Modelling Travel Speed on Signalised Arterial Roads at the Planning Stage

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Performance-oriented road planning is fundamental in ensuring that roadways can satisfactorily serve the purposes for which they are planned. This study is aimed at developing a methodology for the estimation of the expected performance (travel speed) along signalised arterials at the planning stage. To this end, this study reviews common travel speed (or travel time) estimation models, and then investigates travel speed along hypothesised 4-lane signalised arterial roads using microsimulation software VISSIM. The impacts of link length, link travel time, cycle length, traffic flow rate, and offset type (simultaneous or alternating preferential) on travel speed are clarified. The necessity of this work was reiterated, and a discussion made of the necessary steps in order to model the relationship between arterial travel speed and flow.

Key Words : *travel speed, arterial road, planning, signalized intersection, traffic simulation*

1. INTRODUCTION

Signalised arterial roads play a significant role in connecting urban centres to higher hierarchy roads. In order to fulfil their function, arterial roads need to allow vehicles to travel to their destinations at the requisite travel speed (or in a given travel time). The travel speed that can be achieved along signalized arterials is mainly influenced by the traffic control at the intersections.

Not only the presence of traffic signals, but other factors such as traffic demand, signal settings and their spacing have an influence on the achievable travel speed.

Estimation of the "achievable speed" is important for performance-based road planning. However, most speed estimation models tend to be aimed at the operational stage and require a lot of data that is not readily available to planners.

Therefore, this study aims to investigate the factors that have a significant impact on travel speed along signalised arterials, and the necessary steps towards modelling the travel speed for planning purposes.

The analysis is conducted by traffic simulation using microscopic simulation software VISSIM in which several scenarios are explored.

This study is limited to 4-lane signalized arterials (two lanes per travel direction, and a right-turn bay at each intersection approach). Furthermore, the study is limited to passenger cars only in the traffic flow and a 50-50 directional split.

Offset patterns that treat both travel directions along the main travel direction are considered: simultaneous offsets and *alternating preferential offsets* – on an arterial (East – West) with two links, priority is given to the Eastbound traffic in the first link, and to the Westbound traffic in the adjacent link.

2. TRAVEL TIME MODELS

This chapter gives a review of some travel time and travel speed estimation models and methodologies throughout the literature.

The review is conducted in order to identify factors with significant impact on travel speed, as well as potential travel speed model formulations.

(1) The Highway Capacity Manual, HCM

In the HCM[1], the travel time is estimated as a combination of the *free-flow travel time* and signal delay. For planning purposes, the HCM delay model is simplified in order to eliminate the need for variables that are not typically available to planners.

In the Planning and Preliminary Engineering Applications Guide to the Highway Capacity Manual[2], control delay for a lane group is given by **Eq.1.**

$$d = d_1 PF + d_2 + d_{unsig} \tag{1}$$

Where
d = control delay (s/veh)
d_1 = uniform delay (s/veh)
PF = progression adjustment factor (unitless)
d_2 = incremental delay (s/veh)
d_{unsig} = analyst-provided estimate of unsignalised movement delay, if any (s/veh)

Uniform delay d_1 is represented by **Eq.2**.

$$d_1 = \frac{0.5C(1-g/C)^2}{1-[\min(1,X)g/C]} \tag{2}$$

Where
g = effective green time (s)
C = cycle length (s)
X = volume-to-capacity ratio

PF values are recommend based on whether the progression quality is good (some degree of coordination), average (random arrivals), or poor (poor coordination), taking on values of 0.70, 1.00, and 1.25 respectively.

Basically, the better the progression, the lower the *PF*, leading to lower delay. Without detailed information of the progression quality, the *PF* can be defaulted to a value of 1.00 (for random arrivals).

