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Queue Discharge Flow and Speed Models for Signalised Intersections

Rahmi Akçelik and Mark Besley
Akcelik and Associates Pty Ltd, Melbourne, Australia

INTRODUCTION

Traditional traffic theory focuses on modelling of queue discharge *flow rates* (or headways) at signalised intersections with relatively little consideration of the corresponding queue discharge *speed* characteristics. A simplified queue discharge flow model is commonly employed for estimating capacity and various performance statistics (delay, queue length, etc). This model uses a constant saturation flow rate and converts the displayed green time to an effective green time using start loss and end gain time parameters associated with the saturation flow rate (Webster and Cobbe 1966, Akçelik 1981, Teply, et al. 1995, TRB 2000).

Akçelik, Besley and Roper (1999) described exponential queue discharge flow rate and speed models that represent queue discharge behaviour without resorting to various simplifying assumptions needed to derive saturation flows and effective green times. Modelling of queue discharge speed, in addition to the queue discharge flow rate (or headway), makes it possible to derive relationships for traffic parameters such as vehicle spacing, gap length, density, time and space occupancy ratios, gap time, occupancy time, space time as well as vehicle acceleration characteristics. Thus, a complete set of fundamental relationships for queue discharge behaviour at traffic signals is obtained. These relationships are particularly useful for adaptive control purposes (Akçelik 1997, AUSTROADS 1993, Lowrie 1996), and important in relation to microsimulation modelling (Akçelik and Besley 2001a).

Akçelik, et al (1999) discussed the use of the exponential queue discharge flow rate model to derive saturation flow rates, start loss and end gain times that correspond to various traditional saturation flow measurement methods, e.g. the methods described in Akçelik (1981) and TRB (2000). Analysis of uninterrupted flow conditions at signalised intersections, as relevant to conditions after queue clearance (unsaturated part of the green period), was also carried out. This indicated that speed - flow - density models for these conditions are consistent with general models for uninterrupted conditions. These models complement the exponential queue discharge models for saturated conditions, thus allowing for a complete analysis of traffic flow conditions at signalised intersections.

This paper describes the exponential queue discharge flow rate and speed models, and presents a summary of model calibration results based on data from 18 intersections in Melbourne and Sydney, Australia. Relationships among saturation headway and speed, jam spacing, queue departure response time, queue departure wave speed and acceleration characteristics are presented. Implications of the queue discharge flow and speed models for adaptive signal control practice, namely the SCATS control parameters, optimum detector loop (detection zone) length and the gap setting parameter are discussed.

QUEUE DISCHARGE MODELS

Exponential functions of queue discharge flow rate and speed were found to provide the most useful model for representing queue discharge characteristics at a signalised intersection as depicted in *Figure 1*. Bonneson (1992a,b) developed queue discharge headway and speed models that calculated the departure headway and speed as a function of the *vehicle position in queue*. The following models developed by Akçelik, et al (1999) express the queue discharge speed, flow rate and headway as a function of the *time since the start of green*:

$$v_s = v_n \left[1 - e^{-m_v (t - t_r)}\right] \tag{1}$$

$$q_s = q_n \left[1 - e^{-m_q (t - t_r)}\right] \tag{2}$$

$$h_s = h_n / \left[1 - e^{-m_q (t - t_r)}\right] \tag{3}$$

where

t = time since the start of the displayed green period (seconds),

t_r = start response time (a constant value) related to an average driver reaction time for the first vehicle to start moving at the start of the displayed green period (seconds),

v_s = queue discharge speed at time t (km/h),

v_n = maximum queue discharge speed (km/h),

m_v = a parameter in the queue discharge speed model,

q_s = queue discharge flow rate at time t (veh/h),

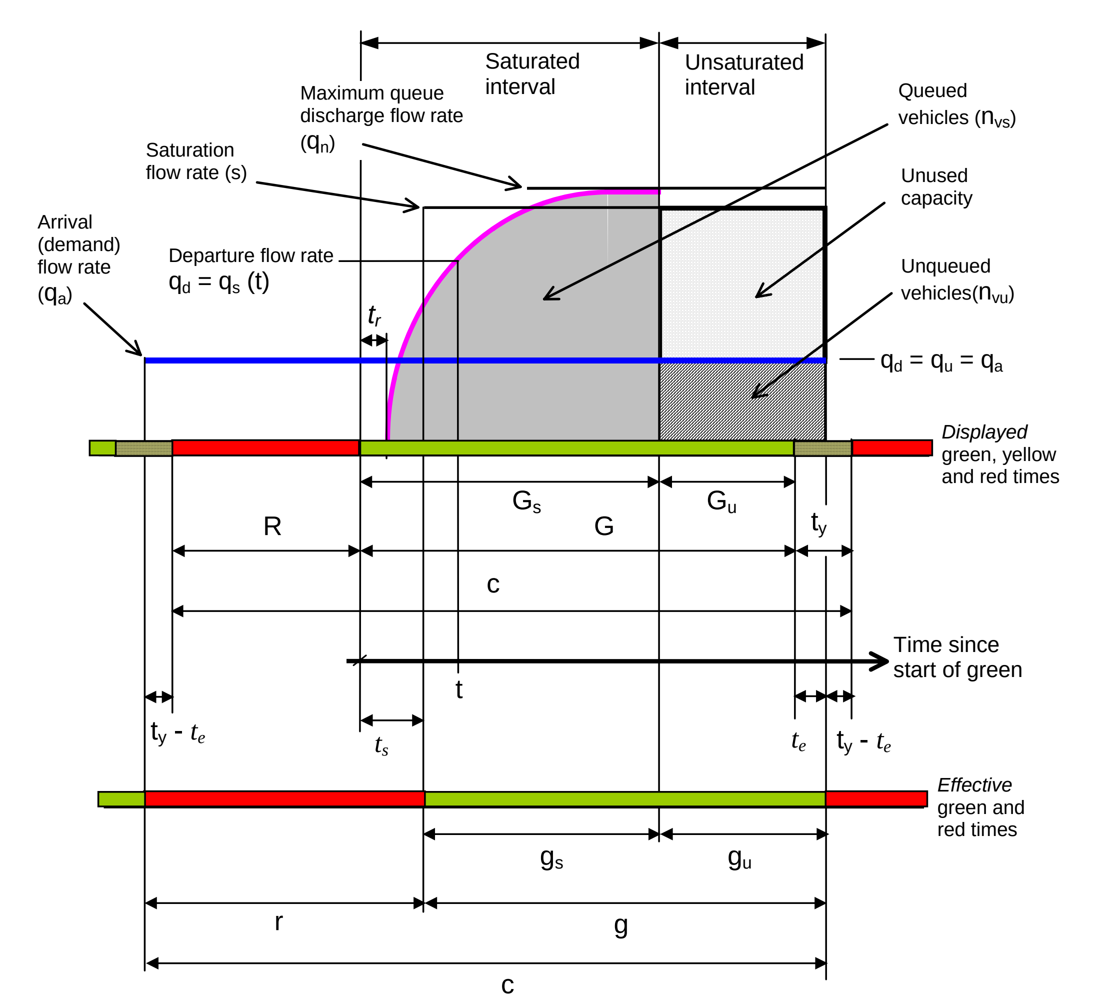
q_n = maximum queue discharge flow rate (veh/h),

m_q = a parameter in the queue discharge flow rate model,

h_s = queue discharge headway at time t (seconds), $h_s = 3600 / q_s$, and

h_n = minimum queue discharge headway (seconds), $h_n = 3600 / q_n$.

