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Reference:

AKÇELIK, R., and BESLEY M. (2001). Microsimulation and analytical methods for modelling urban traffic. Paper presented at the *Conference on Advance Modeling Techniques and Quality of Service in Highway Capacity Analysis, Truckee, California, USA, July 2001.*

October 2001

Microsimulation and analytical methods for modelling urban traffic

Rahmi Akçelik and Mark Besley

Since a theoretical calculation of delay is very complex and direct observation of delay on the road is complicated by uncontrollable variations, it was decided to use a method whereby the events on the road are reproduced in the laboratory by means of some machine which simulates behaviour of traffic …
Webster (1958) [1]

Simulation is a powerful tool, and like all powerful tools it can be dangerous in the wrong hands. The increased emphasis on simulation studies and the corresponding lack of experience on the part of some people who attempt to apply the method can lead to a type of pseudosimulation. Pitfalls exist in simulation as in every human attempt to abstract and idealize. Some rules to follow in avoiding these pitfalls are (1) no assumption should be made before its effects are clearly defined, (2) no variables should be combined into a working system unless each one is properly explained and its relationships to the other variables are set and understood, and (3) it must be remembered that simplification is desirable but oversimplification can be fatal. ***Drew (1968)*** [2]

It is paradoxical however that the development of more "natural" and embodied interfaces leads to "unnatural" adaptations or changes in the user. In the progressively tighter coupling of user to interface, the user evolves as a cyborg.
Bargiela (2000) [3]

1 INTRODUCTION

Traffic simulation techniques have been used since the early days of the development of traffic theory [1,2,4-7]. An example of the development and use of a microsimulation model for analysing the combined route control and signal control problem in the 1970s, which has been a popular ITS (Intelligent Transport System) research topic in more recent years, has been described by Akçelik and Maher [5]. The ever-increasing power of personal computers and search for ITS solutions to growing urban transport problems has led to the emergence of a number of microscopic simulation models as practical traffic analysis tools [6,7]. Pursula [6] suggests that "simulation is now an everyday tool for practitioners and researchers in all fields of the (traffic and transportation engineering) profession".

There is great potential for useful application of microsimulation models to the analysis of complex traffic problems in urban areas, alongside the analytical techniques that are in use. Microsimulation is useful due to increasing levels of system complexity and uncertainty involved in the operation of urban traffic networks. However, concerns are often expressed regarding misuse of microsimulation. Response to a survey of microsimulation model users was summarised as "microsimulation is useful but dangerous" [7].

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This paper focuses on compatibility between microsimulation methods and established analytical techniques that are used in traffic engineering, considered as a useful method of model verification. Several key components of traffic models are discussed, and various recommendations are made, with a view to improving the practical usefulness of microsimulation models. These include:

(i) the use of simulation for capacity analysis, including the dependence of capacity on demand flow rates;

(ii) modelling of queue discharge (saturation) flow rate, queue discharge speed and other queue discharge parameters at signalised intersections, and relating them to the general queuing, acceleration and car-following models used in microsimulation;

(iii) modelling of gap-acceptance situations at all types of traffic facilities, and

(iv) estimation of lane flows at intersection approaches, and relating this to lane changing models used in microsimulation.

The consistency of definitions and measurement methods for traffic performance variables such as delay (stopped, geometric, etc), queue length (cycle average and back of queue) and stops (effective stop rate and proportion stopped) is also discussed.

It is suggested that comparison of specific microsimulation and analytical model components is useful towards model benchmarking for evaluation of new and existing models. Towards this end, a simple signalised intersection case is specified in sufficient detail to enable assessment of two basic traffic model components, namely queue discharge flow rate and lane flow distribution. While there is a large number of factors that affect queue discharge characteristics at signalised intersections, the example is set only to test how alternative models allow for the turning vehicle, heavy vehicle and road grade effects.

