AURORA RNM – A MACROSCOPIC SIMULATION TOOL FOR ARTERIAL TRAFFIC MODELING AND CONTROL

Andy H. F. Chow (corresponding author)
California PATH
1357 S. 46th Street,
Richmond Field Station, Bldg 452,
Richmond, CA 94804
Phone: (510) 665 3657, Fax: (510) 665 3537
ahfchow@path.berkeley.edu

Gabriel Gomes
California PATH
1357 S. 46th Street,
Richmond Field Station, Bldg 452,
Richmond, CA 94804
Phone: (510) 665 2120, Fax: (510) 665 3537
gomes@path.berkeley.edu

Alex A. Kurzhanskiy
Electrical Engineering and Computer Science
University of California,
Berkeley, CA 94720
Phone: (510) 374-4606
akurzhan@eecs.berkeley.edu

Pravin Varaiya
Electrical Engineering and Computer Science
University of California,
Berkeley, CA 94720
Phone: (510) 642-5270, Fax: (510) 642-7815
varaiya@eecs.berkeley.edu

Submitted on: July 31, 2009; in revised form: November 8, 2009.

Word count: 4350 + 10 Figures + 1 Table = 7100

ABSTRACT

This paper presents an extension of the macroscopic Aurora-RNM traffic simulator from freeways to arterial networks developed under the TOPL (Tools for operational planning) project at University of California, Berkeley. Aurora-RNM is a graphical traffic simulation environment which implements the cell transmission model. The developed arterial model can simulate both pre-timed and actuated signal controllers. Each signal controller at an intersection can coordinate up to eight signal phases. To simulate actuated controllers, the macroscopic traffic quantities in Aurora-RNM are converted to discrete vehicle actuations by using Poisson approximation. We select a 0.9 mile-long arterial of San Pablo Avenue in Albany, CA, to test the performance of the arterial model. Traffic data are collected by wireless sensors installed at the field. Simulation experiments show that Aurora RNM can represent the actual traffic condition and the popular Paramics micro-simulation reasonably well in terms of travel time estimation. This research contributes to the area of arterial traffic management as our proposed macroscopic model will be more efficient than the widely used microscopic models.

Keywords: Tools for operational planning, arterial traffic signal control, cell transmission model, macroscopic simulation.

1. INTRODUCTION

This paper documents an extension of the Aurora-RNM traffic simulator from freeways to arterial networks developed under the TOPL project. TOPL, the Tools for Operations and Planning (*1*), is a set of software programs designed to facilitate the construction, calibration, maintenance, and use of macroscopic models of vehicular traffic systems. Aurora-RNM is a graphical traffic simulation environment which implements the cell transmission model of Daganzo (*2*; *3*). Until now Aurora-RNM has been limited to freeways. However the advantages of macroscopic simulation over microscopic simulation - increased simulation speeds, automated calibration, etc. - apply to both freeways and arterials. These added capabilities will enable the simulation of entire networks within Aurora-RNM, and thus the evaluation of system-wide control strategies.

Several considerations arise in the development of a macroscopic model for arterials. First, arterial signalling systems are more complex than the onramp metering systems of freeways because they incorporate multiple phases that would consist of conflicting movements. Second, in contrast to freeway control algorithms (e.g. ALINEA in *4*), intersection controllers can be influenced by individual vehicle actuations, as is the case with the green extensions involved in the actuated control algorithm. This consideration led to the loop detector model of Section 2.3. Finally, the measures of effectiveness typically used in arterial studies are different from those that pertain to freeways, and include quantities such as the quality of progression, the number of stops, and the incidence of oversaturation.

There are many control algorithms for intersections with varying degrees of complexity. The most standard ones are pre-timed, isolated actuated, and coordinated actuated control.Under pre-timed control, intersections are operated according to a predefined timing plan with fixed cycle lengths, phase intervals, and phase sequences. Pre-timed signal controllers do not adapt to current traffic conditions. Nevertheless, they can be used to coordinate a string of intersections in order to enhance the progression of traffic. An isolated actuated signal controller sets the signal timings based on vehicle actuations measured by detectors. The signal controller determines which phase to start, when it should start, and its duration according to the number of vehicles detected by the various detectors. The algorithms of pre-timed control and actuated control can also be combined to form an actuated controller with coordination. Coordinated actuated controller operates according to an underlying signal timing plan, with the green splits in each cycle adjustable based on the real time traffic detection. Further details can be found in Gomes and Skabardonis (*5*).

The effectiveness of an arterial traffic control algorithm is often measured by travel time along the arterial. There are many simulation models capable of predicting travel time, with widely varying levels of detail. On the coarser end of the spectrum are the steady-state formulas of the Highway Capacity Manual (*6*). HCM estimates the average delay at a signal controlled intersection based on mean traffic conditions. The average delay is given by $d = d_1 PF + d_2$, with

$$d_1 = \frac{1}{2}\frac{C[1-(g/C)]^2}{1-(g/C)\omega}, \tag{1}$$

$$d_2 = 900T\left[(\omega-1)+\sqrt{(\omega-1)^2+\frac{8\kappa x}{QT}}\right]. \tag{2}$$

g and C represent the effective green time and cycle time at the intersection; ω is the ratio of incoming flow rate to the maximum outflow (saturation flow) Q; T is the study horizon; κ is a parameter which is set to 0.5 for pre-timed controllers, and depends on the flow-capacity ratio x for actuated controllers. DF is the delay factor which equals 1 for pre-timed, and 0.85 for actuated control. Further detail on the delay formulae can be found in Rouphail et al. (*7*). After determining the delays at individual intersections, the arterial travel time is then derived by adding the delays to the free flow travel time of the arterial. The disadvantages of the HCM method include its independence of several important tunable parameters in both the pre-timed and actuated algorithms, and its inability to capture traffic dynamics and other temporal effects such as coordination.