In *Equations (1) to (3)*, parameters v_n, q_n and h_n are for the actual traffic mix including heavy vehicles although calibration results given in this paper are for light vehicles (mainly cars) only. In *Figure 1*, q_a and q_d represent the arrival and departure flow rates, respectively. The departure flow rate is $q_d = q_s$ (t) during the *saturated* part of the green period, and $q_d = q_u = q_a$ during the *unsaturated* part of the green period. In the case of platooned arrivals, the arrival flow rates are different during the red and green periods (Akçelik 1995).



c	cycle time ($c = R + G + t_y = r + g$)
G, R	displayed green and red times
G_s, G_u	saturated and unsaturated parts of the displayed green period ($G = G_s + G_u$)
g, r	effective green and red times
g_s, g_u	saturated and unsaturated parts of the effective green period ($g = g_s + g_u$)
t_y	yellow time
t_s, t_e	start loss and end gain times

Figure 1 - Departures during the saturated and unsaturated portions of the green period with the exponential queue discharge model and saturation flow approximation shown

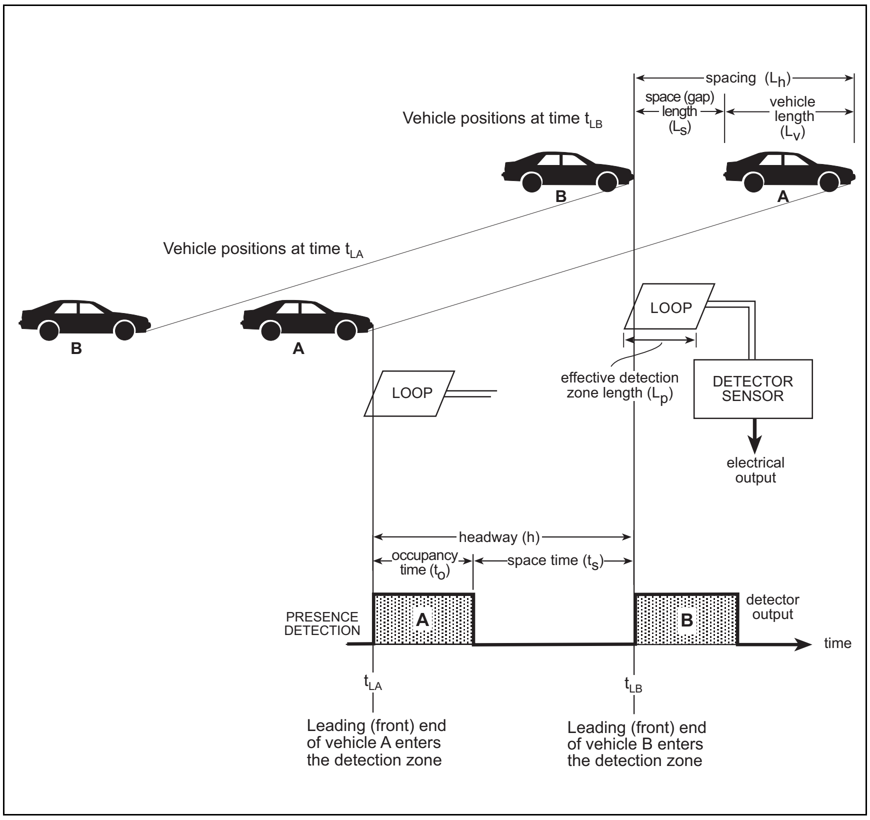


Figure 2 - Definition of headway, spacing, occupancy time and space time parameters

The vehicle spacing, L_{hs} (m/veh), occupancy time, t_{os} (seconds), and space time, t_{ss} (seconds) and the cumulative queue discharge flow, n_s (vehicles), at time t during queue discharge can be determined from:

$$L_{hs} = v_s h_s / 3.6 = 1000 v_s / q_s \quad (4)$$

$$t_{os} = 3.6 (L_p + L_v) / v_s \quad (5)$$

$$t_{ss} = h_s - t_{os} \quad (6)$$

$$n_s = \int_{t_r}^{t} \frac{q_s}{3600} dt \qquad \textit{for } t > t_r \quad (7)$$

$$= \frac{q_n}{3600} \left[(t - t_r) - \frac{1 - e^{-m_q (t - t_r)}}{m_q} \right]$$

where v_s (km/h), h_s and h_n (seconds), q_s and q_n (veh/h) are as in *Equations (1) to (3)*, L_p is the detection zone length (m) and L_v is the average vehicle length (m).

The vehicle spacing, L_{hn}, occupancy time, t_{on} and space time, t_{sn} at maximum queue discharge flow rate (or minimum queue discharge headway) can be calculated from *Equations (4) to (6)* by replacing v_s, h_s and q_s by v_n, h_n and q_n, respectively.

General definitions of headway (h), spacing (L_h), occupancy and space time (t_o, t_s) parameters with presence detection are shown in *Figure 2* (based on measurements from the *front* of the leading vehicle to the *front* of the following vehicle).

Applying the boundary condition that speed is zero, $v_s = 0$, when the vehicle spacing during queue discharge equals the jam spacing, $L_{hs} = L_{hj}$, the parameters m_v and m_q are related through:

$$m_q = m_v \frac{L_{hn}}{L_{hj}} = 1000\ m_v \frac{v_n}{q_n\ L_{hj}} \tag{8}$$

where L_{hj} is the average jam spacing, i.e. the spacing between vehicles in the queue (m/veh), and L_{hn} is the average spacing (m/veh) at the minimum queue discharge headway, or maximum queue discharge flow rate ($L_{hn} = v_n\ h_n / 3.6 = 1000\ v_n / q_n$). Average jam spacing is the sum of average gap (space) length and average vehicle length for vehicles in a stationary queue ($L_{hj} = L_{sj} + L_V$).

MODEL CALIBRATION RESULTS

Calibration results for the exponential queue discharge flow and speed models based on light vehicle data from 18 intersections in Melbourne and Sydney are summarised in *Table 1*. Calibration results given in *Table 1* are based on t_r (start response time) = 0. The data collection and analysis methods, and survey site characteristics are described, and alternative calibration methods including the use of $t_r > 0$ are discussed in detail in Akçelik, et al (1999).

Table 1 gives the queue discharge model parameters for individual sites as well as the average values for arrow-controlled (protected) right-turn (left-turn for driving on the right-hand side of the road) isolated sites, through isolated sites and paired intersection sites. The term *isolated* used in this context means a single intersection site with a reasonably long distance to the downstream intersection as opposed to a *paired* (closely-spaced) intersection site, and does not relate to signal coordination. In addition to the parameters that appear in *Equations (1) to (8)*, the free-flow speed determined as the approach speed limit (v_f) and the ratio of the queue discharge speed to the free-flow speed (v_n/v_f) are given in *Table 1*.

Average site values for parameters q_n, m_q, L_{hn}, t_{sn}, L_{sj} and v_n/v_f in *Table 1* are values calculated using relevant average site parameters according to the relationships expressed in the previous section. Other parameters (v_n, m_v, L_{hj}, v_f) in *Table 1* are simple arithmetical averages of individual site values in each group.