A comprehensive literature review is outside the scope of this paper, and it is probable that some of the issues discussed here have been raised in previous publications. Some useful references are provided in the references list [2-9]. It is recognised that this paper includes some general statements regarding microsimulation models, and particular software packages may already offer some of the features recommended in this paper.

2 SIMULATION FOR CAPACITY ANALYSIS

Capacity is the most widely used concept in traffic engineering practice [10-13]. Analytical models are built on the use of this basic traffic parameter whereas microsimulation models generally ignore it. The reason is the perceived difficulty of measuring capacity in simulation. This is because it is usually considered that capacity can be measured under saturated conditions only.

Akçelik, Chung and Besley [13] discuss two methods for measuring capacity at intersections:

(i) measuring departure flow rates under saturated (continuous queuing) conditions, and

(ii) measuring departure flow rates during saturated portions of individual stop-go cycles (traffic signal or gap-acceptance cycles), and calculating capacity as the potential departure flow rate that would be achieved if all cycles were fully saturated under higher demand flow rates.

The use of method (i) in simulation would require increasing the demand flow rate artificially in order to create continuous queuing, and measuring the departure flow rate as the capacity. This would mean additional simulation runs. A more serious problem associated with this method is the dependence of capacity on demand flow rates. Capacity is defined as the maximum flow rate under *prevailing traffic conditions*. When demand flow rates are increased to create saturated conditions, the prevailing conditions are changed due to interactions among traffic streams. This is relevant to all types of intersection. For example, at roundabouts, the increased demand flow rate on one leg would affect the capacity of other legs due to increased circulating flows and directional flow effects, thus affecting the capacity of the subject traffic stream ultimately.

On the other hand, method (ii) allows the measurement of capacity when demand flow rates are below capacity. It can be used with ease without changing the current demand flow rates, and therefore without need for additional simulation runs. It is no different from how the capacity of signalised intersections is determined in practice. Akçelik et al [13] suggest that this method can be applied to gap-acceptance situations.

The discussion in *Section 3* is related to capacity of signalised intersections, and the discussion in *Section 4* is related to capacity in gap-acceptance situations (e.g. give-way and stop-sign control and unsignalised roundabouts).

3 QUEUE DISCHARGE FLOW RATES AND SPEEDS

While saturation flow and lost time for signalised intersections are the most widely used parameters in traffic engineering practice, and are employed by analytical models extensively, microsimulation models generally ignore them, as in the case of the capacity parameter. The reason in this case is the use of a different modelling paradigm, i.e. one based on queuing, acceleration and car-following behaviour of individual vehicles rather than one based on the use of saturation headways between vehicles observed at the stop line. It is interesting to note that "The simulated behaviour of queue formation and discharge at traffic signals was reviewed. Values for queue discharge lost times were questioned as to their validity. Concern was similarly expressed regarding the acceleration versus speed relationships …" in a workshop on simulation models in early 1980s (FHWA, page 72) [4].

In a major study of departure flow characteristics of traffic at signalised intersections, Akçelik, Besley and Roper [14] identified a maximum queue discharge (saturation) speed (v_s) that corresponds to the maximum queue discharge flow rate (s) observed at the signal stop line. This saturation speed was found to be around 0.4 of the approach speed limit for arrow-controlled (protected) right-turn movements (left-turn movements for driving on the right-hand side of the road), and in the range 0.4 to 0.8 of the approach speed limit for through movements.

Queue discharge speeds and headways observed at an isolated intersection site with a very long green period during morning peak traffic period are shown in *Figures 1 and 2* (based on the surveys described in Akçelik et al [14]). The data was collected for lane 2 of three through lanes, lane width was 3.4 m, and distance to downstream signals was 2700 m. For this site, the saturation flow rate was s = 2278 veh/h (saturation headway, h_s = 1.58 s), start loss was t_s = 3.4 s, the saturation speed was v_s = 52.8 km/h where speed limit was 70 km/h, and the jam spacing was L_{hj} = 6.6 m. The full set of parameters determined for this site as well as "average" through and arrow-controlled (protected) right-turn traffic sites are given in *Table 1*. Saturation flow and start loss values in *Table 1* are based on the average saturation headway excluding the vehicles departing during the first 10 seconds (approximately first 4 vehicles) [14].