On the other side of the granularity spectrum we find the widely used microscopic models. These models apply certain predefined rules to reproduce the trajectories of individual vehicles in the traffic environment. Popular commercial packages include Paramics, VISSIM and CORSIM. In principle, microsimulation can capture fine detail of a real world system. However, calibrating and running a microsimulation model can be expensive in terms of computational and human resources.

To the best of our knowledge, Lo (*8*) should be the pioneer who studied arterial traffic and the associated signal control system by using macroscopic cell transmission model. Lo et al. (*9*) validated the cell-based signal control model by using a small arterial in Hong Kong. An optimal pre-timed plan was also derived by using Genetic Algorithm (GA). Lo and Chow (*10*) adopted the same traffic model and studied the effects of data quality and performance of the GA-based optimizer in a range of scenarios. Maher (*11*) and Wong et al. (*12*) adopted the cell transmission model to investigate the reverse capacity (see Allsop, *13*) of closely packed signalized intersections. Nevertheless, one should note that the above studies only consider pre-timed control system, which is independent of real-time traffic conditions. Moreover, these studies were limited to small networks, without implication on large-scale applications. Our study fills the gap by developing a real-time and operational model for arterial traffic estimation and control.

This paper reports an initial study, intends to document our recent findings, and is not conclusive. The paper is organized as follows. Section 2 presents the arterial model in Aurora RNM. Section 3 details the case study for testing the proposed model. Section 4 gives concluding remarks.

2. Aurora arterial model

Aurora-RNM is a link-node based adaptation of the cell-transmission model. Links represent uninterrupted stretches of road, which are joined at nodes. Traffic dynamics along the links are captured by macroscopic cell transmission model (Daganzo, *2*; *3*). Any division or merging of traffic streams, including intersections, is represented within Aurora-RNM by a node. This section starts with a review on cell transmission model. The representation of signal-controlled intersections and the associated traffic signal controllers are then presented.

2.1 Review of the cell transmission model (CTM)

Cell transmission model (CTM) was formulated by Daganzo (*2*; *3*) as a convergent discretization of the Lighthill-Whitham-Richards (LWR) model (*14*; *15*) with certain advantageous properties. Under CTM, the road network is discretized into a collection of sections or 'cells' as shown in Figure 1. Each section i has a jam density $\bar{\rho}_i$. When there is no congestion, the traffic stream moves from one section to the next at free flow speed, v.

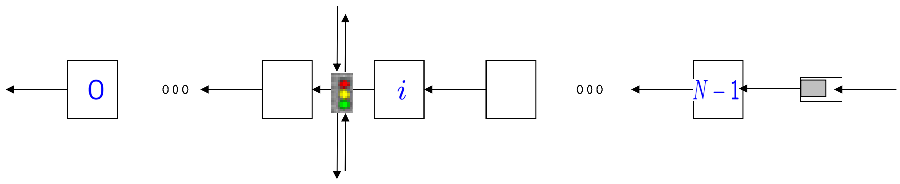

FIGURE 1. Cell representation of a link

The sections are numbered from the downstream 0, to the upstream N-1. For a given time step t, the traffic density in each section i is denoted by $\rho_i(t)$, and per-step flow of traffic ready to enter it, $f_i(t)$. The inflow into section i (i.e. the outflow of section i+1) within the time step at t is governed by a fundamental diagram given by

$$f_i(t) = \min\{v\rho_{i+1}(t), Q_i, w[\bar{\rho}_i - \rho_i(t)]\}, \tag{3}$$

where Q_i is the maximum flow that can enter section i within a given time step; w is the backward shockwave speed specified by the fundamental diagram. This formulation covers both congested and uncongested regimes. After determining $f_i(t)$ for all i at the current time t, the density in cell i at the next time step is updated by the following conservation equation:

$$\rho_i(t+1) = \rho_i(t) + \frac{\Delta t}{\Delta x_i}[f_i(t) - f_{i-1}(t)], \tag{4}$$

where Δt and Δx_i are respectively the lengths of simulation time step and the cell i.

2.2 Signal-controlled intersection

In the current version of Aurora RNM, each signal controller at an intersection can coordinate up to eight signal phases, which is same as the NEMA (*16*) standard. Figure 2 shows the representation of a complete eight-phase intersection in Aurora RNM. Node 10 represents the intersection controlled by the traffic signal. There are intermediate nodes (Nodes 12, 14, 16, and 18) on each incoming link (Links 120, 140, 160, and 180) where the traffic splits into through and left-turning portions. The odd-numbered links (i.e. 121, 141, 161, and 181) are for through traffic, while the even-numbered links (i.e. 122, 142, 162, and 182) are for left-turn traffic. Aurora RNM does not explicitly consider right-turn traffic because right-turn movements are often not protected at signalized intersections in the United States. In our case study arterial (see Section 3), none of the right-turn phases is protected.

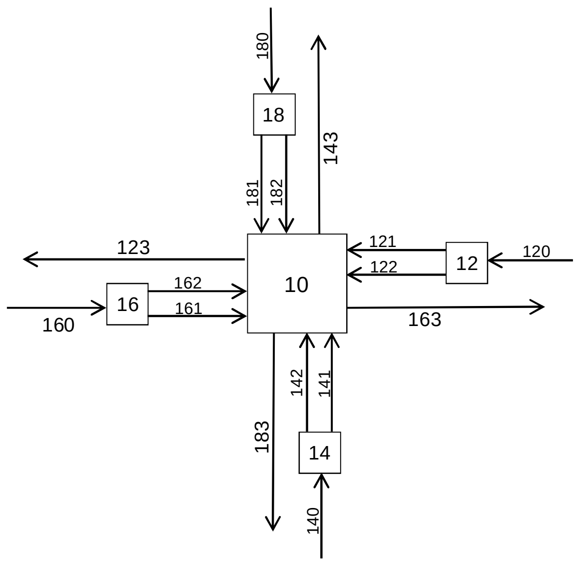

FIGURE 2. Aurora RNM representation of an intersection

2.3 Modeling traffic signals

Following Lo (*8*), the effect of a signal controller can be captured by formulating the capacity term Q_i of a cell *i* after an intersection as a binary time-varying variable as:

$$Q_i(t) = \begin{cases} q_{\max} & \text{during green phase} \\ 0 & \text{during red phase} \end{cases}, \tag{5}$$

where q_{max} is the maximum discharge rate from the intersection.

