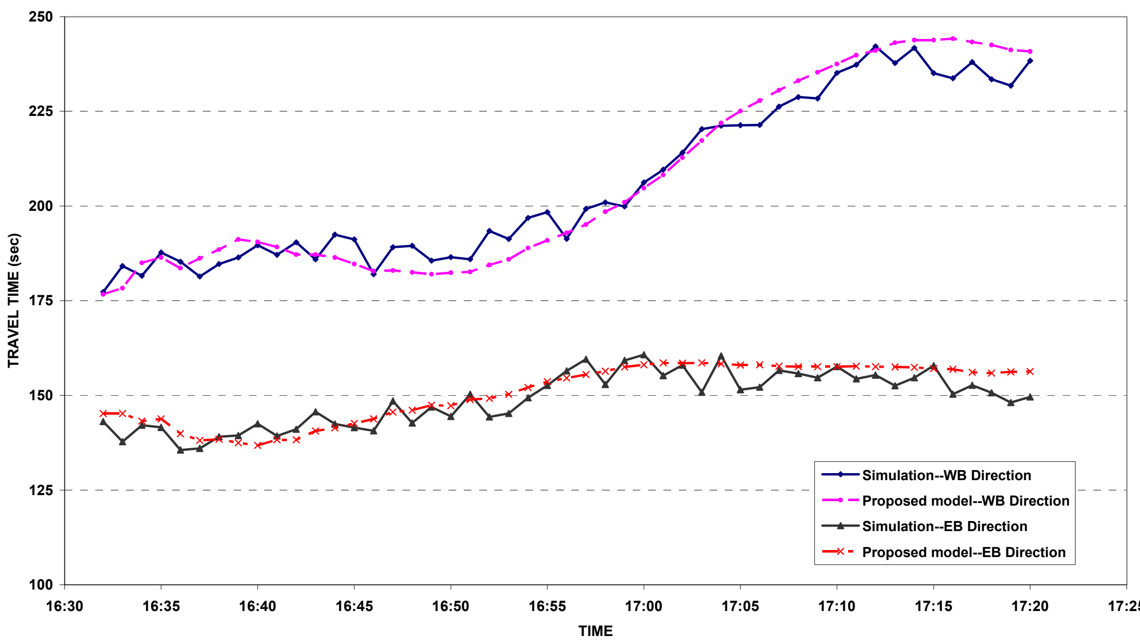

Figure 11. Simulated vs. model predicted travel times on M Street arterial

Lincoln Avenue, Los Angeles

The selected test site is 1.42 mile long stretch of a major urban arterial north of the Los Angeles International Airport, between Fiji Way and Venice Boulevard in the cities of Los Angeles and Santa Monica. The study section includes with seven signalized intersections with link lengths varying from 500 to 1,600 feet. The number of lanes for through traffic per link is three lanes per direction for the length of the study area. Additional lanes for turning movements are provided at intersection approaches. The free flow speed is 35 mph. Traffic signals are all multiphase operating as coordinated under traffic responsive control as part of the Los Angeles ATSAC central traffic control system. System loop detectors are located on each lane approximately 300 ft upstream of the intersection stopline. Detectors are also placed on the major cross street approaches. Data (vehicle count and occupancy) from each loop detector are collected every 30 seconds and stored in the ATSAC database.

A field study was undertaken obtain a comprehensive database of operating conditions in the study area. First, basic data on intersection geometrics, spacing and free-flow speeds were obtain from field surveys. Next, manual turning movement counts at each intersection and floating car studies were undertaken for a four hour period (6-10 am) on Wednesday May 26.th. The study period enabled us to obtain data for a wide range of traffic conditions (from low volume off-peak conditions, peak period conditions and post-peak mid-day flow conditions). Floating cars runs were performed at 7 minute headways. The floating cars were instrumented with laptop computers and GPS units that recorded vehicle location and speed on each second. Finally, the loop detector and signal timing data for the study period were obtained from the ATSAC database.

Traffic demand is high especially during the peak hour. Traffic volumes are heavily directional with the higher through and turning volumes in the northbound direction. The average travel speeds on the test section are 25 mph during the offpeak times and drop to about 10 mph during the peak hour in the heavily traveled northbound direction. The average travel speeds are about 25 mph and remain fairly constant in the southbound direction throughout the analysis period. System cycle lengths range from 100 seconds early in the analysis period (6:00 to 6:30 am) to a maximum of 150 sec during the periods of highest traffic volume (7:30 to 8:30 am).

Figure 12 shows a comparison between the loop and manual counts on intersection approaches at the test sites (a total of 336 data points). The results verify that the loop detectors are working properly at the study section. The loop counts on the average are about 6% higher than the manual counts, with most of the differences on the high through volume approaches. This “undercounting” of manual counts translates to about 2 veh/lane/signal cycle which could be expected in manual counts at multilane intersection approaches.

