Aurora Arterial Modeler – A Macroscopic Tool for Urban Traffic Signal Control

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Abstract: This paper describes a macroscopic simulation model for urban signal control based on the Aurora Road Network Modeler (RNM) software package. The paper first presents the implementation of pre-timed and actuated signal controllers in a cell transmission model based simulator. Then a case study with wireless sensor traffic data is carried out to test the performance of the model. The results show that the macroscopic model produces good estimates of arterial travel time, as compared to predictions of the Highway Capacity Manual. The actuated controller produces significantly higher delay than its pre-timed counterpart, which agrees with our previous findings using micro-simulation. This research should contribute to the area of arterial traffic signal simulation and control as the proposed model is more computationally efficient than the more widely used microscopic simulation models.

Keywords: traffic light, urban traffic control, actuated controller, cell transmission model.

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 (Kurzhanskiy, 2007), 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 a modified version of the cell transmission model of Daganzo (1994; 1995). 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 are coordinated by a dual-ring structure which guarantees no simultaneous conflicting movements. Second, in contrast to freeway control algorithms (e.g. ALINEA in Papageorgiou et al., 1991), 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 green duration given to a particular phase consists of two portions: the initial green and the extension interval. The initial green is calculated based on the number of vehicles registered by the detectors during the preceding red interval. The initial green is limited by predefined maximum and minimum values. During the green interval, vehicles detected by an approach detector (some 100 ft. upstream of the intersection) are each given a certain amount of time, typically 2-3 sec, to move through the intersection. The extension interval ends when either the maximum green time is reached or the time gap between consecutive vehicle actuations exceeds the ‘largest permitted gap’. The ‘largest permitted gap’ is typically a decreasing function of time. It starts at a maximum value and starts to decrease after a vehicle is detected on a conflicting phase, until it reaches a prescribed minimum value. The algorithms of pre-timed control and actuated control can also be combined to form a coordinated actuated controller with coordination. Further details can be found in Gomes and Skabardonis (2006).

The effectiveness of a traffic control algorithm, whether pre-timed or actuated, is typically quantified in terms of the travel delay incurred by all users under typical conditions. There are many simulation models capable of predicting travel delay, with widely varying levels of detail.

On the coarser end of the spectrum are the steady-state formulas of the Highway Capacity Manual (HCM 2000, TRB). These estimate 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

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

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

g and C represent the effective green time and cycle time at the intersection; x 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. (2000). 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.

Our objective is to demonstrate that a macroscopic model of an arterial network is capable of producing estimates of travel delay comparable to both the HCM and microscopic models. This paper reports an initial study, intended to document our recent findings, and 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. METHODOLOGY

Aurora-RNM is a link-node based adaptation of the cell-transmission model. Links represent uninterrupted stretches of road, which are joined at nodes. Any division or merging of traffic streams, including an intersection, is represented within Aurora-RNM by a node. The following section describes the network topologies that model intersections in Aurora-RNM.

2.1 Signal-controlled Intersection

In addition to links and nodes, the intersection also contains a signal, which regulates the flow of traffic through the intersection. Each signal can coordinate up to 8 signal phases, that is, 8 independent stream of traffic that traverse the node. Following the NEMA (1998) standard, each phase is associated with a phase number: odd numbers for left turns and even numbers for through movements. The typical numbering scheme can be found in Gomes and Skabardonis (2006).

Figure 1 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.