Pre-timed controllers operate the signal according to a predefined timing plan while actuated controllers operate based on discrete vehicle actuations. To simulate the actuated controllers, the macroscopic traffic quantities in Aurora-RNM need to be converted to discrete (microscopic) vehicle actuations. Assuming exponentially distributed inter-vehicle spacing, the probability of having *m* loop detector actuations within a given time interval Δt follows Poisson distribution:

$$P(m|\lambda) = \frac{\lambda^m e^{-\lambda}}{m!}. \tag{6}$$

We then show how the expected number of actuations λ can be estimated based on the macroscopic quantities. We consider that a vehicle will be detected whenever it contacts the detector. Consider a stream of vehicles with macroscopic quantities: flow *q*, density ρ, mean speed *v*, travelling through the detector. The average number of actuations λ induced by this traffic stream in time interval Δt can be determined as the density ρ multiplied by the length spanned by the traffic stream and the detector in Δt. That is,

$$\lambda = \theta\rho, \tag{7}$$

where the length θ spanned by the traffic stream and the detector within Δt is,

$$\theta = v\Delta t + L_{veh} + L_{\det}, \tag{8}$$

in which L_{veh} and L_{det} are respectively the average vehicle and loop detector lengths. In the United States, the average length of a vehicle L_{veh} is 16 ft and of a detector L_{det} is 6 ft. Hence, we have

$$\lambda = q\Delta t + \rho(L_{veh} + L_{\det}). \tag{9}$$

After defining the probabilistic distribution and its parameter, the number of vehicle actuations is then generated over time by using Monte Carlo method for each simulation time step Δt.

With the vehicle actuations, the actuated controller decides the green time allocated to each phase accordingly. The green time allocated to a particular phase consists of two portions: the initial green and the extended green. The initial green is calculated based on the number of vehicles registered by the detectors during the preceding red. This initial green is limited by predefined maximum and minimum values. The minimum green specifies the green allocated to the phase even when no vehicle is registered. During the green phase, each detected approaching vehicle is given an extended green (typically 2 -3 sec) to move through the intersection. This extended green terminates when either:

1. **max-out:** the predefined maximum green time is reached;
2. **gap-out:** the time gap between consecutive actuations exceeds the largest permitted gap.

The 'largest permitted gap' is a decreasing function of time as shown in Figure 3. It starts at a maximum value (maxgap) and starts to decrease after a vehicle is detected on a conflicting phase, until it reaches a prescribed minimum value (mingap).

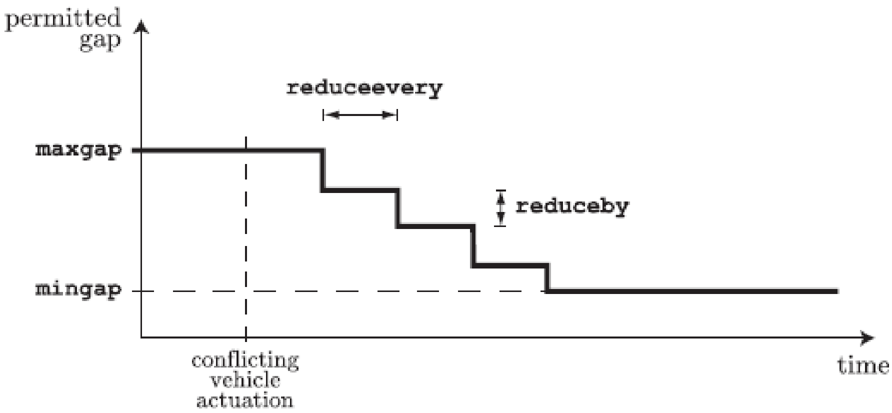

FIGURE 3. Largest permitted gap function

3. CASE STUDY

3.1 Location and signal control information

We select a 0.9 mile-long test segment of San Pablo Avenue in Albany, CA as shown in Figure 4. The arterial segment starts at intersection Fairmount and ends at intersection Buchanan. The segment consists of seven signal-controlled intersections at: Fairmount (point A), Carlson, Brighton, Clay, Washington (point B), Solano (point C), and Buchanan (point D).