(2) Travel speed model by Tarko et al (2006)

In the 2006 work by Tarko *et al*[3] the authors reformulated the HCM delay formula by representing the proportion of the cycle length used by the different movements at an intersection using the corresponding demand flows. Additionally, the remaining variables that would not be known at the planning stage were replaced by model parameters. The resulting model is shown in **Eq.3**.

$$V_i = \frac{3600}{\dfrac{3600}{V_0} + \dfrac{a_1}{l} \bullet \exp(a_2 l) \bullet \left(1 - \dfrac{a_3 F_i}{F_1 + F_2}\right) \bullet \dfrac{\left(1 + a_4 \dfrac{F_s}{n_s}\right)^2}{1 - a_5 \dfrac{F_i}{n_i}}} \tag{3}$$

Where
V_i = travel speed in direction i (mph)
V_0 = cruise speed (mph)
l = average distance between adjacent signalised intersections (mi)
F_i = flow in the analysed direction
F_1 and F_2 = one-way flows along the arterial street
F_s = flow crossing the major road (veh/h) [select the stronger one-way volume on each side street and calculate the average]
n_s = average number of through lanes in one direction on side streets
n_i = average number of through lanes in the considered direction on the major streets
a_1, a_2, a_3, a_4, a_5 = model parameters to be calibrated

Generating a data set of the variables in their model using microsimulation software CORSIM, the model parameters were estimated. Except for a slight overestimation, which was corrected by applying an adjustment factor, their model provided a good estimate of travel speed to both simulated data and data from a field study.

This is a logical approach as most signal setting procedures, including the one used in this study, rely on approach demand to allocate green time to each movement. Tarko et al's results also show that a reasonable approximation of the travel speed can be obtained using the limited data available to planners.

(3) Skabardonis-Dowling Model

With the aim to improve travel time estimation for planning purposes, Dowling and Skabardonis[4,5] made modifications to the standard BPR function that involved increasing the rate of drop in speed at capacity. Additionally, they proposed a queuing analysis process for *v/c* ratios over 1.0, from which a form of the travel time function was shown as in **Eq.5** by Xiong and Davis[6].

$$TT = \left(\frac{L}{FFS} + 0.5NC\left(1 - \frac{g}{C}\right)^2 PF\right)\left(1 + 0.05\left(\frac{v}{c}\right)^{10}\right) \tag{5}$$

Where
FFS = free-flow travel speed
N = number of signals in the link
g = effective green time
C = cycle length
PF = progression adjustment factor

The progression adjustment factor is given by:

$$PF = \frac{(1-P)\, f_{PA}}{1 - \dfrac{g}{C}} \tag{6}$$

Where
P = proportion of vehicles arriving on green
g/C = proportion of green time available

f_{PA} = supplemental adjustment factor for platoon arriving during green (approximately equal to 1)

(4) Comments on the models

The models of travel time and travel speed that have been discussed differ in their input requirements. Altogether, some factors can be identified which are necessary for the estimation of travel time and therefore travel speed. These include *traffic volume*, free flow travel time (*free-flow speed* and *link length*), *capacity*, *number of intersections*, and signal timing information (*effective green time*, *cycle length*, and the *progression adjustment factor*).

Some of the factors discussed do not act in isolation in their effect on travel speed. For example, assuming the same free-flow travel time, whether or not a vehicle can immediately pass through the downstream intersection depends on the signal indication as the vehicle arrives. And the number of vehicles that can pass through without stopping is similarly dependent on how long the effective green time is (and essentially, the cycle length). This interrelationship between the factors was analysed by Koshi[7] who determined that it was possible to minimise average delay by setting the cycle length as a function of the roundtrip link travel time.

Through the review of several models of travel time and travel speed estimation, various factors that can potentially influence the travel speed along signalised arterials were identified.

Although the impacts of some factors can be easily understood and perhaps modelled, there exist several interactions that might make the development of a purely theoretical model challenging.

For example, it is known that vehicles departing from an upstream signalised intersection tend to travel in a platoon. If the platoon travels a short distance to the downstream signalised intersection, more vehicles can make use of the downstream signal green time (if the signals are coordinated). But, the longer the distance between the two intersections, the more the platoon disperses, hence fewer vehicles arrive on green. This means the impact of the signal coordination on delay becomes smaller.

None of the models reviewed explicitly consider the possibility of platoon dispersion along the arterial and how it can impact the travel speed, nor the offset patterns hypothesised in this study.

One possibility of incorporating these factors in travel speed estimates is by developing a reasonable range of various scenarios and based on the resulting travel speed (for example through simulation), combinations of factors that yield a certain level of coordination can be grouped for modelling.