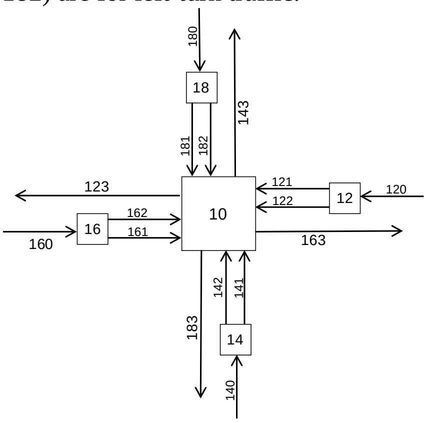

Fig. 1. Representation of an intersection in Aurora RNM

2.2 Review of the Cell Transmission Model (CTM)

CTM was formulated by Daganzo (1994; 1995) as a convergent discretization of the LWR model (Lighthill and Whitham, 1955; Richards, 1956) with certain advantageous properties. Under CTM, the road network is discretized into a collection of sections or ‘cells’ as shown in Figure 2. Each section k has a vehicle holding capacity N_k, which equals to the jam density multiplied by the length of section. When there is no congestion, the traffic stream moves from one section to the next at free flow speed.

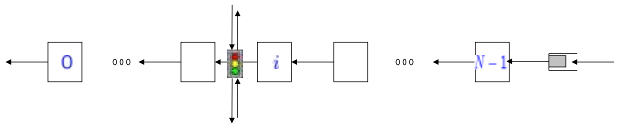

Fig. 2. Cell representation of a link

The sections are numbered from the downstream 0, to the upstream *j*. For a given time step, each section k has a number of vehicles in it, n_k, and per-step flow of vehicles ready to enter it, f_k. The inflow into section *k* (or the outflow of section *k*+1) within the time step at t is governed by a fundamental diagram given by

$$f_k(t) = \min\{n_{k+1}(t), Q_k, \delta[N_k - n_k(t)]\}, \tag{3}$$

wherein Q_k is the maximum number of vehicles that can enter section k within a given time step; $N_k - n_k(t)$ is the "available space" in section k at time *t*; and δ is the ratio of shockwave speed to free-flow speed (i.e., w/v). This formulation covers both congested and uncongested regimes.

After the flows have been determined for each cell for a specified time step, the network is updated with the following conservation equation:

$$n_{k+1}(t+1) = n_{k+1}(t) + [\, f_{k+1}(t) - f_k(t)\,]. \tag{4}$$

2.3 Modelling Traffic Signals in CTM

The signal controller influences the flow on the links by manipulating the $Q_i(t)$ in (3) for links entering the signal controlled node. During the green phase (determined by the signal controller), the capacity of the link is set to an intersection discharge rate $q_{\max}$, and during the red phase it is set to zero:

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

Pre-timed controllers operate the signal according to a predefined timing plan while actuated controllers operate based on discrete vehicle actuations. Aurora-RNM considers traffic in macroscopic quantities: flow, density, and speed. Therefore, a model of a loop detector is required to convert these macroscopic quantities to microscopic vehicle actuations.

Define flow, density, and speed of a traffic stream as *q*, ρ, *v*; L_v and L_d are the average vehicle and loop detector lengths respectively. Typically in the United States, L_v equals 16 ft and L_d is 6 ft. Assuming normally distributed inter-vehicle spacing, we have that the probability of *m* loop detector actuations within a given interval Δt behaves according to the following Poisson distribution:

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

The following shows 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 that there is a traffic stream of vehicles with macroscopic quantities: *q*, ρ, *v*, travelling through the detector. The average number of actuations λ induced by that traffic stream in Δt can be determined as the density ρ multiplied by the length spanned by the traffic stream and the detector in Δt. That is,

$$\lambda = \bar{s}\rho, \tag{7}$$

where the length $\bar{s}$ spanned by the traffic stream and the detector within Δt is,

$$\bar{s} = v\Delta t + L_v + L_d. \tag{8}$$

Hence, we have

$$\lambda = q\Delta t + \rho(L_v + L_d). \tag{9}$$

After defining the probabilistic distribution and its parameter, realizations of the number of actuations are then generated over time, and hence the associated actuated green durations are derived.

3. CASE STUDY

We selected a 0.9 mile-long test segment on southbound San Pablo Avenue in Albany, CA as shown in Figure 3. The arterial segment starts at Fairmount and ends at Buchanan. The segment consists of seven signal-controlled intersections at: Fairmount, Carlson, Brighton, Clay, Washington, Solano, and Buchanan. The free flow speed is 30 mph, the estimated saturation flow is 1800 veh/hr, and the jam density is 200 vehs/mile. The traffic signals are actuated with coordination in which the signals follow an underlying timing plan while the green splits are adjustable according to vehicle actuations.