The results given in *Table 1* show that queue discharge characteristics for through and right-turn movements differ significantly although maximum queue discharge flow rates (q_n) are found to be similar. Lower jam spacing (L_{hj}), shorter response time (t_x) and lower maximum queue discharge speeds (v_n) at right-turn sites help to achieve low queue discharge headways, therefore high maximum flow rates. The vehicle spacing (L_{hn}) and space time (t_{sn}) values at maximum queue discharge flow are also seen to be lower for right-turn sites.

Table 1 - Queue discharge model parameters for 18 intersections in Melbourne and Sydney

Site	v_n (km/h)	q_n (veh/h)	m_v	m_q	h_n (s)	L_{hn} (m)	t_{sn} *(1)* (s)	L_{hj} (m)	L_{sj} *(1)* (m)	v_f (km/h)	v_n/v_f
Average site values (q_n, m_q, L_{hn}, t_{sn}, L_{sj}, v_n/v_f are values calculated using relevant average site parameters)											
Right-turn (isolated) sites *(2)*	24.5	2033	0.307	0.582	1.771	12.0	0.46	6.4	2.0	65	0.38
Through (isolated) sites	45.1	2086	0.118	0.369	1.725	21.6	1.02	6.9	2.5	69	0.65
Through (paired intersection) sites	30.9	1958	0.244	0.550	1.839	15.8	0.80	7.0 *(3)*	2.6	67	0.46
Right-turn (isolated) sites *(2)*											
Sydney 1	24.7	2098	0.317	0.621	1.716	11.8	0.42	6.0	1.6	60	0.41
Sydney 2	21.7	1966	0.373	0.698	1.831	11.0	0.35	5.9	1.5	60	0.36
Melbourne 1	24.4	1948	0.287	0.545	1.848	12.5	0.53	6.6	2.2	70	0.35
Melbourne 2	27.1	2133	0.252	0.464	1.687	12.7	0.51	6.9	2.5	70	0.39
Through (isolated) sites											
Sydney 3	39.5	1790	0.103	0.334	2.011	22.1	1.20	6.8	2.4	60	0.66
Sydney 4	33.2	1801	0.200	0.542	1.999	18.4	1.03	6.8	2.4	60	0.55
Sydney 5	**52.8**	**2283**	**0.078**	**0.273**	**1.577**	**23.1**	**0.97**	**6.6**	**2.2**	**70**	**0.75**
Melbourne 3	31.7	1892	0.150	0.359	1.903	16.8	0.89	7.0	2.6	60	0.53
Melbourne 4	36.4	1938	0.135	0.347	1.857	18.8	0.98	7.3	2.9	60	0.61
Melbourne 5	46.4	1999	0.102	0.343	1.801	23.2	1.11	6.9	2.5	70	0.66
Melbourne 6	53.8	2422	0.118	0.374	1.486	22.2	0.89	7.0 *(3)*	2.6	80	0.67
Melbourne 7	56.1	2558	0.104	0.326	1.407	21.9	0.84	7.0 *(3)*	2.6	80	0.70
Melbourne 8	46.2	2423	0.126	0.343	1.485	19.1	0.79	7.0 *(3)*	2.6	80	0.58
Melbourne 9	47.9	2217	0.089	0.275	1.624	21.6	0.95	7.0 *(3)*	2.6	60	0.80
Melbourne 10	52.4	1968	0.097	0.369	1.830	26.6	1.22	7.0 *(3)*	2.6	80	0.66
Through (closely-spaced intersection) sites											
Melbourne 11	30.9	1982	0.233	0.519	1.816	15.6	0.78	7.0 *(3)*	2.6	60	0.52
Melbourne 12	27.1	1804	0.310	0.665	1.995	15.0	0.81	7.0 *(3)*	2.6	60	0.45
Melbourne 13	34.6	2112	0.190	0.445	1.705	16.4	0.78	7.0 *(3)*	2.6	80	0.43

All parameters in this table are for light vehicles (cars) only.

(1) For t_{sn} and L_{sj}: Average vehicle length, L_v = 4.4 m, Detection zone length, L_p = 4.5 m.

(2) Left-turn for driving on the right-hand side of the road.

(3) Nominal values (jam spacing was not measured during these early surveys except at Melbourne 10 site where L_{hj} = 7.2 m was measured).

Sydney 3 : 9 per cent uphill grade

Melbourne 3 : Shared through and left turn (15 per cent left turn)

Melbourne 4 : 6 per cent uphill grade

The survey results presented in Akçelik, et al (1999) did not indicate any decrease in the maximum flow rate during the green period (with no downstream interference), even in the case of long green times (e.g. Teply, et al 1995). As an example, this is seen in *Figures 3 and 4* for the Sydney 5 site that had a very long green period (120 s) during the morning peak traffic period (data was collected for lane 2 of three through lanes, lane width was 3.4 m, and distance to downstream signals was 2700 m).

Table 1 indicates that the maximum queue discharge speed (v_n) is around 0.4 of the free-flow speed (v_f) for arrow-controlled (protected) right-turn movements, and in the range 0.4 to 0.8 of the free-flow speed for through movements (average 0.6).

Queue discharge model parameters for closely-spaced (paired) intersection sites are found to be significantly different from those for isolated through traffic sites. Even under conditions without downstream queue interference, maximum queue departure flow rates, speeds at maximum flow and spacings at maximum flow are lower, and queue discharge flow and speed models indicate quicker achievement of maximum flow and speed conditions (higher m_q and m_v values).

Analysis of conditions with downstream queue interference showed that the downstream queue interference occurred only when the distance to the back of queue was very small. For all practical purposes, it was concluded that queue interaction occurred when the downstream queue storage space was fully occupied (Akçelik, et al 1999).

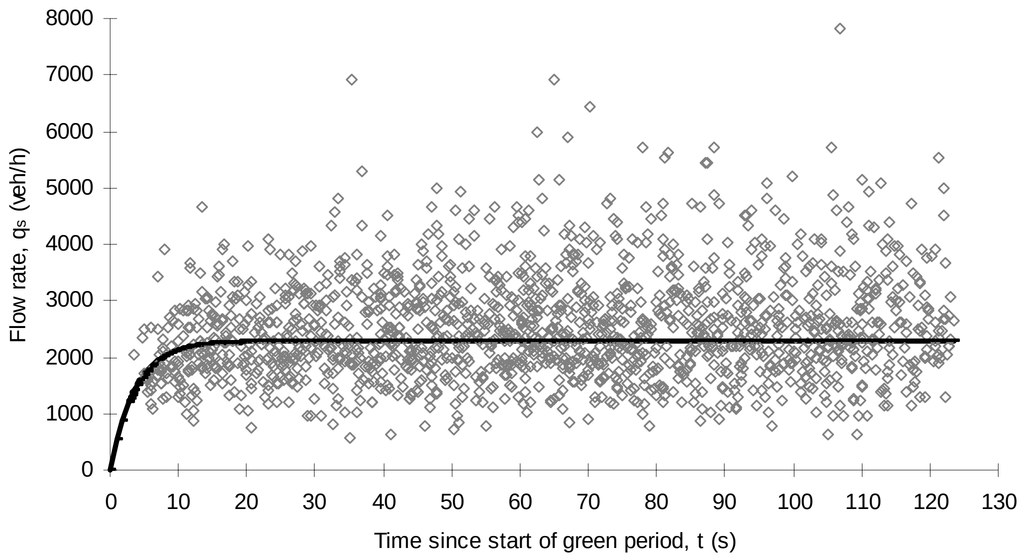


Figure 3 - Queue discharge flow rates observed at the intersection of General Holmes Drive and Bestic Street in Sydney (Through Isolated site - Sydney 5 in Table 1)