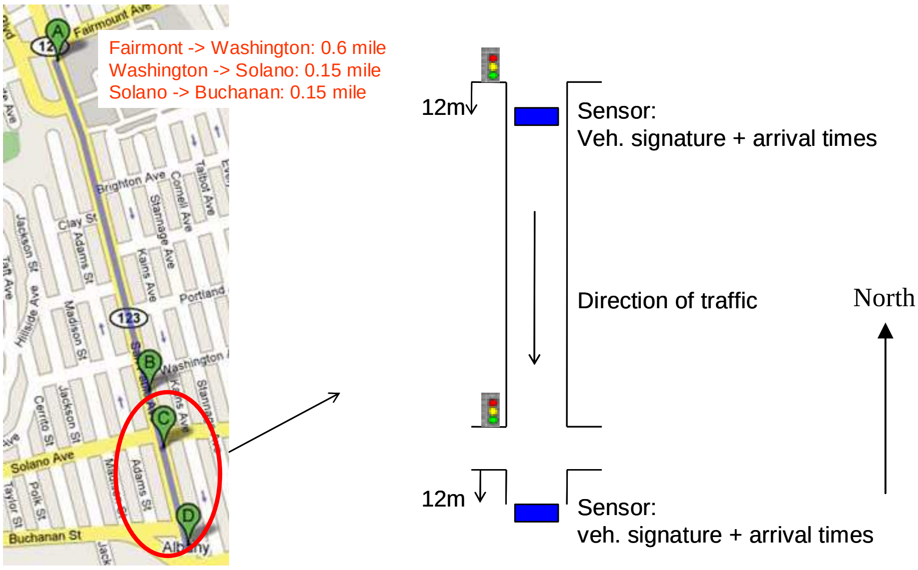

FIGURE 4. San Pablo Avenue, Albany, CA

The traffic signals are operated by a coordinated actuated controller in which the signals follow an underlying timing plan while the green splits are adjustable according to real time traffic detection. Table 1 shows the underlying pre-timed plan in use during our study period. The table includes the offsets, phase durations, and phase sequences. The phases are numbered based on the NEMA *(16)* system as shown in Figure 5. Phases 2 and 6 are assigned respectively to northbound (from D to A) and southbound (from A to D) mainline traffic. Other phases are then assigned accordingly. A phase (1-8) does not appear at an intersection in Table 1 means that the phase is not protected or it does not exist at that intersection (e.g. in case of a T-junction). The offset of an intersection refers to the time difference between the end time of green given to the mainline traffic (i.e. Phases 2-6, which are bold in Table 1) at that intersection and the master system base time. Offsets are used to coordinate traffic movement along the mainline. The intersections are running under a common cycle time, 108 seconds. The phase duration includes the green time, an all-red clearance time which is 2 sec, and a yellow time which is 4 sec for Phases 2-6, and 3 sec for other phases.

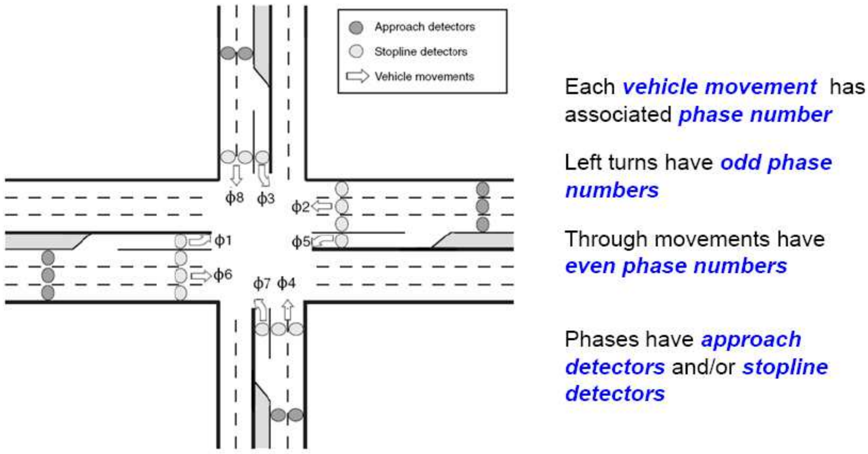

FIGURE 5. NEMA specification of intersection

Based on the pre-timed plan in Table 1, each vehicle detected during the green phase may further receive a green extension of 2 sec with the actuated component. This is decided based upon whether the "max-out" or "gap-out" condition is matched as discussed in Section 2.3. The maximum duration for the mainline Phases 2-6 is 90 sec. The maximum durations for the cross streets are adjusted accordingly such that the sum of the durations allocated add up to the cycle time, 108 sec. The largest permitted gap is fixed at 2 sec for all phases except Phases 2-6. For Phases 2-6, the maximum value of the largest permitted gap is 5 sec. When a vehicle is detected on a conflicting phase, the largest permitted gap function starts to decrease by 0.1 sec for every 2 sec until it reaches the predefined minimum value, which is 3 sec.

Intersection	Offset (sec)	Phase 1	Phase 2	Duration (sec)
Carlson	79	**2**	**6**	**37**
		2	5	18
		4	8	31
		1	6	22
Brighton	28	**2**	**6**	**83**
		4	8	25
Clay	61	**2**	**6**	**82**
		4	-	26
Washington	53	**2**	**6**	**86**
		4	-	22
Solano	31	**2**	**6**	**38**
		2	5	12
		3	7	22
		4	8	22
		1	6	14
Buchanan	82	**2**	**6**	**81**
		4	-	27

TABLE 1 Pre-timed plan – San Pablo Avenue

3.2 Traffic data

Wireless sensors are installed at locations A, B, C, and D as shown in the figure to collect data of southbound mainline traffic (i.e. Phase 6 in our notation). The sensors are located 12m downstream of the associated intersections. Note that segment AB spans four signalized intersections (Fairmount, Carlson, Brighton, and Clay), while segments BC and CD each span one signalized intersection. The sensors collect signatures of vehicles passing through and the corresponding times of detection. This study adopts data collected from 13:00 to 13:30 on 23 May 2008, which was a peak period as reflected in Kwong et al. (*17*). Kwong et al. (*17*) developed an algorithm to match the signatures of vehicles between segments AB, BC, CD, and the entire arterial AD. Consequently, travel times of vehicles travelling along each segment are determined. With the wireless sensor data and Kwong's matching algorithm, we can derive the demand profile, segment free flow speeds, and intersection discharge rates.

3.2.1 Demand

Kwong et al. (*17*) reported that there were 211 vehicles detected at the upstream location A during 13:00 – 13:30 on the day of interest. The discrete vehicle actuations are aggregated into 5-sec flow rates, and the corresponding demand profile is plotted in Figure 6. It is noted that we cannot derive traffic inflows from the cross streets because we do not have any sensor on the cross streets.