Sensys Networks Inc. installed wireless sensors at locations A, B, C, and D as shown in the figure. Those sensor locations are immediately downstream (12m) of the associated intersections. Note that segment AB spans four signalized intersections (Fairmount, Carlson, Brighton, and Clay) segments BC and CD each span one signalized intersection. The sensors collect signatures of vehicles passing through and the corresponding times of detection. Kwong et al. (2008) developed a methodology to match the signatures such that vehicles can be re-identified at different sensor locations. As a result, the travel time of each vehicle travelling along the arterial is known.

Kwong et al. (2008) performed a test with data collected from 13:00 to 13:30 on 23 May 2008. They matched the signatures of vehicles between segments AB, BC, CD, and the entire arterial AD. They report that there were 252 vehicles detected at A, which is the entry to the arterial segment of interest.

Moreover, there were 68 out of 211 matches (matching rate: 32%) between A and D. The travel times of these 68 vehicles is adopted as the 'ground truth' for the experiment in this paper.

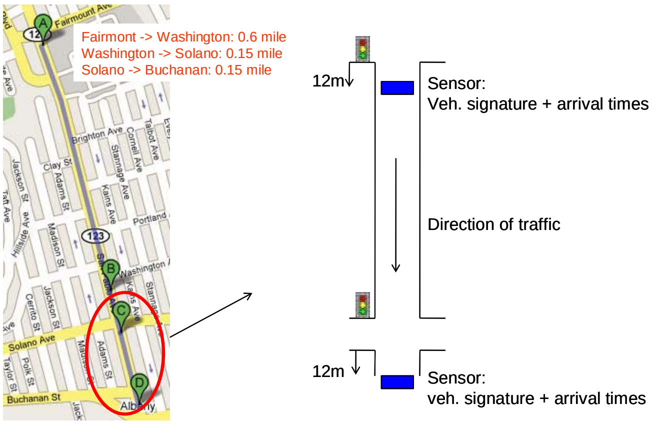

Fig. 3. San Pablo Avenue, Albany, CA

Signal timings and traffic demand are required to formulate an arterial traffic model. We also need to derive the arterial travel time profile from Aurora RNM simulation in order to compare with Kwong's result.

3.1 Signal Timings

Table 1 shows the underlying pre-timed plan in use at the site during the study period (13:00 – 13:30). The timing plan was designed by California Department of Transportation (Caltrans). The numbers in the table are in the unit of seconds. The entry '-' means that the phase is unprotected or does not exist at that intersection. In the table, the offset of an intersection refers to the time difference between the end time of green given to the main arterial traffic (i.e. Phases 2 and 6) at that intersection and the master system base time. The timing plans have a common cycle time of 108 seconds. The all-red clearance time is 2 sec for all phases. The yellow time is 4 sec for Phases 2 and 6, and 3 sec for the other phases. Note that the phase number of the traffic stream (southbound San Pablo Ave.) that we are looking at is '6'.

		Green time per phase							
Intersection	Offset	1	2	3	4	5	6	7	8
Carlson	79	22	55	-	31	18	59	-	31
Brighton	63	-	83	-	25	-	83	-	25
Clay	58	-	82	-	26	-	82	-	-
Washington	51	-	86	-	22	-	86	-	-
Solano	77	14	50	22	22	12	52	22	22
Buchanan	107	-	81	-	27	-	81	-	-

Table 1 Pre-timed plan – San Pablo Avenue

Table 2 depicts the parameters of the actuated signal controller. In the table, 'min initial' is the minimum initial green, 'max gap' and 'min gap' are respectively the maximum and minimum values of the permitted time gap between consecutive vehicle actuations for extending green. All numbers in the table are in seconds.