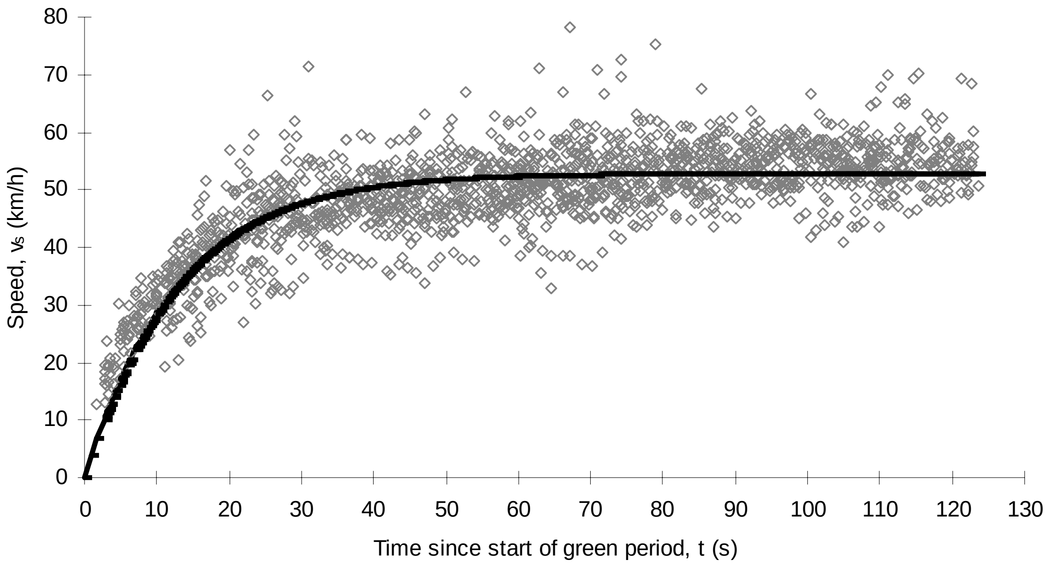


Figure 4 - Queue discharge speeds observed at the intersection of General Holmes Drive and Bestic Street in Sydney (Through Isolated site - Sydney 5 in Table 1)

QUEUE DEPARTURE WAVE SPEED AND ACCELERATION CHARACTERISTICS

An important fundamental relationship representing queue discharge characteristics is that the maximum queue discharge rate (q_n) is determined by the jam spacing (L_{hj}), queue departure response time for the next vehicle in the queue to start moving (t_x) and the maximum queue discharge speed (v_n). For the purpose of this discussion:

(i) the maximum queue discharge rate will be replaced by the traditional *saturation flow rate* (s), which can be approximated by the maximum queue discharge flow rate ($s \approx q_n$) or determined using one of the traditional saturation flow measurement methods, e.g. the methods described in Akçelik (1981) and TRB (2000), and

(ii) the maximum queue discharge speed will be referred to as the *saturation speed* ($v_S = v_n$).

As shown in *Figure 5*, the *saturation headway* ($h_S = 3600 / s$) can be expressed as follows:

$$h_S = t_x + 3.6\, L_{hj} / v_S \qquad (9)$$

where h_S and t_x are in seconds, L_{hj} is in metres and v_S is in km/h.

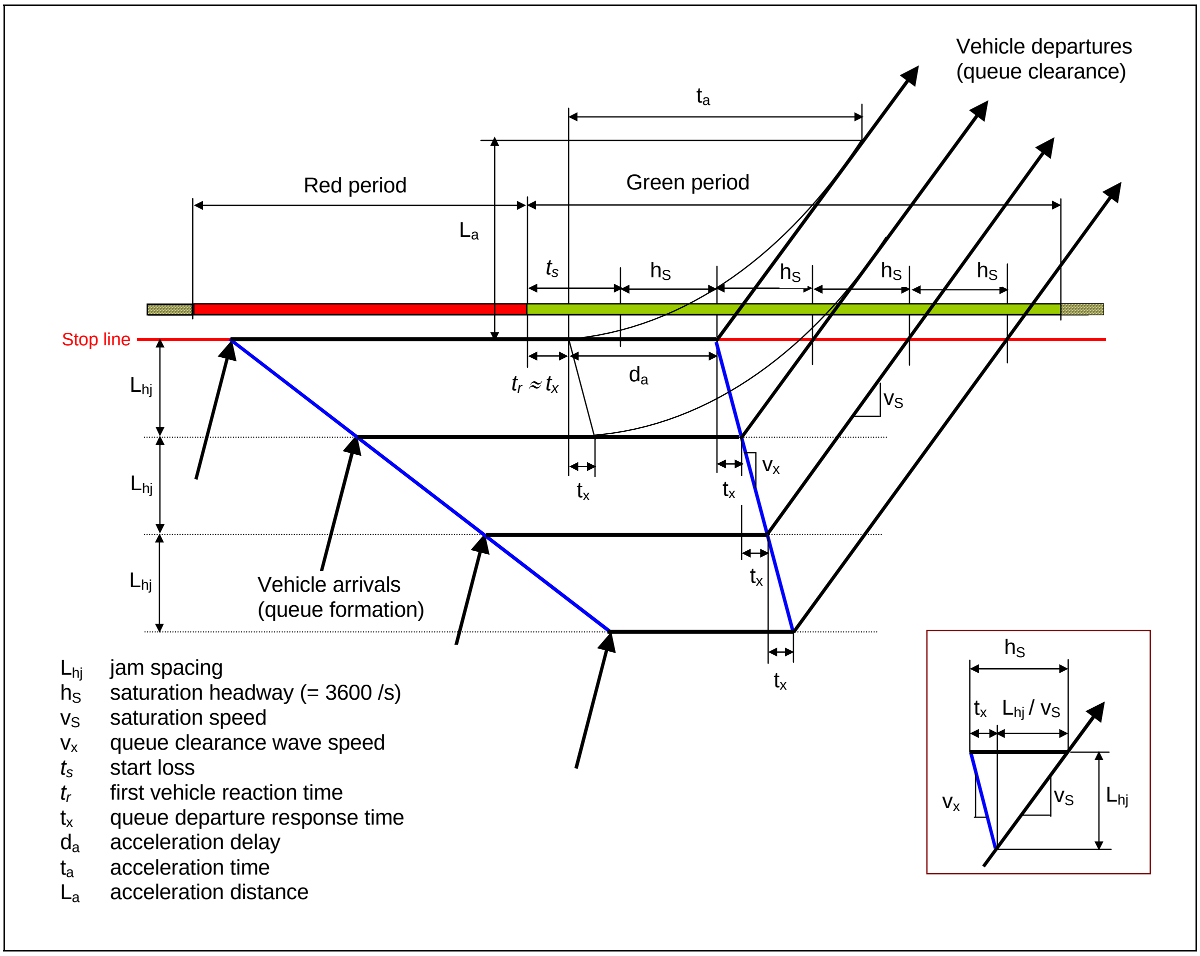


Figure 5 - The relationship between the saturation headway, maximum queue discharge speed, jam spacing, queue departure response time, queue departure wave speed and acceleration characteristics

Equation (9) can be used to determine saturation flow rates and pcu factors for various vehicles when the jam spacing and queue discharge speed are known. Where h_S, v_S and L_{hj} are known, the queue departure response time can be calculated from:

$$t_x = h_S - 3.6\ L_{hj} / v_S \qquad (10)$$

Other parameters shown in *Figure 5* are t_s (start loss for calculating an effective green time), t_r (departure response time of the first vehicle in the queue), d_a (average acceleration delay), and v_x (queue clearance wave speed). The relationships for parameters d_a and v_x are (assuming the departure response time of the first vehicle in the queue is $t_r = t_x$):

$$d_a = t_s + h_S - t_r = t_s + h_S - t_x = t_s + 3.6\ L_{hj} / v_S \qquad (11)$$

$$v_x = 3.6\ L_{hj} / t_x = 3.6\ L_{hj} / (h_S - 3.6\ L_{hj} / v_S) \qquad (12)$$

where t_r, d_a, t_s and t_x are in seconds, L_{hj} is in metres, v_x and v_S are in km/h.