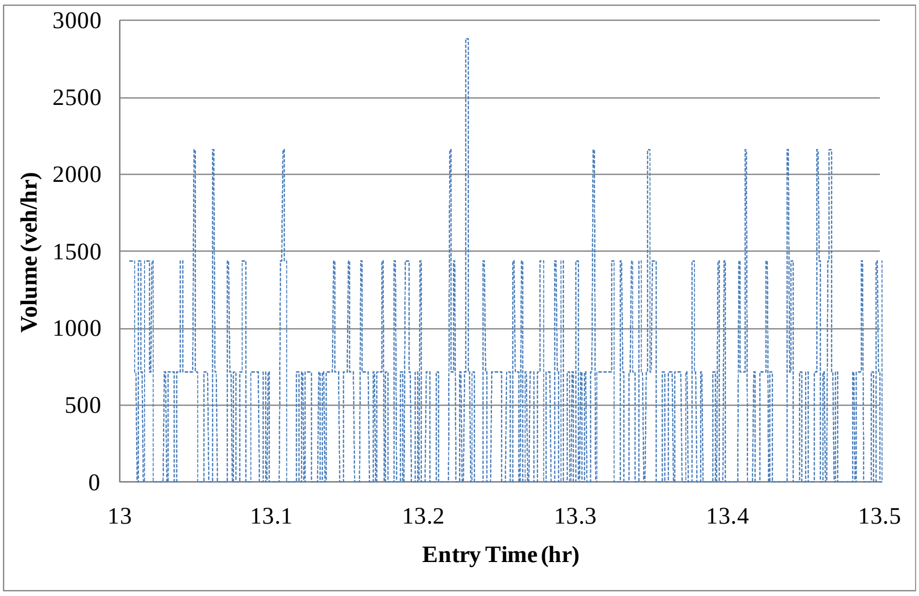

FIGURE 6 Demand profile

3.2.2 Free flow speed

The free flow speed along a segment can be derived from the free flow travel time along that segment. The free flow travel time can be determined from Kwong's estimates. Figure 7 shows that the travel times estimated by Kwong et al. (*17*) for segments A-B, B-C, and C-D respectively by using the vehicle matching algorithm. The travel times are plotted against the corresponding entry times during the period 13:00 – 13:30. To remove the unreasonable outliers due to estimation error (e.g. zero travel times), we regard the free flow travel times as the 10^{th}

percentile travel times along each segment. Consequently, we estimate the free flow travel times of segments A-B, B-C, and C-D are respectively 80sec, 25sec, and 25 sec. The length of segments A–B (Fairmount to Washington), B-C (Washington to Solano), and C-D (Solano to Buchanan) are given in Figure 4. The free flow speed for segments A-B, B-C, and C-D are then calculated respectively as 27 mph, 21.6 mph, and 21.6 mph.

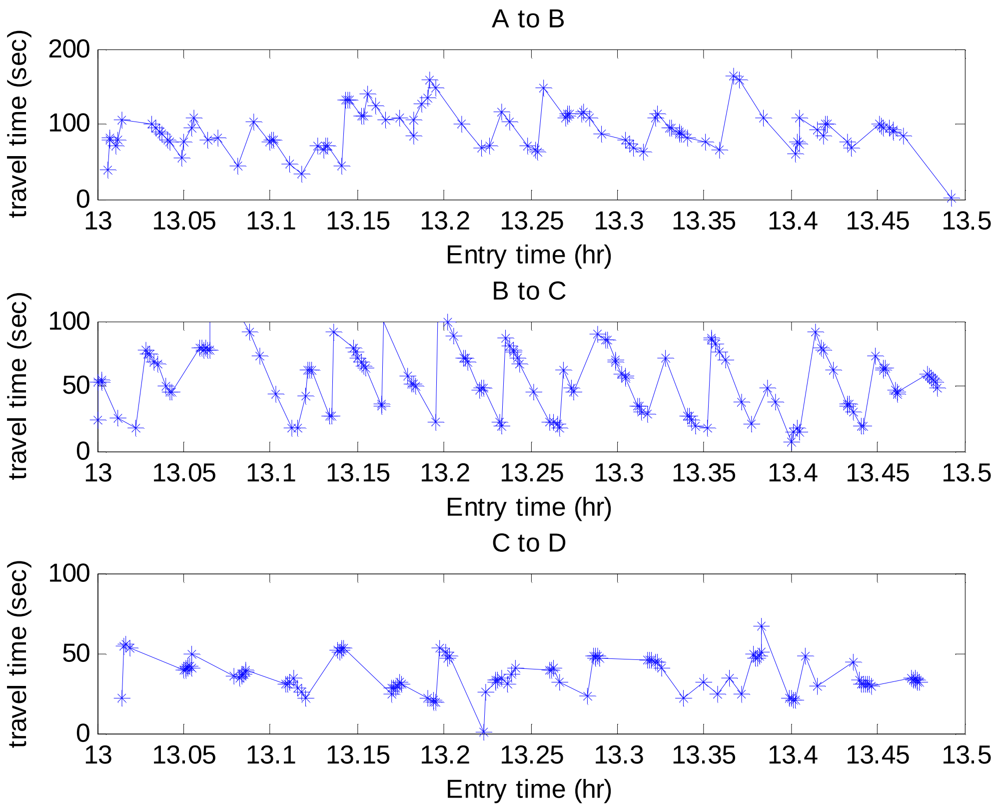

FIGURE 7 Travel time estimates (Kwong et al., *17*)

3.2.3 Discharge rate

Figure 8 shows the stem-plot of the link travel times against the corresponding exit times of vehicles from locations B, C, and D. The exit times are calculated by summing the link entry times of the vehicles with the associated link travel times. The clusters of stems in the figure correspond to the platoons of vehicles discharged from the intersection at the starts of green.

Maximum discharge rate of an intersection is determined as the headway in seconds between vehicles in a discharging platoon, divided into 3600 seconds per hour (FHWA, *18*). It is common practice to observe the discharge times between the first five vehicles in the platoon (see Roess et al., *19*; Kwong et al., *17*), i.e.

$$q_{\max} = \frac{5}{(t_5 - t_1)/3600}, \tag{10}$$

where t_1 and t_5 respectively are the times of the first and the fifth vehicles crossing the downstream sensor at the start of the green of each cycle. The maximum discharge rate from an intersection is determined by averaging the $q_{\max}$ over several cycles. Following (10), the discharge rates (in veh/hr/lane) from locations B, C, and D are estimated respectively as 1753.8, 2056.3, and 1882.8.