		1	2	3	4	5	6	7	8
Carlson	min initial	10	20	---	15	4	20	---	15
	maxgap	2	5	---	2	2	5	---	2
	mingap	1.2	5	---	1	1	5	---	1
Brighton	min initial	---	20	---	12	---	20	---	12
	maxgap	---	5	---	3.5	---	5	---	3.5
	mingap	---	3	---	2	---	3	---	2
Clay	min initial	---	20	---	4	---	20	---	---
	maxgap	---	5	---	3.5	---	5	---	---
	mingap	---	3	---	1.5	---	3	---	---
Washington	min initial	---	30	---	4	---	30	---	---
	maxgap	---	5	---	3	---	5	---	---
	mingap	---	3	---	1	---	3	---	---
Solano	min initial	6	22	2	2	4	22	4	6
	maxgap	3	5	2	2	3	5	3	3
	mingap	1	3	2	2	1	3	1	1
Buchanan	min initial	---	20	---	8	---	20	---	---
	maxgap	---	5	---	4.5	---	5	---	---
	mingap	---	3	---	2.5	---	3	---	---

Table 2 Parameters of actuated controller – San Pablo Ave.

It is further noted that pre-timed plans are only in operation during 0700 – 2000 each day. At other times, the signals are set 'free' of the timing plan and they run simply in an actuated manner without coordination.

3.2 Demand Profile

Based on the data collected in Kwong et al (2008), the demand profile between entry times 46800 (13:00) and 48600 (13:30) is constructed as in Figure 4. The demand profile is derived from the actuations of those 252 vehicles detected by sensor A. The demand is aggregated into 5-second intervals.

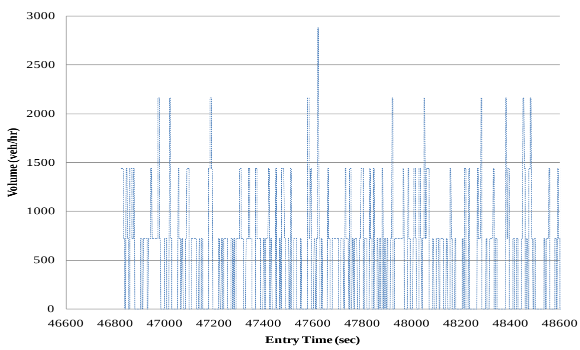

Fig. 4 Demand profile

3.3 Arterial Travel Time

The aim of this study is to compare the arterial travel time estimated by Aurora RNM with Kwong et al. (2008). Given the signal timings and the demand, the underlying CTM in Aurora RNM calculates the mean speed of traffic $v(i,z)$ in each link i during each time interval z.

To determine the travel time, consider an infinitesimal probe vehicle entering a link on the arterial at time s_0. We suppose that this vehicle is a particle with no mass so its presence does not affect the density and hence the speed on other links.

Define $x_i(s)$ be the distance travelled by the probe in link i by time s, we have

$$x_i(s) = \sum_{z=s_0}^{s} v(i,z)\Delta t \,. \tag{10}$$

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

$$x_i(s_1) \geq D_i \,, \tag{11}$$

where D_i is the length of link i. 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.

3.4 Results

Figure 5 shows the travel time profiles estimated by Kwong et al. (2008), Aurora RNM with pre-timed and coordinated actuated controllers. Kwong et al. (2008) 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 is due to those outliers around or even higher than 250 sec, and those lower than 150 sec as shown in Figure 5. Aurora RNM estimates the mean travel times of 183.7sec and 189.5sec for pretimed and actuated controllers respectively. The associated standard deviations are 18.4sec and 11.8sec,

For comparison, we also calculate the delay using the HCM formulas. The HCM delay estimates are shown in Table 3. It is noted that most intersections have a Level of Service (LOS) A, except Carlson (LOS B) and Solano (LOS C)[1]. The delay estimates are added to the arterial free flow travel time, which is 160 sec, this gives the total travel time, 203 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.