The queue departure wave and acceleration parameters for survey sites are summarised in *Table 2*. Saturation flow rate (s) and start loss (t_s) values (for individual sites as well as average sites) in *Table 2* are based on the average saturation headway (h_S = 3600/s) excluding the vehicles departing during the first 10 seconds (approximately first 4 vehicles) based on the method described in Akçelik (1981), and the end gain (t_e) is determined assuming n_e =1.5 vehicles departing after the end of the green period ($t_e = 3600\ n_e / s$). The method for determining the start loss (t_s) parameter is discussed in detail in Akçelik, et al (1999). The saturation flow rates are seen to be very close to the maximum queue discharge flow rate values given in *Table 1* ($s \approx q_n$).

Average site values for all parameters except L_{hj} in *Table 2* are values calculated using relevant average site parameters as discussed in this section. Parameter L_{hj} is a simple arithmetical average of individual site L_{hj} values in each group.

Information about the acceleration delay is useful in determining parameters for relevant acceleration manoeuvres. Acceleration time (t_a in seconds), distance (L_a in metres) and average and maximum acceleration rates (a_a and a_m in m/s^2) given in *Table 2* were calculated using the polynomial acceleration model described by Akçelik and Biggs (1987) in the form adopted in the aaSIDRA software (Akçelik and Besley 2001b, Akcelik and Associates 2002). For vehicles accelerating from zero initial speed to the saturation speed (v_S):

$$d_a = t_a - 3.6\ L_a / v_S \qquad (13)$$

$$t_a = v_S / (3.6\ a_a) \qquad (14)$$

$$L_a = m_a\ v_S\ t_a / 3.6 = m_a\ v_S^2 / (12.96\ a_a) \qquad (15)$$

$$d_a = (1 - m_a)\ v_S / (3.6\ a_a) \qquad (16)$$

where m_a is an acceleration model parameter, $m_a = v_a / v_S$, i.e. the ratio of the average speed during acceleration ($v_a = L_a / t_a$) to the final speed (v_S), and is estimated from:

$$m_a = 0.467 + 0.002\ v_S \qquad \textit{subject to } m_a \le 0.700 \qquad (17)$$

Thus, from *Equations (11) and (16)*, the average acceleration rate can be estimated from:

$$a_a = (1 - m_a)\ v_S / (3.6\ d_a) = (1 - m_a)\ v_S / (3.6\ t_s + 12.96\ L_{hj} / v_S) \qquad (18)$$

The acceleration time (t_a in seconds) and acceleration distance (L_a in metres) can be determined form *Equations (14) and (15)* using the average acceleration rate (a_a in m/s^2) in *Equation (18)*. Determination of the maximum acceleration rate (a_m in m/s^2) experienced during the acceleration manoeuvre requires a more complicated set of equations (Akçelik and Besley 2001b, Akçelik and Biggs 1987). The values shown in *Table 2* indicate that the average acceleration rate is about 60 per cent of the maximum acceleration rate ($a_a / a_m = 0.6$).

Figure 6 shows the acceleration rate, speed and distance profiles for queue discharge manoeuvres determined using the polynomial acceleration model for the site shown in *Figures 3 and 4.*

Table 2 - Queue departure wave and acceleration parameters for 18 intersections in Melbourne and Sydney

Site	s (veh/h)	h_S (s)	t_s (s)	t_e (s)	v_S (km/h)	L_{hj} (m)	t_x (s)	v_x (km/h)	d_a (s)	t_a (s)	L_a (m)	a_a (m/s^2)	a_m (m/s^2)
Average site values (all parameters except L_{hj} are values calculated using relevant average site parameters)													
Right-turn (isolated) sites *(1)*	2032	1.772	1.7	2.7	24.5	6.4	0.84	27.3	2.64	5.45	19	1.25	2.13
Through (isolated) sites	2083	1.728	2.6	2.6	45.1	6.9	1.17	21.3	3.19	7.20	50	1.74	3.01
Through (paired intersection) sites	1958	1.839	1.8	2.8	30.9	7.0 *(2)*	1.02	24.6	2.63	5.57	25	1.54	2.63
Right-turn (isolated) sites *(1)*													
Sydney 1	2096	1.717	1.6	2.6	24.7	6.0	0.84	25.6	2.48	5.12	18	1.34	2.29
Sydney 2	1965	1.832	1.4	2.7	21.7	5.9	0.85	24.9	2.41	4.92	15	1.23	2.09
Melbourne 1	1946	1.850	1.8	2.8	24.4	6.6	0.88	27.1	2.80	5.77	20	1.17	2.00
Melbourne 2	2130	1.690	2.1	2.5	27.1	6.9	0.77	32.1	3.04	6.35	25	1.18	2.02
Through (isolated) sites													
Sydney 3	1786	2.015	2.9	3.0	39.5	6.8	1.40	17.5	3.50	7.70	46	1.42	2.45
Sydney 4	1801	1.999	1.8	3.0	33.2	6.8	1.26	19.4	2.57	5.51	27	1.67	2.87
Sydney 5	**2278**	**1.580**	**3.4**	**2.4**	**52.8**	**6.6**	**1.13**	**21.0**	**3.86**	**9.04**	**76**	**1.62**	**2.83**
Melbourne 3	1885	1.909	2.7	2.9	31.7	7.0	1.11	22.6	3.48	7.42	35	1.19	2.03
Melbourne 4	1933	1.863	2.8	2.8	36.4	7.3	1.14	23.0	3.50	7.61	42	1.33	2.28
Melbourne 5	1995	1.805	2.8	2.7	46.4	6.9	1.27	19.6	3.34	7.60	55	1.70	2.93
Melbourne 6	2416	1.490	2.6	2.2	53.8	7.0 *(2)*	1.02	24.7	3.06	7.20	62	2.07	3.62
Melbourne 7	2549	1.412	2.9	2.1	56.1	7.0 *(2)*	0.96	26.2	3.38	8.03	72	1.94	3.39
Melbourne 8	2418	1.489	2.8	2.2	46.2	7.0 *(2)*	0.94	26.7	3.36	7.62	55	1.68	2.91
Melbourne 9	2208	1.630	3.4	2.4	47.9	7.0 *(2)*	1.10	22.8	3.91	8.94	67	1.49	2.58
Melbourne 10	1963	1.834	2.6	2.8	52.4	7.0 *(2)*	1.35	18.6	3.11	7.26	60	2.01	3.49
Through (closely-spaced intersection) sites													
Melbourne 11	1981	1.817	1.9	2.7	30.9	7.0 *(2)*	1.00	25.2	2.73	5.79	26	1.48	2.54
Melbourne 12	1804	1.996	1.5	3.0	27.1	7.0 *(2)*	1.07	23.6	2.43	5.08	20	1.48	2.53
Melbourne 13	2110	1.706	2.2	2.6	34.6	7.0 *(2)*	0.98	25.8	2.94	6.35	33	1.51	2.60

All parameters in this table are for light vehicles (cars) only.