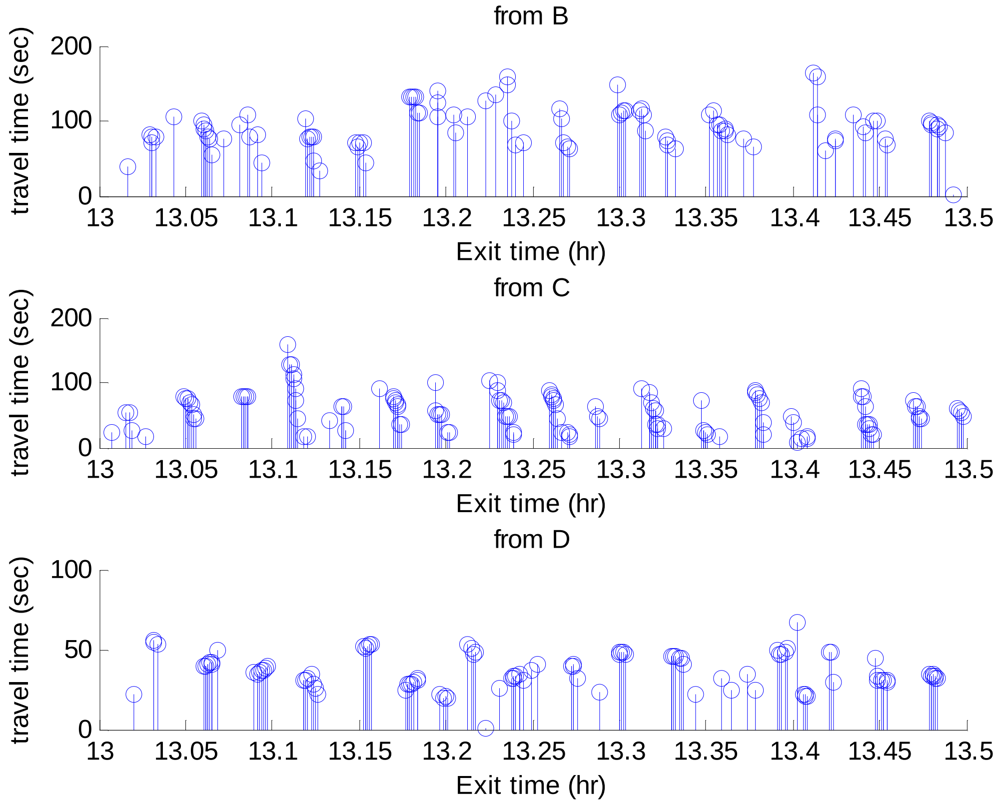

FIGURE 8 Discharge traffic estimates (Kwong et al., *17*)

3.3 Experiment

The information derived in Sections 3.1 and 3.2 can now be used to construct an arterial model of San Pablo Avenue with Aurora RNM. It is noted that as we do not explicitly collect traffic flow-density data, we cannot estimate the jam density for the arterial model. The present study assumes that the jam density is 200 veh/mile/lane, which should be a reasonable assumption for urban streets. Kwong et al. (*17*) matched the vehicles between A and D and derived their travel times. The travel times of the vehicles are adopted as the ‘ground truth’ for the experiment. The aim of this experiment is to compare the arterial travel time estimated by Aurora RNM with the one by Kwong et al. (*17*). Aurora RNM calculates the mean speed of traffic $v(i,z)$ in each segment (i.e. cell in the model representation) i during each time interval z. To determine the travel time in Aurora RNM, consider an infinitesimal probe vehicle entering the segment at time s_0. We then define $x_i(s)$ be the distance travelled by the probe in link i by time s, where

$$x_i(s) = \sum_{z=s_0}^{s} v(i,z)\Delta t \,. \tag{11}$$

The probe is said to be exiting the current link and entering the subsequent link i+1 on the arterial at time s_1 when

$$x_i(s_1) \geq \Delta x_i, \tag{12}$$

where Δx_i is the length of link i as discussed in Section 2.1. The travel time of the probe through this link is then determined as $(s_1 - s_0)\Delta t$. Apply the same methodology to other links, the travel time through the entire arterial can then be derived.

Figure 9 shows the travel time estimates by Kwong et al. (*17*), Aurora RNM with pre-timed and coordinated actuated controllers. Kwong et al. (*17*) estimated a mean travel time of 189.4sec with a relatively high standard deviation 41.3 sec (coefficient of variation, COV, 22%). The high COV was due to those outliers around or even higher than 250 sec, and those lower than 150 sec as shown in Figure 9. Aurora RNM estimates the mean travel times of 183.7sec and 189.5sec with pre-timed and actuated controllers respectively. The associated standard deviations are 18.4sec and 11.8sec. The performance of actuated controller is subject to further investigation as we do not have traffic volumes from the cross streets in the present study. With respect to Kwong's estimate, the percentage differences associated with the Aurora RNM controllers (pretimed and actuated) are within 3%. In fact, the good agreement between Aurora RNM and Kwong's estimates is surprising, because the demand profile should underestimate the amount of traffic on the road by neglecting the turning volumes. One explanation is that the arterial is far from saturation with most intersections having LOS A. Moreover, there are two lanes on the arterial for through traffic, plus one auxiliary lane for left turners at each intersection. This implies that the maximum flow on the arterial can reach about 4,000 vph. It is revealed from Table 1 that the average green-to-cycle ratio (g/c ratio) allocated to the mainline at the intersections is around 0.8 except Carlson (55/108 = 0.51) and Solano (50/108 = 0.46). Suppose that the green to cycle-time ratio (g/c ratio) allocated to the mainline at each intersection is 0.7, which is a conservative estimate, the capacity of each intersection along the arterial will then be 4,000 vph x 0.7 = 2,800 vph. This 2,800 vph capacity is higher than the demand at all times except a 5-sec interval at entry time 13:13 in which the demand reaches 2,880 vph. This suggests that demand volume may not have considerable effect on the travel time, while the signalling do.