Figure 3 (based on Akçelik et al [14]) shows various queue discharge parameters and their relationships. Important parameters that determine the saturation headway (h_s) are the jam spacing, i.e. spacing between vehicles in the queue (L_{hj}), queue departure response time for the next vehicle in the queue to start moving (t_x) and the saturation (maximum queue discharge) speed:

$$h_s = t_x + 3.6\ L_{hj} / v_s \quad (1)$$

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

Equation (1) 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:

$$t_x = h_s - 3.6\ L_{hj} / v_s \quad (2)$$

Other parameters shown in *Figure 3* 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 acceleration delay can be calculated from:

$$d_a = t_s + h_s - t_r \quad (3)$$

and assuming $t_r = t_x$:

$$d_a = t_s + 3.6\ L_{hj} / v_s \quad (4)$$

and the queue clearance wave speed can be calculated from:

$$v_x = 3.6\ L_{hj} / t_x = 3.6\ L_{hj} / (h_s - 3.6\ L_{hj} / v_s) \quad (5)$$

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 method for determining the start loss (t_s) parameter is discussed in detail in Akçelik et al [14].

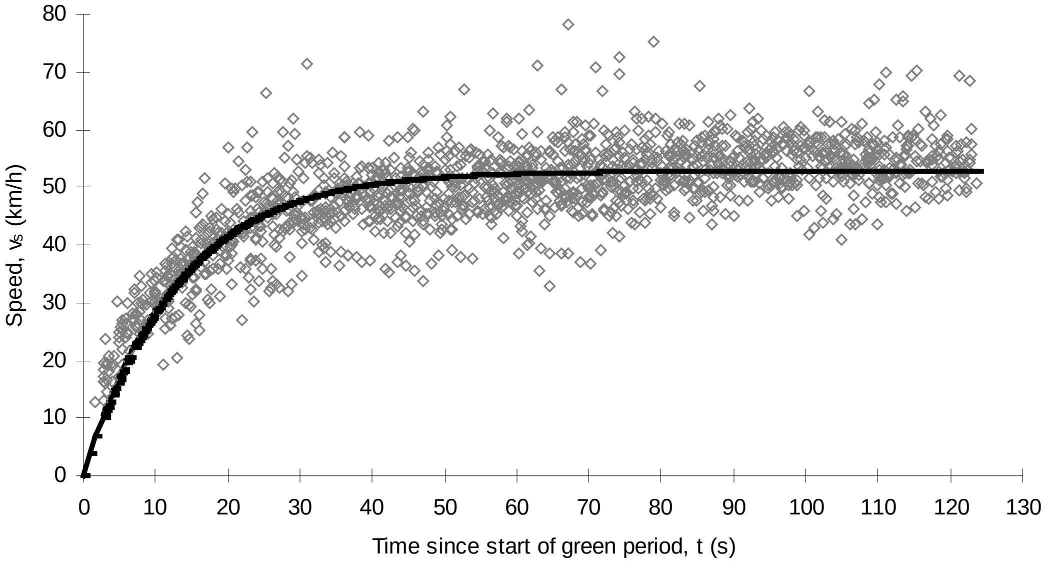

Figure 1 - Queue discharge speeds observed at the intersection of General Holmes Drive and Bestic Street in Sydney (through lane) [14]