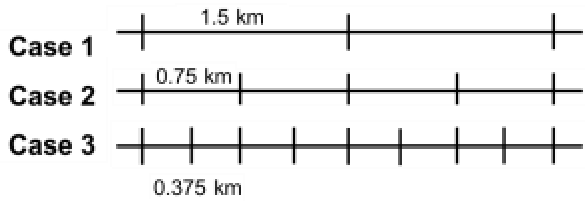

Fig.1 Arterial network layout

In this study, an attempt is made to hypothesise a range of possible scenarios regarding signalised arterial layouts, traffic flow, signal timing information among other factors. Then, using traffic simulation methods, the average travel speed for all combinations of the factors is estimated. The simulation settings and scenarios are discussed in the next chapter.

3. SIMULATION DESIGN AND ANALYSIS

(1) Simulation settings and calibration

For calibration of the network, only the travel time was calibrated, and this was based on a study section in Nagoya City, where travel time and flow data were collected along a 600-metre section with 4 signalised intersections. The calibration procedure outlined by Park and Qi[8] was followed, and by changing *only* the desired speed and “waiting time before diffusion” parameters in VISSIM, the field observed travel time was reproduced in the VISSIM network.

(2) Scenario design

Fig.1 shows the hypothesised arterial networks. Each is 3 km long, but the link lengths become shorter from case 1 to case 3. The signalised arterials were built in VISSIM 7, with 2 lanes (3.0m) per travel direction, two signalised intersections at each end, with one shared left and through lane, one exclusive through lane, and a right-turn bay at each approach.

At each intersection, the East-West and North-South approaches all had the same approach volume, and the same turning ratios of 10:80:10 for the Left Turn : Through : Right Turn movements. Four phase control was applied at the intersections, with a permitted RT movement.

For the hypothesised arterials in this study, the following influencing factors were considered: Mean desired speed (45, 50, 54, 60 km/h), Cycle length (80, 90, 100, 108, 120, 150, 160 s), Flow (100 – 1900 veh/h), Offset type (simultaneous, and alternating preferential offsets).

The signal timings were designed based on the Japan Society of Traffic Engineers' Manual on Traffic Signal Control[9]. **Table 1** shows the signal green times for each phase for the cycle lengths considered

Five simulation runs were conducted for each scenario, each for 45 minutes (15 min warm-up time, 30 min for data collection). Individual vehicle travel times were collected starting *after* the vehicle passes the upstream stopline and *after* passing the downstream stopline. In this way, the signal impact was only considered at the downstream intersections. The average travel time of vehicles crossing *both* the upstream and downstream signal stoplines were obtained, and the average travel speed calculated.

4. FINDINGS FROM SIMULATION

The results from the simulation analysis – travel speed under each scenario are described in this chapter. The results are represented by *travel speed – flow curves*, and the impact of the investigated factors on travel speed are discussed. <u>*It should be noted that the term "flow" in travel speed – flow curve refers to the input demand.*</u>

Because the travel speed was computed for vehicles that traversed the entire arterial, the sample size was usually smaller for case 3 (with 8 links) as there was a higher chance of the vehicles from the most upstream intersection approach turning out of the arterial at one of the many downstream intersections. This introduced some variance in the Westbound Travel Speed and the Eastbound Travel Speed estimates. For the proceeding discussion, the travel speed was averaged over both travel directions.

Although four different desired speed distributions were considered, the impact of this factor is not presented in isolation in this paper. This is because the trends are similar for all desired speed distributions. Instead, the focus is on the link travel time which is a function of the link length and mean desired speed.

(1) Impact of flow

Fig.2 shows the relationship between travel speed and flow in all three cases when the arterial has a desired speed of 60km/h, all intersections have a cycle length of 90 s, and simultaneous offsets are used. The relationship between travel speed and flow is well documented, and it is understood that an increase in flow gradually leads to a reduction in the travel speed. This relationship was observed in this study. Two of the curves in **Fig.2** (the two longer links) show that the travel speed is fairly uniform at low flow conditions, after which it gradually starts to decrease.