(1) Left-turn for driving on the right-hand side of the road.

(2) Nominal values (jam spacing was not measured during these early surveys except at Melbourne 10 site where L_{hj} = 7.2 m was measured).

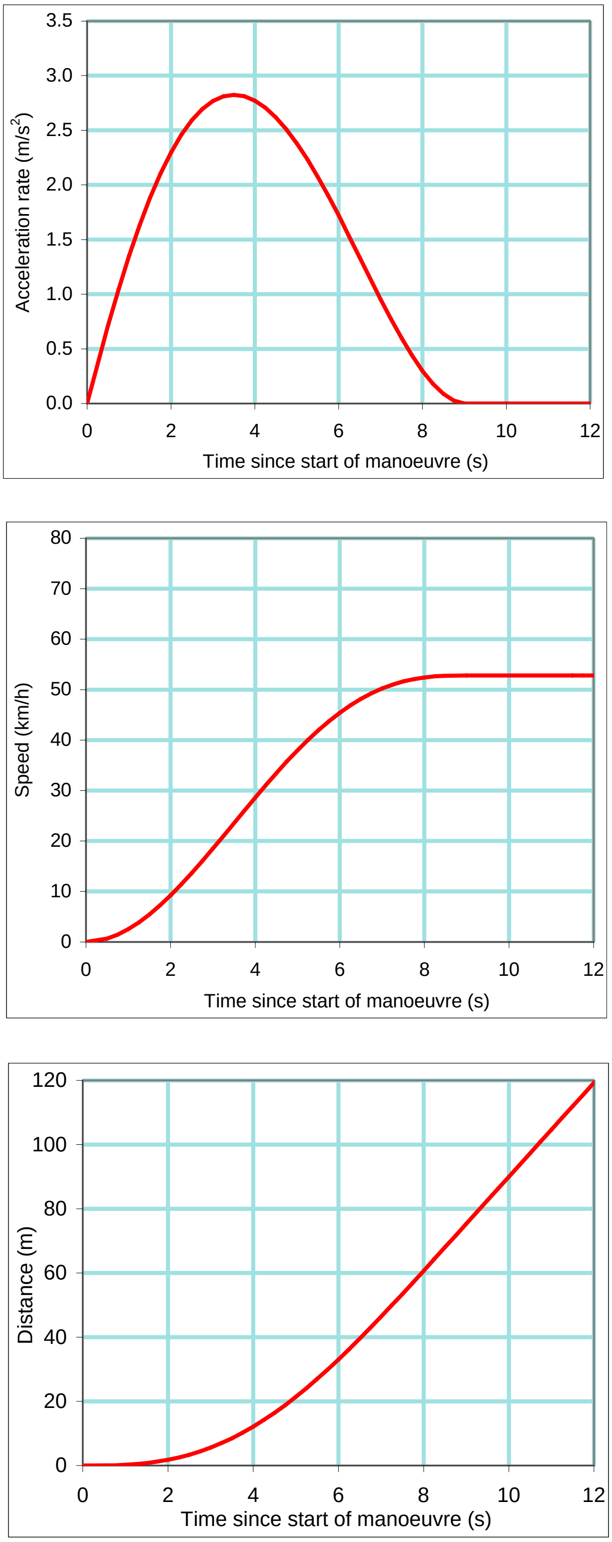


Figure 6 - Acceleration rate, speed and distance profiles for queue discharge manoeuvres determined for the site shown in Figures 3 and 4

IMPLICATIONS FOR ADAPTIVE SIGNAL CONTROL

The exponential queue discharge flow and speed models presented in previous sections allow analysis of various important issues for adaptive signal control, including SCATS (Akçelik 1997, Lowrie 1996) control parameters, optimum stop-line detector loop length and gap setting parameter for actuated signal control.

SCATS Control Parameters

The SCATS control parameters considered in Akçelik, et al (1999) include MF (maximum flow), HW (headway at maximum flow), KP (occupancy time at maximum flow), space time (HW – KP), and DS (degree of saturation). The SCATS method determines these parameters from on-line traffic measurements. Analyses indicated a reasonable match between these parameters and the corresponding analytical estimates. For example, the analytical parameter corresponding to the MF parameter, s_{MF} is given by:

$$s_{MF} = \frac{q_m \left[(G_{max} - t_r) - \dfrac{1 - e^{-m_q (G_{max} - t_r)}}{m_q} \right] + 3600\, n_e}{G_{max} + I_t} \qquad (19)$$

where q_m is the maximum queue discharge flow rate (per hour) for a traffic stream consisting of passenger car units (light vehicles) only, G_{max} is the maximum green time (seconds), I_t is the terminating intergreen time (seconds), n_e is the number of vehicles that depart during the terminating intergreen period, and parameters m_q and t_r are as in *Equation (2)*.

Figure 7 shows the correspondence between SCATS-reported MF values and the corresponding analytical parameter s_{MF} for Sydney 1 to 5 and Melbourne 1 to 6 sites listed in *Table 1*. The linear trendline indicates good correspondence given basic differences between the measurement and estimation methods.

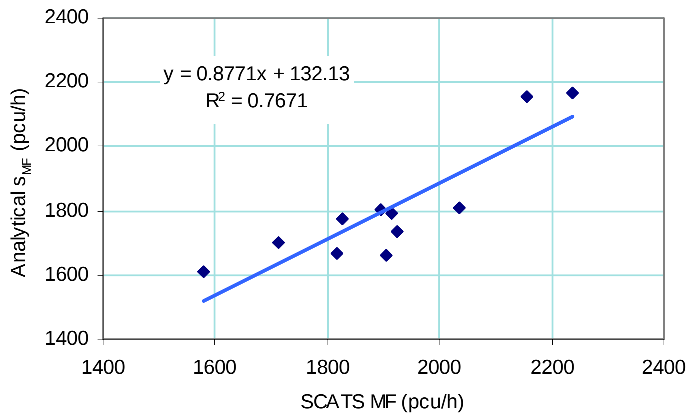


Figure 7 - Correspondence between SCATS-reported maximum flow parameter MF and the analytical parameter s_{MF} for Sydney and Melbourne sites

Optimum Detector Loop Length

An optimum detection zone length is sought in terms of the best ability to detect traffic variables relevant to adaptive control. Lowrie (1984) discussed the choice of loop length considering the relationships between vehicle speed, gap length and space time. AUSTROADS (1993) Guide to Traffic Signals discusses the relationship between density and space time as a function of the detection zone length for selecting an appropriate loop length. The results presented in this paper have been adopted to update the relationships in the forthcoming edition of this publication.

For the dual purpose of counting vehicles and measuring space time, desired characteristics of the space time variable for determining optimum loop length can be explained in terms of space time - speed relationship:

(i) non-zero space time values should be obtained at low speeds, and at the same time,

(ii) a single speed value should be obtained for each given space time.

Thus, the optimum length for a loop is one that is as short as possible but not so short as to result in a double valued space-time relationship. To determine the optimum loop length, a limiting (low) speed value (5 to 10 km/h) that gives zero space time ($t_s = 0$) is chosen. Space time - speed relationships for detection zone lengths in the range L_p = 0.5 m to 6.0 m for average arrow-controlled right-turn and through (isolated and closely-spaced) sites are shown in *Figures 8 to 10.* These figures represent both saturated (queue discharge) and unsaturated conditions during the green period.