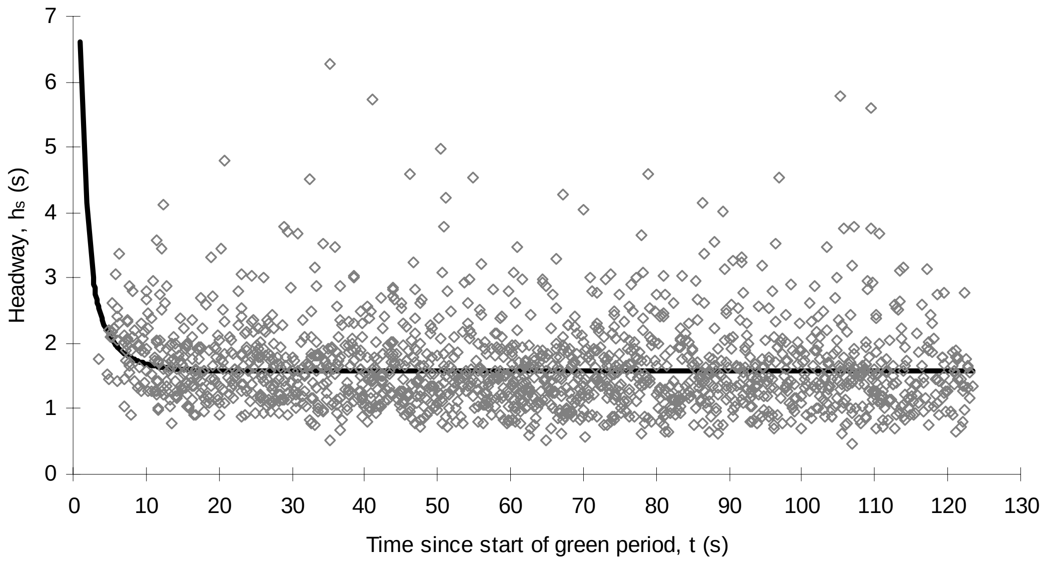

Figure 2 - Queue discharge headways observed at the intersection of General Holmes Drive and Bestic Street in Sydney (through lane) [14]

Table 1 - Queue discharge model parameters

Site	s (veh/h)	h_s (s)	v_s (km/h)	L_{hj} (m)	t_x (s)	v_x (km/h)	t_s (s)	d_a (s)	m_a	t_a (s)	L_a (m)	a_a (m/s^2)	a_m (m/s^2)
Average through traffic site	2056	1.75	42.1	7.0	1.16	21.7	2.30	2.90	0.551	6.46	42	1.81	3.12
Average right-turn traffic site *(1)*	2032	1.77	24.5	6.4	0.84	27.3	1.71	2.64	0.516	5.46	19	1.25	2.13
Site represented by *Figures 1 and 2*	2278	1.58	52.8	6.6	1.13	21.0	3.41	3.86	0.573	9.03	76	1.62	2.83
Example in the *Appendix*	1900	1.89	42.0	7.6 *(2)*	1.24	22.0	2.69	3.34	0.551	7.44	48	1.57	2.70

(1) Isolated arrow-controlled (protected) movement (Left-turn for driving on the right-hand side of the road)
(2) Vehicle length of L_v = 4.8 m, and jam gap distance of L_{sj} = 2.8 m assumed.