Table 1 Traffic signal settings for all intersections

Movement		Signal phasing (sec)										Cycle length (sec)
		φ1		φ2			φ3		φ4			
		1	2	3	4	5	6	7	8	9	10	
E-W	Vehicle											
	Right-turning vehicle											
S-N	Vehicle											
	Right-turning vehicle											
All Key Intersections		26	3	5	3	3	26	3	5	3	3	80
		30	3	6	3	3	30	3	6	3	3	90
		35	3	6	3	3	35	3	6	3	3	100
		38	3	7	3	3	38	3	7	3	3	108
		44	3	7	3	3	44	3	7	3	3	120
		56	3	10	3	3	56	3	10	3	3	150
		60	3	11	3	3	60	3	11	3	3	160

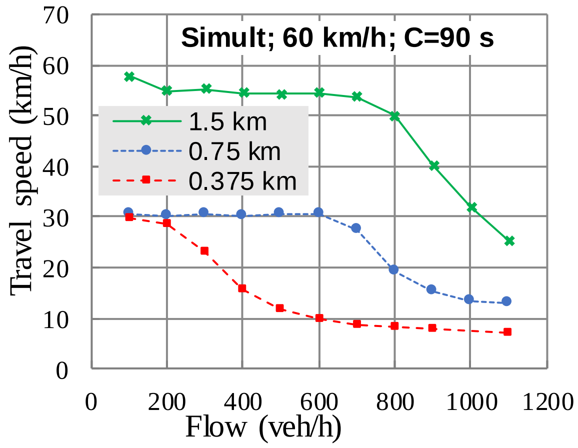

Fig.2 Simultaneous offsets, 60km/h, C = 90s

The decrease in travel speed with an increase in the flow arises due to increased vehicle interactions, where vehicles are likely to be stuck behind slower vehicles or left-turning vehicles (in case of shared left-turn lanes), or they need to decelerate to allow overtaking vehicles enter their lane. The probability of these occurrences increases with flow, leading to longer travel times and lower travel speed.

Similar trends can be observed in **Fig.3** where the offset type is alternating preferential, although the reduction in travel speed is much sharper. The reason for this is to do with vehicle arrival patterns, which are discussed later.

(2) Impact of offset pattern

Fig.3 shows the travel speed – flow curves for a similar arterial as in **Fig.2**, except the offset pattern is alternating preferential. The clearest difference between these figures is case 2 with link lengths of 0.75km. Until a flow of 600 veh/h, a high travel speed (55km/h) can be maintained using alternating preferential offsets, yet when simultaneous offsets are used a much lower travel speed (30km/h) is obtained.

The cause for this discrepancy has to do with the downstream signal indication at the time of vehicle arrivals. In this instance, alternating preferential offsets ensure at least one direction is prioritised, but simply setting simultaneous offsets results in vehicles consistently arriving downstream during a red signal indication and having to stop at practically every intersection along the arterial.

However, it should be noted that this effect is not fixed based on the offset type alone, but rather also depends on the link travel time and the cycle length.

(3) Impact of cycle length

Cycle lengths of 80 – 160 seconds were investigated in this study. Results showed that *generally*, higher cycle lengths led to lower travel speeds. **Fig.4** shows the travel speed along arterial with average link length 1.5km, desired speed of 60 km/h, alternating preferential offsets, and only 4 of the analysed cycle lengths. The figure shows that at lower flows, the longer cycle lengths tend to lead to lower travel speed, while shorter cycle lengths lead to higher travel speed.

While this is certainly the case in **Fig.4**, and in many of the cases studied, the relationship between travel speed and cycle length is not linear. Longer cycle lengths can lead to higher travel speed if the coordination conditions are met.

(4) Impact of link length

It can be seen in **Fig.3** that when vehicle demand exceeds 600 veh/h, there is a sharper reduction in travel speed on the arterial with 0.75km links compared to that with link lengths of 1.5km. This is attributed to the queue build up along the links, which limits the number of vehicles that can be discharged from the upstream intersections, effectively increasing the travel time and reducing the travel speed of all vehicles.
For the arterial with shorter links (0.375km), even at low flows, the travel speed is much lower, and the effects of queue build up occur at even lower flows.