Delay (sec)	Carlson	12.7
	Brighton	3.5
	Clay	3.8
	Washington	2.7
	Solano	16.3
	Buchanan	4.1
Free-flow travel time (sec)		**160.0**
Total travel time (sec)		**203.0**

Table 3 HCM delay estimates

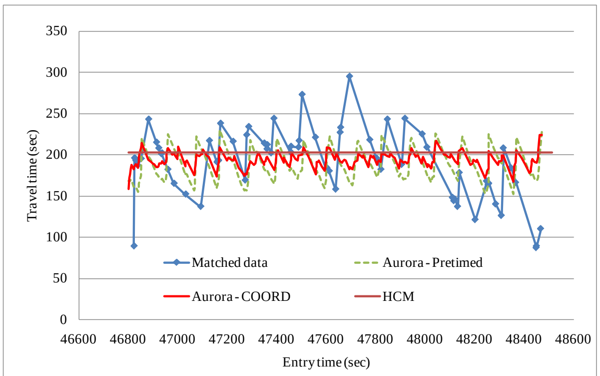

Fig. 5 Travel time profiles

[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.

Figure 5 shows the travel time estimates of Aurora RNM can represent the real situation reasonably well. With respect to Kwong's estimate, the percentage differences associated with the Aurora RNM controllers (pretimed and actuated) are within 3%. In fact, such good estimation 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. Suppose that the green to cycle-time ratio (g/c ratio) at each intersection is around 0.5, which is a conservative estimate, the capacity of each intersection will then be 4,000 vph x 0.5 = 2,000 vph. This 2000 vph capacity is significantly higher than the maximum demand rate, 1440 vph. This suggests that demand volume may not have considerable effect on the travel time, while the signal timings do. Nevertheless, one should note that the so-called 'ground truth' given by the matched data indeed is an estimate itself. Currently Sensys Networks Inc. has set up video cameras at the site to test the performance of their matching algorithm. Results will be reported in the future.

The arterial was also coded in Paramics micro-simulation and Figure 6 shows the results. We only consider pretimed control as standard Paramics software only supports pretimed controller, although some plugins (e.g. Gomes and Skabardonis, 2006) have been developed through the application programming interface **(**API**)** to capture actuated control. The travel time in Paramics was given by a built-in function 'Trip-Analysis'. The figure suggests that the travel time estimates given by Aurora RNM and Paramics are close to each other. Aurora RNM, a much less computationally demanding tool, may be used as a substitute to Paramics.

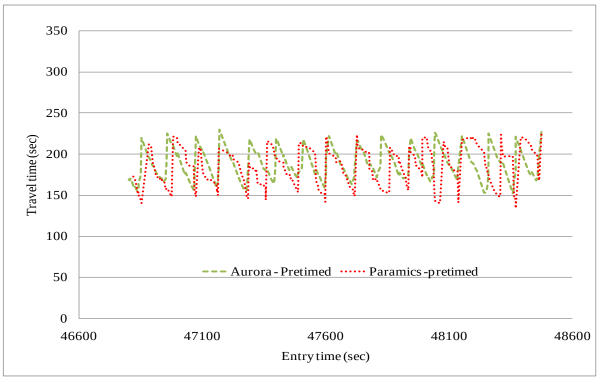

Fig. 6 Travel time estimates – Paramics vs Aurora RNM

4. CONCLUSIONS

This paper demonstrates a macroscopic tool for arterial traffic signal control based on the Aurora RNM software package. The paper presents the implementation of pre-timed and actuated signal controllers in a CTM based macroscopic simulator. A case study at San Pablo Avenue with wireless sensor traffic data was carried out to test the performance of the model.

The results show that the macroscopic Aurora RNM is able to give good estimates of arterial travel time, although this may be due to the low traffic volume on the arterial. We will look at situations with different and higher degrees of saturation in the future. Further study also includes investigation of estimating turning traffic with limited sensor data. We will also investigate the design of control strategies based on the simulation. This research should contribute to the area of arterial traffic signal control as our proposed macroscopic model is more efficient and computationally economical than the widely used microscopic models.

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