For further comparison, we also calculate the arterial travel time by using the HCM delay formulas. Following formulas (1) and (2), the HCM delay estimates (in seconds) of the intersections at Carlson, Brighton, Clay Washington, Solano and Buchanan are respectively 12.7, 3.5, 3.8, 2.7, 16.3, and 4.1. It is noted that most intersections have a Level of Service (LOS) A, except Carlson (LOS B) and Solano (LOS C)[1]. The free flow travel time along the stretch is determined as 80 + 25 + 25 = 130 (min). The delay estimates are added to this free flow travel time and gives the total arterial travel time 173 sec. Note that the HCM approach uses a steady-state analysis that only gives the average travel time of all vehicles and does not consider any temporal variation. It is also noted that HCM estimate is 7% lower than Kwong's measurement. The underestimation may due to the fact that HCM ignores the temporal variation of travel time.

The arterial is also coded in Paramics micro-simulation. It is noted the micro-simulation parameters (e.g. gap acceptance for lane-changing; acceptable gaps between vehicles) are set as default as we do not have enough data to calibrate those parameters in the present study. Figure 10 shows the travel time estimates by Paramics. We only consider pretimed control as standard Paramics software only supports pretimed controller, although some plugins (e.g. *5*) have been

[1] According to HCM 2000, an intersection having a LOS A has an average delay < 5.0 sec; for LOS B, the average delay is in between 5.0 sec and 15.0 sec; for LOS C, the average delay lies between 15.0 sec and 25.0 sec.

developed through the application programming interface **(**API**)** to capture actuated control. The travel time in Paramics is given by a built-in function 'Trip-Analysis'. The results show that the travel time estimates given by Aurora RNM and Paramics are close to each other. The higher standard deviation in travel times in Paramics is due to the non-linear effect from the interaction between individual vehicles in microscopic simulation. It suggests that Aurora RNM, a much less computationally demanding tool, may be used as a substitute to Paramics for arterial traffic modelling and management.

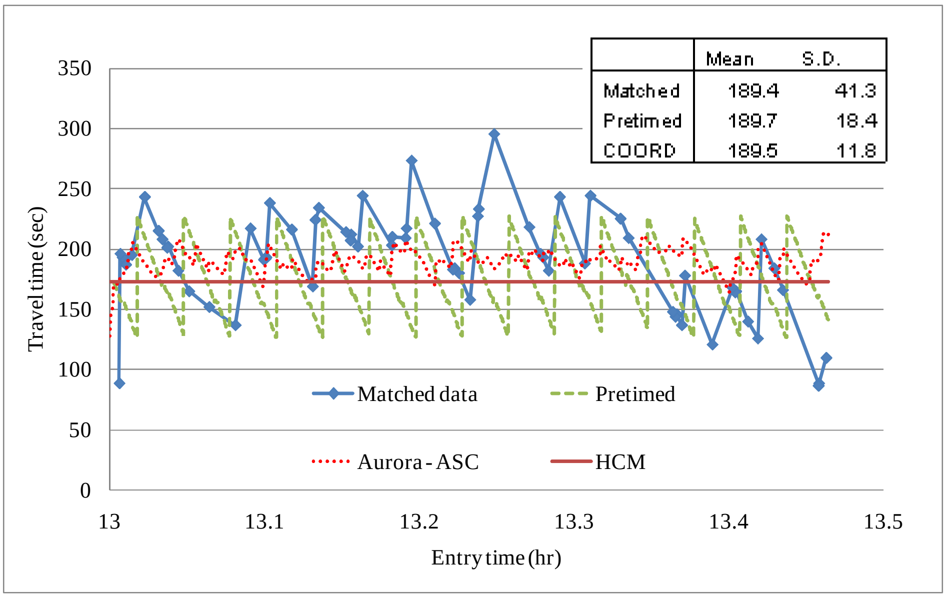

FIGURE 9 Travel time profiles

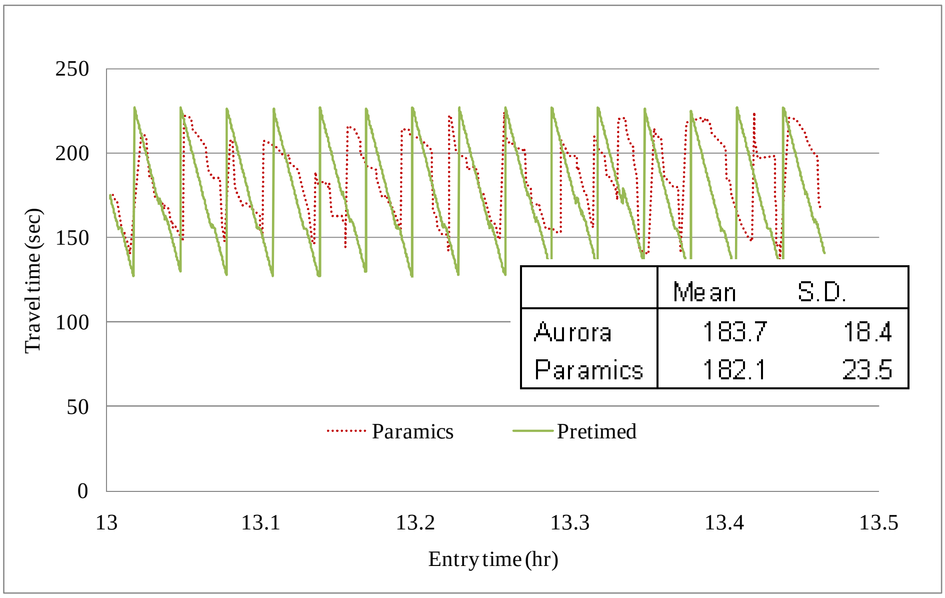

FIGURE 10 Travel time estimates of Paramics and Aurora RNM

4. CONCLUSIONS

This paper presents an extension of the macroscopic Aurora-RNM traffic simulator from freeways to arterial networks developed under the TOPL project at UC Berkeley. Aurora-RNM is a graphical traffic simulation environment which implements the cell transmission model. The developed arterial model can simulate both pre-timed and actuated signal controllers. Each signal controller can coordinate up to eight signal phases. To simulate actuated controllers, the macroscopic traffic quantities in Aurora-RNM are converted to discrete (microscopic) vehicle actuations by using Poisson approximation.