Analysis of optimum loop length values for all survey sites indicate that the optimum detection zone length is independent of the maximum queue discharge speed, the maximum flow rate and the spacing at maximum flow, and clearly related to jam gap length (jam spacing less the average vehicle length, $L_{sj} = L_{hj} - L_v$). On the basis of a limiting speed of 5 km/h, the range of optimum loop length is 3.0 to 3.8 m for through traffic sites, with an average value of 3.4 m, and 2.2 to 3.1 m for right-turn traffic sites, with an average value of 2.6 m (the current loop length used with the SCATS system is 4.0 - 4.5 m).

The following exponential regression models (see *Figures 11 and 12*) can be used to determine the optimum loop length when more detailed analysis is not possible:

$$L_{p5} = 1.3\, e^{0.39\, L_{sj}} \tag{20}$$

$$L_{p10} = 1.9\, e^{0.33\, L_{sj}} \tag{21}$$

where L_{p5} and L_{p10} are the optimum detection zone lengths based on limiting speed values of 5 km/h and 10 km/h, respectively.

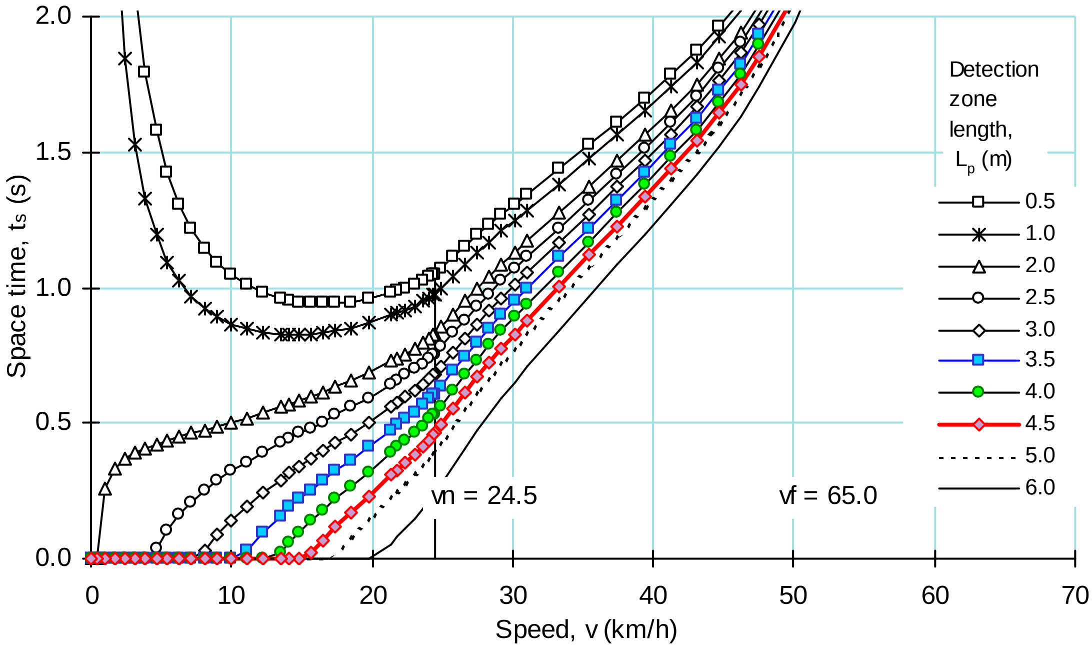


Figure 8 - Space time - speed relationships as a function of the detection zone length for an average right-turn (isolated) site

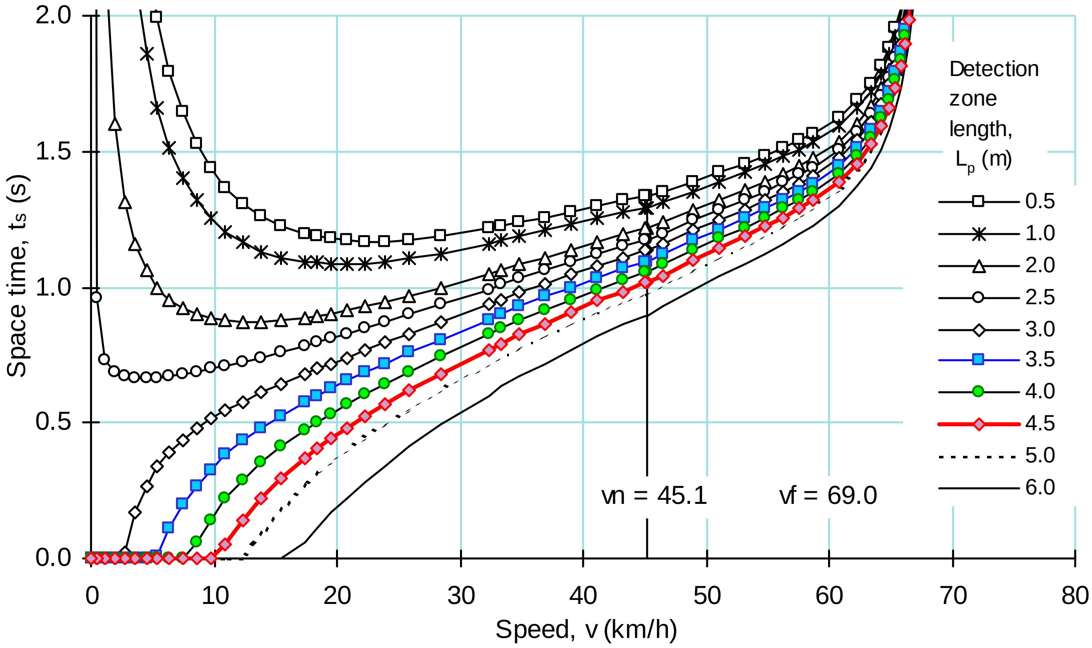


Figure 9 - Space time - speed relationships as a function of the detection zone length for an average through (isolated) site

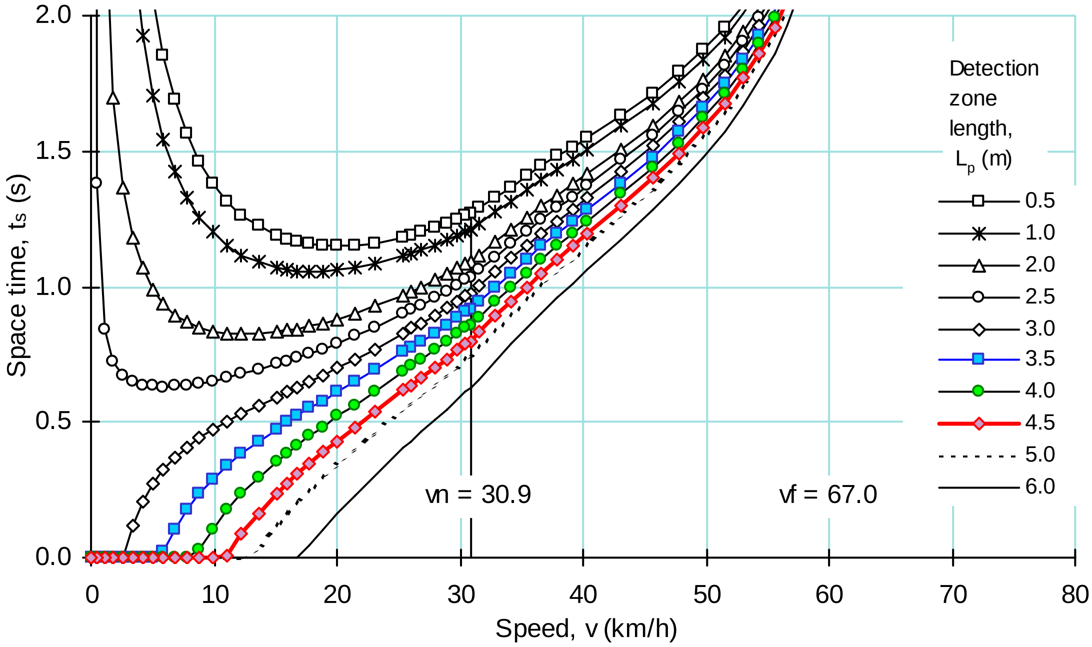


Figure 10 - Space time - speed relationships as a function of the detection zone length for an average through (closely-spaced intersection) site