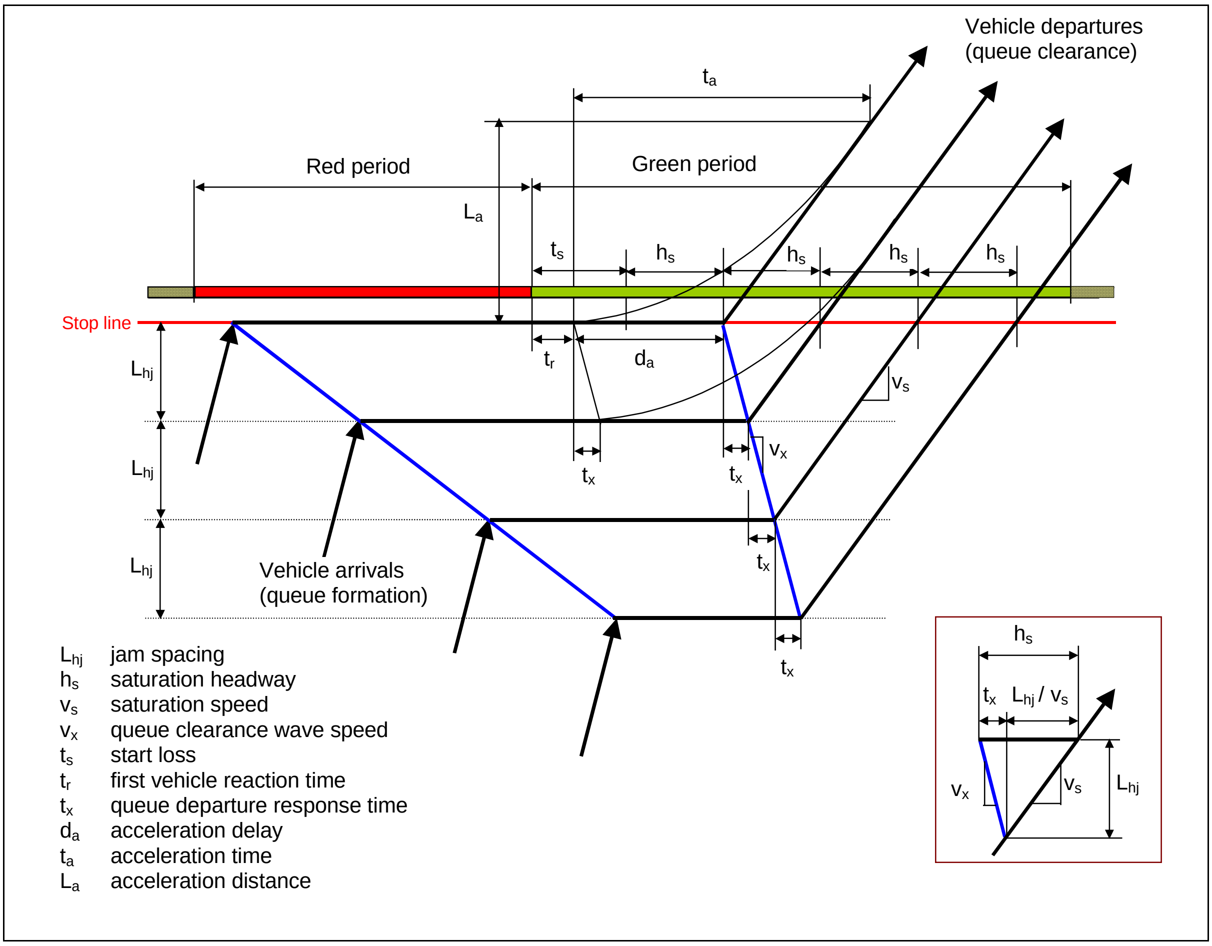

Figure 3 - The relationship between saturation headway, saturation speed, jam spacing, queue departure response time and queue departure wave speed

Information about the acceleration delay can be used for determining parameters for relevant acceleration manoeuvres. Acceleration time (t_a in seconds), acceleration distance (L_a in metres) and the average and maximum acceleration rates (a_a and a_m in m/s^2) given in *Table 1* were calculated using the polynomial acceleration model described by Akçelik and Biggs [15] in the form adopted in the aaSIDRA software [11]. For vehicles accelerating from zero initial speed to the queue discharge speed (v_s):

$$d_a = t_a - 3.6\ L_a / v_s \tag{6}$$

$$t_a = v_s / (3.6\ a_a) \tag{7}$$

$$L_a = m_a\ v_s\ t_a / 3.6 = m_a\ v_s^2 / (12.96\ a_a) \tag{8}$$

$$d_a = (1 - m_a)\ v_s / (3.6\ a_a) \tag{9}$$

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 can be estimated from:

$$m_a = 0.467 + 0.002\ v_s \tag{10}$$

Figure 4 shows the acceleration rate, speed and distance profiles for queue discharge manoeuvres determined using the polynomial acceleration model for the site shown in *Figures 1 and 2.*