The proposed model was then applied to the test using as inputs the loop detector and signal timing data from the ATSAC system to estimate the link and total travel times on each travel direction. The test section was also simulated using the CORSIM model. Figure 13 and 14 show for each travel direction the field measured travel times from the floating cars, the predicted travel times from the proposed model, and the CORSIM simulated travel times.

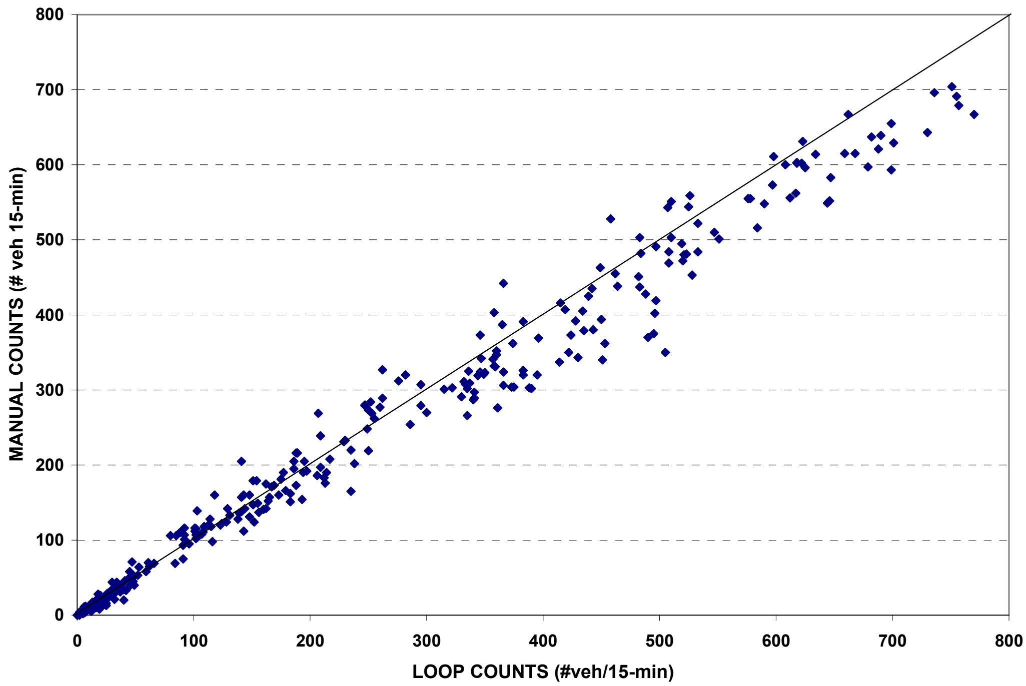

Figure 12. Lincoln Avenue: comparison of manual and loop counts

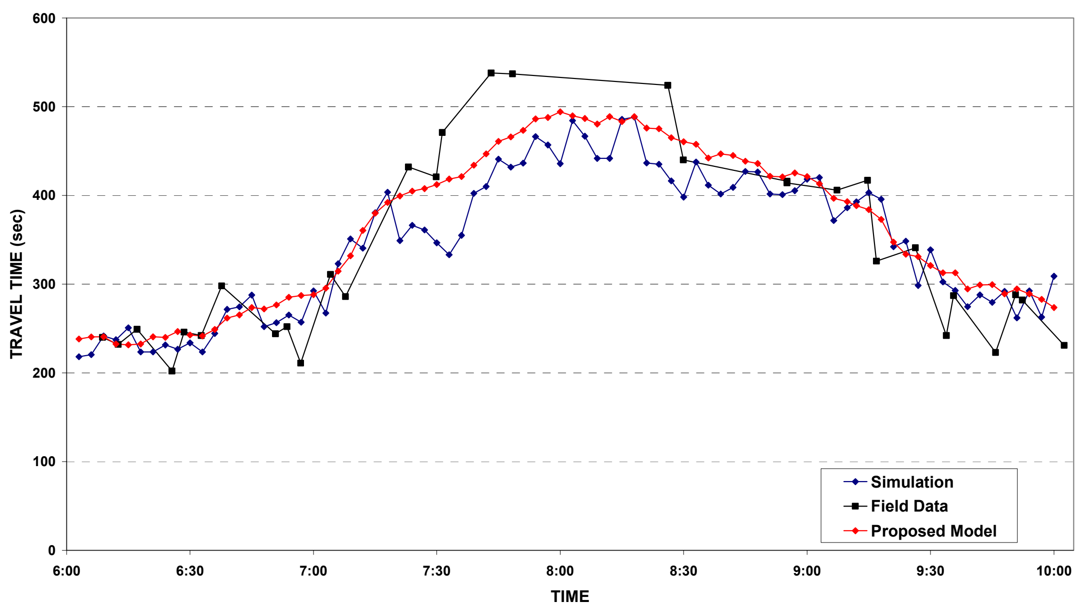

Figure 13. Travel time on Lincoln Avenue northbound direction

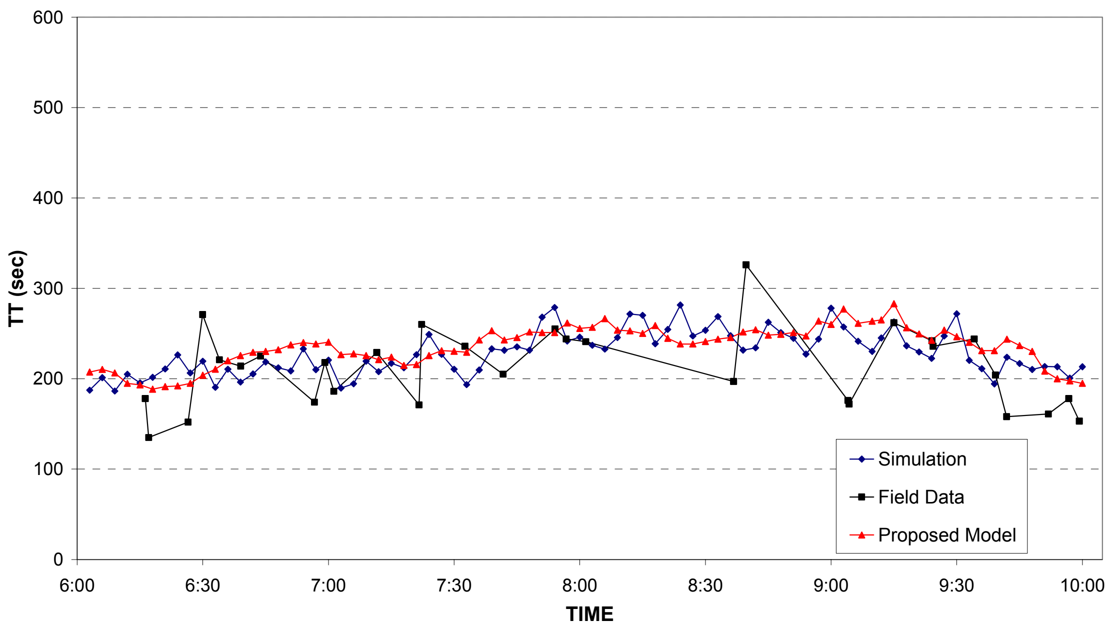

Figure 14. Travel Time on Lincoln avenue southbound direction

The results show that the model accurately estimates the arterial travel times on both directions of the arterial over the entire analysis period. The results also show that the CORSIM model can reasonably replicate observed conditions so it can be used as a supplementary data source to test the model predictions under different demand patterns and control scenarios. The evaluation of the differences between field data and model predictions particularly during the peak periods should take into consideration that the model predicts the average travel time per cycle, while the field measurements represent the average of a sample of vehicles throughout the signal cycle.

CONCLUSIONS

An analytical model is proposed to estimate the travel times on arterial streets based on data commonly provided by system loop detectors (flow and occupancy) and the signal settings (cycle length, green times and offsets) at each traffic signal. The model is based on kinematic wave theory to model the spatial and temporal queuing at the traffic signals and explicitly considers the signal coordination in estimating traffic arrivals at the intersection. The model is straightforward to implement and unlike other approaches does not depend on site specific parameters or short term traffic flow predictions that make very difficult their transferability to other locations.