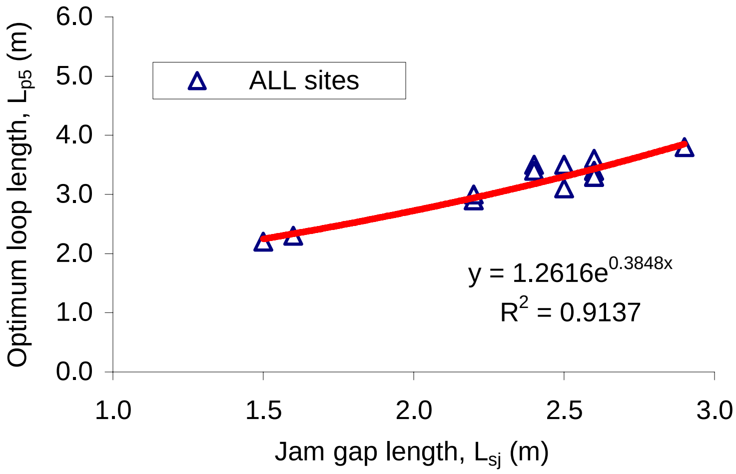


Figure 11 - Optimum loop length based on the chosen limiting speed value of 5 km/h, L_{p5} as a function of jam gap length, L_{sj}

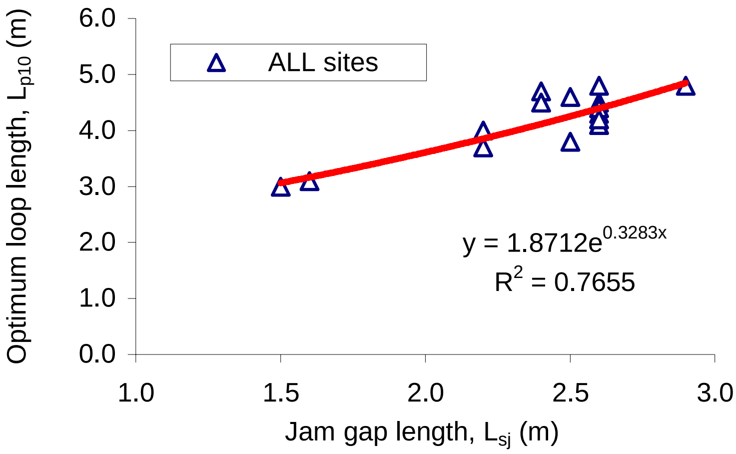


Figure 12 - Optimum loop length based on the chosen limiting speed value of 10 km/h, L_{p10} as a function of jam gap length, L_{sj}

Gap Setting Parameter

The gap setting (unit extension) parameter in actuated signal control with presence detection corresponds to a space time (headway time less occupancy time) as shown in *Figure 2*. To achieve small signal cycle times for efficient traffic control, the gap setting should be as small as possible while ensuring that gap change (green period termination due to measured space time exceeding the gap setting) does not occur during the saturated part of the green period. Therefore, the gap setting can be related to the space time at maximum queue discharge flow and speed conditions, t_{sn} (see *Table 1* for t_{sn} values for survey sites).

Considering the cycle-by-cycle variation of queue discharge parameters at a given site, a factor can be applied to the t_{sn} value to determine an appropriate gap setting value. The 98th percentile space time at maximum flow can be as high as twice the average value. Using a factor of 2.0, gap settings calculated as 2.0 t_{sn} for detection zone lengths of 3.0 to 4.5 m (using an average vehicle length of 4.4 m) are in the range 1.6 to 2.9 s for through traffic sites, and 0.7 to 1.5 s for right-turn traffic sites. These values are generally lower than the gap settings used in practice, e.g. AUSTROADS (1993) recommends 3.0 s for through traffic and 2.5 s for arrow-controlled right-turn movements. In practice, there may be various reasons for using longer gap settings. For example, a larger gap setting may be preferred for minor actuated movements at an intersection controlled by coordinated actuated signals in order to avoid undue gapping out of minor signal phases in order to preserve signal offsets that achieve best traffic progression.

CONCLUDING REMARKS

The exponential queue discharge flow and speed relationships described in this paper present a new realm of possibilities for more realistic traffic modelling instead of simpler modelling based on the use of constant saturation flow and effective green assumptions used to date. This has implications on traffic performance (delay, queue length, etc) estimation as well as signal timing optimisation (e.g. impact of short green periods vs long green periods). Modelling of queue discharge speeds in addition to queue discharge headways allows analysis of many aspects of adaptive traffic signal control as seen from the issues discussed.

Information provided in this paper is also useful towards investigating the queue discharge characteristics employed in microsimulation models (Akçelik and Besley 2001a). Queue discharge headway (or flow rate), speed and other queue discharge parameters could be observed in microsimulation in order to assess reasonableness and accuracy of the queuing, acceleration and car-following models used. It is recommended that queue discharge flow rate (headway) and speed patterns (in a form similar to *Figures 3 and 4*) and relevant parameter values (as summarised in *Tables 1 and 2*) for signalised intersections generated by microsimulation models are compared with the exponential models described in this paper.

An important aspect of queue departure patterns to be considered in evaluating microsimulation models, as indicated by the information presented in this paper, is that saturation speed, headway (time) and spacing (distance) between vehicles as they pass the stop line remain fairly constant during the saturated part of the green period (in other words, queued vehicles do not start accelerating to the cruise speed until after they clear the intersection).

Further surveys are recommended for model calibration, especially at paired (closely-spaced) intersections, CBD type intersection sites, and arrow-controlled right-turns including surveys at dual right-turn lane sites with a view to establishing differences in queue discharge parameters of the inside and outside lanes.

Analyses of heavy vehicle effects are needed, particularly with a view to the effect on jam spacing and jam gap length. Further research on acceleration characteristics at the signal stop line in relation to queue discharge model parameters is also recommended.

An investigation of the effect of the distance to the downstream intersection on queue discharge model parameters (with and without the effect of downstream queue interference) is recommended for paired intersection and CBD type intersection sites.

Surveys described in this report were carried out mostly during morning and afternoon peak periods. It would be useful to carry out additional surveys to investigate differences between peak and off-peak traffic characteristics. Similarly, conducting surveys at the same site under adverse light and weather conditions (dark, rainy) would be useful in order to determine the impact of such adverse conditions on queue discharge parameters.

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