From *Equations (4) and (9),* the average acceleration rate can be estimated from:

$$a_a = (1 - m_a)\ v_s / (3.6\ d_a) = (1 - m_a)\ v_s / [3.6\ (t_s + 3.6\ L_{hj} / v_s)] \tag{11}$$

It is recommended that saturation headway (or saturation flow rate), saturation speed and other queue discharge parameters described above are observed in simulation as a function of the queuing, acceleration and car-following model parameters used in simulation. This would be useful to assess reasonableness and accuracy of parameters used in simulation. It could also be possible for the users of microsimulation models to specify saturation flow and saturation speed values observed in the field, or in accordance with accepted methods to predict these parameters, and for the simulation model to be able to adjust its queuing, acceleration and car-following model parameters to achieve the specified saturation flow and speed values.

It is also recommended that queue discharge flow rate (headway) and speed patterns for signalised intersections generated by microsimulation models (in a form similar to *Figures 1 and 2*) are compared with the exponential models proposed by Akçelik et al [14]. These models imply constant saturation speed, headway (time) and spacing (distance) between vehicles as they pass the stop line, which means that queued vehicles do not start accelerating to the cruise speed until after they clear the intersection.

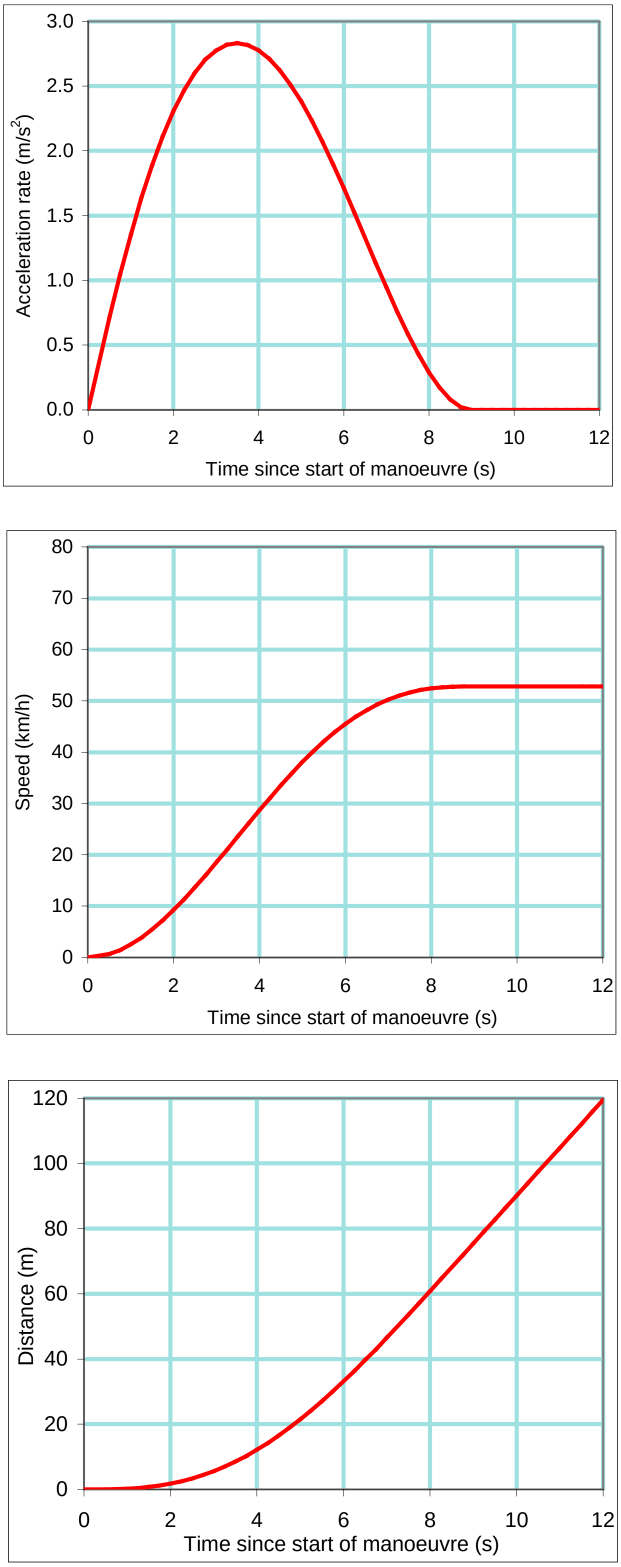

Figure 4 - Acceleration rate, speed and distance profiles for queue discharge manoeuvres determined for the site shown in Figures 1 and 2

4 MODELLING OF GAP-ACCEPTANCE SITUATIONS

Microsimulation models offer a great potential for modelling complex gap-acceptance situations experienced in many situations in urban traffic, e.g. permitted or filter turns (right-turn or left-turn) at signalised intersections, minor movements at give-way or stop signs, traffic entering unsignalised roundabouts, and freeway and other traffic merging situations.

The issues involved in selecting gap-acceptance parameters for analytical models apply equally well to microsimulation models. The gap-acceptance parameters representing driver behaviour include the critical gap and follow-up headway for the minor (opposed or entering) stream, and headway distribution model parameters, e.g. minimum intra-bunch headway and proportion bunched, for the major (opposing or circulating) stream. The use of constant values of the critical gap and follow-up headway parameters against their dependence on intersection geometry and traffic flow conditions (e.g. decreased values as the opposing flow rate increase) is an important issue generally. The reader is referred to Akçelik et al [13] for extensive discussion of the issues in determining gap-acceptance parameters for roundabouts.