We select a 0.9 mile-long arterial of San Pablo Avenue in Albany, CA, to test the performance of the arterial model. Traffic signals at the field are operated by a coordinated actuated controller in which the signals follow an underlying timing plan while the green splits are adjustable according to real time traffic detection. Wireless sensors are installed at several intersections along the arterial to collect traffic data. We show the derivations of demand profile, segment free flow speeds, and intersection discharge rates from the wireless sensor data and Kwong’s matching algorithm. The travel time profile along the entire arterial is also determined.

The aim of the case study is to compare the arterial travel time estimate by Aurora RNM with the one by Kwong’s vehicle matching algorithm. The difference between the travel time estimates by Kwong and Aurora RNM is within 3%. This suggests that Aurora RNM can represent the actual traffic condition reasonably well. Nevertheless, the actuated controller is subject to further investigation as we do not have traffic volumes from the cross streets in the present study. Aurora RNM is also compared with the result generated by the popular Paramics micro-simulation. It shows that the travel time estimates given by Aurora RNM and Paramics are close to each other. It may suggest that Aurora RNM can be a promising substitute to micro-simulation. This research contributes to the area of arterial traffic management as our proposed macroscopic model will be more efficient than the widely used microscopic models in practice.

ACKNOWLEDGEMENTS

Comments from the anonymous referees and the TOPL research group are gratefully acknowledged. Some results in this paper were presented at 12th IFAC Symposium on Control in Transportation Systems on 2-4 September, 2009. This work was supported by the California Department of Transportation (Caltrans) T.O. 6614. The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California. This paper does not constitute a standard, specification, or regulation. The authors would also like to thank Karric Kwong of Sensys Networks for providing the wireless sensor data, and Meng Li of California PATH for providing the signal timing information.

REFERENCES

1. Kurzhanskiy, A.A. *Modeling and Software Tools for Freeway Operational Planning*. PhD thesis, Department of Electrical Engineering and Computer Science (EECS-2007-148), University of California, Berkeley, USA, 2007. Accessible on:

http://www.eecs.berkeley.edu/Pubs/TechRpts/2007/EECS-2007-148.html

2. Daganzo, C.F. The cell-transmission model: a simple dynamic representation of highway traffic. *Transportation Research Part B*, Vol. 28(4), 1994, pp. 269-287.

3. Daganzo, C.F. The cell-transmission model, part II: network traffic. *Transportation Research Part B*, Vol. 29(2), 1995, pp. 79-93.

4. Papageorgiou, M., H. Hadj-Salem, and J.M. Blosseville. ALINEA: A Local Feedback Control Law for On-Ramp Metering. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1320, Transportation Research Board of the National Academies, Washington, D.C., 1991, pp. 58-64.

5. Gomes, G. and A. Skabardonis. Paramics Plugin for Actuated Signal Control and First Generation UTCS. *California PATH Working paper* UCB-ITS-PWP-2006-08, 2006.

6. Transportation Research Board. *Highway Capacity Manual 2000.* National Research Council, Washington, D.C., 2000.

7. Rouphail, N, A. Tarko, and J. Li. Traffic flow at signalized intersections. In *Transportation Research Board Special Report 165 (Traffic flow theory)*, 2000, Chapter 9, Accessible on: http://www-cta.ornl.gov/cta/research/trb/tft.html.

8. Lo, H. A novel traffic signal control formulation. *Transportation Research Part A*, Vol. 33, 1999, pp. 433-448.

9. Lo, H., E. Chang and Y.C. Chan. Dynamic network traffic control. *Transportation Research Part A*, Vol. 35(8), 2001, pp.721-744.

10. Lo, H. and A.H.F. Chow. Adaptive traffic control system: control strategy, prediction resolution, and accuracy. *Journal of Advanced Transportation: ITS Special issue*, Vol. 36(3), 2002, pp. 323-347.

11. Maher M.J. Reserve capacity for a set of closely-spaced intersections. In: *Mathematics in Transport* (ed. B.G. Heydecker): *Selected Proceedings of 4th IMA international conference on Mathematics in Transport*, Elsevier, 2007, pp. 321-331.

12. Wong, C.K., S.C. Wong and H. Lo. Reserve capacity of a signal-controlled network considering the effect of physical queuing. In: *Transportation and Traffic Theory* (eds. R.E. Allsop, M.G.H. Bell, B.G. Heydecker): *Selected Proceedings of 17th International Symposium of Transportation and Traffic Theory*, Elsevier, 2007, pp. 533-554.

13. Allsop, R.E. Estimating the traffic capacity of a signalized road junction. *Transportation Research*, Vol. 6, 1972, pp. 245-255.

14. Lighthill, M.J. and J.B. Whitham. On kinematic waves: II. A theory of traffic flow on long crowded roads. *Proceedings of the Royal Society*, Vol. 229A, 1955, pp. 317-345.

15. Richards, P.I. Shockwaves on the highway. *Operations Research*, Vol. 4, 1956, pp. 42–51.

16. National Electrical Manufacturers Association (NEMA) *NEMA TS2 Traffic Controller Assemblies with NTCIP Requirements*, 1998.

17. Kwong, K., R. Kavaler, R. Rajagopal and P. Varaiya. Arterial travel time estimation based on vehicle re-identification using wireless magnetic sensors. *Transportation Research Part C*, Vol 17(6), 2009, pp. 586-606.

18. Federal Highway Administration. *Traffic Signal Timing Manual*, 2008. Accessible on: http://ops.fhwa.dot.gov/publications/fhwahop08024/index.htm.

19. Roess, R.P., W.R. McShane and E.S. Prassas. *Traffic Engineering* (3rd edition), Prentice Hall, 2004.