The application of the proposed model on two arterial sites and comparisons of the estimated travel times with simulated and field data show that the model accurately predicts travel times on the selected sites. The difference between predicted and measured link travel times per cycle was within 5 percent under a wide range of traffic conditions.

For comparison purposes, we estimated the travel times on both test sites using several commonly used travel time estimation methods discussed in the introduction. The results indicate that these methods produced large differences against field and simulated values (Skabardonis, et al, 2004). For example, the method based on the average speeds estimated from the loop detector data (spot speed method) underestimates travel times by 36 percent on M Street arterial and overestimates more than 100 percent on Lincoln Avenue site. These results indicate that the proposed model is superior to existing approaches and can provide estimates that are accurate for both operational analyses and real-time monitoring of traffic conditions.

Ongoing work by the research team includes a) the further testing and evaluation of the model on other arterial sites, b) the refinement of the model to estimate additional performance measures commonly used by operating agencies, e.g., number of stops, cycle failures, travel time reliability and c) its implementation in a pilot arterial performance measurement system for the city of Los Angeles (APeMS) similar to the PeMS system for freeways (Choe et al, 2002). The data from loop detectors and signal settings from the ATSAC system will be transferred to UC Berkeley over the internet. The APeMS system will process and verify the data, and calculate the performance measures in real-time. Current and historical information on the performance measures will be provided via a standard web interface accessible via the internet.

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