(5) Combined effect off all factors

In **Fig.2** and **Fig.3**, it can be seen that at low flow, the travel speeds are similar for both offset types in the two link arterial case (1.5 km). At low flows (up

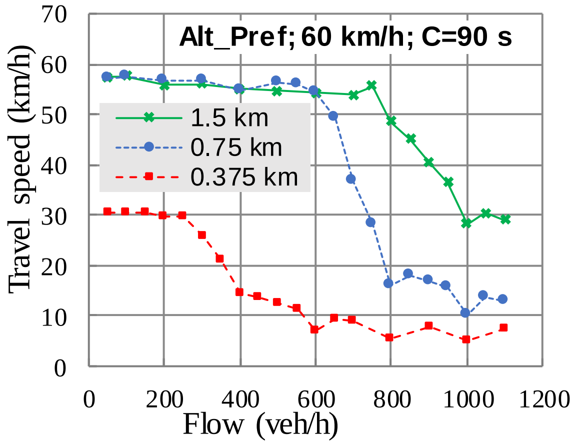

Fig.3 Alternating preferential offsets, 60km/h, C = 90s

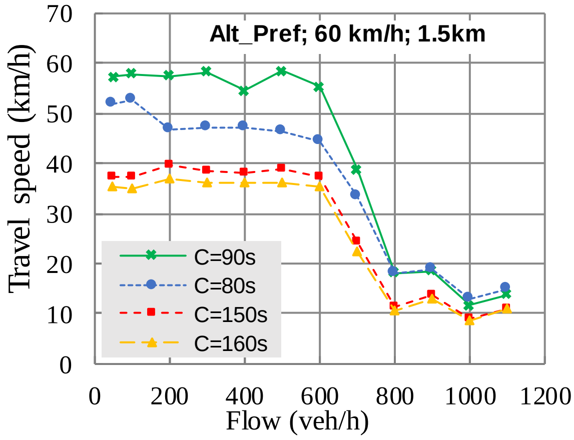

Fig.4 Alternating preferential offsets, 60km/h, 1.5km links

to 600 veh/h), both offset patterns lead to similar travel speeds for this case. This is because the cycle length is equal to the link travel time, and both offset patterns effectively function in the same way. If the cycle length were different from the link travel time, the simultaneous offset pattern would likely have lower travel speed as vehicles would not be guaranteed preferential offsets as is the case for the alternating preferential offsets.

However, in case 2, where the link length is 0.75km, the offset pattern selected has a significant impact. When the link length is 0.75km, **Fig.2** shows a drastic reduction in travel speed compared to the 1.5km case. This is because with simultaneous offsets, all vehicles leaving the upstream signal arrive at the downstream when the signal is red (*depending in the link travel time and the cycle length*), all through the arterial. However, this issue does not occur when alternating preferential offsets are used since each alternating direction is given priority. This can be observed in **Fig.3**.

The results discussed in this section show that the impact of link length on travel speed is also related to the link travel time (link length divided by the free-flow speed), as well as the cycle length. Although the impacts of these factors individually can be approximated, their interrelationships also merit consideration.

5. DISCUSSION

This study is motivated by the need to propose a methodology of estimating the expected performance of planned signalised arterials, in terms of the travel speed.

It is common in some countries' practice that the focus is on the intersection capacity for the planning and design of roadways, without due consideration of the quality of traffic flow that can be expected. In order to contribute to the performance-oriented road planning for signalised arterials, this study aimed to identify how the traffic signals in combination with other factors affect the quality of traffic flow.

In this paper, a review of some travel time estimation models was conducted from which a list of factors with an influence on the travel time (and travel speed) were identified. Based on the identified factors, several scenarios were designed and analysed by traffic simulation to study their impact on arterial travel speed.

The results obtained from this study were useful for confirming the factors that have an influence on travel speed.

For planning purposes, the development of travel speed vs. flow (actual outflow from the arterial) curves is necessary so as to show the number of vehicles that can be accommodated by the network, and the travel speed that can be achieved.

These curves are planned to be presented at the opportunity of the conference.

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