It is important for simulation models to report all gap-acceptance parameters used in simulation for the user to assess the reasonableness and accuracy of the model. It would also be useful for the users of microsimulation models to be able to specify all relevant gap-acceptance parameters as observed in the field, or in accordance with accepted methods to predict these parameters.

Microsimulation models generate individual vehicles at external points of entry to the system in accordance with specified headway distributions. Headway distributions change as vehicles travel along the roadway according to car-following and lane-changing rules, and queue at and depart from traffic interruption points such as intersections. For example, the headway distributions in roundabout circulating streams will change in accordance with the queuing process of approach streams contributing to each circulating stream. An interesting issue to explore is how the headway distribution of vehicles in the major (opposing or circulating) traffic stream at the point of conflict (or entry point) matches various theoretical distribution models (bunched exponential, simple negative exponential, etc).

5 LANE USE AT INTERSECTIONS

A general discussion of the lane choice issue for traffic arriving at intersections can be found in Akçelik [16]. A traditional shortcoming of microsimulation models is the use of a simple lane choice algorithm that assigns individual vehicles to the *shortest queue* in any lane considering the lanes available for vehicles with the same destination. Unequal lane utilisation is common at real-life intersections due to geometric and flow conditions, e.g. as caused by a downstream short lane (lane drop), lanes with relatively high proportion of buses or heavy commercial vehicles, shared lanes with turning vehicles, especially with permitted turns subject to opposing vehicle or pedestrian streams, in addition to unequal lane utilisation caused by different destinations of vehicles.

Microsimulation models should allow for the effects of unequal lane utilisation, and provide estimates of lane flows to facilitate comparison with those observed in the field. The user should have the ability to specify lane utilisation rates to affect lane choice algorithms of microsimulation models in order to achieve lane flows observed in the field, or in accordance with accepted methods to predict lane flows.

A relevant issue is the modelling of lane flows at roundabouts. The Australian method defines dominant and sub-dominant lanes at roundabouts on the basis of different capacities and lane flows of entry lanes as observed at real-life roundabouts [12,13]. It is desirable that microsimulation models allow for this behavioural characteristic of traffic at roundabouts. The lane use issue is also important in simulation of freeway flows, e.g. see Stewart [8].

6 DEFINITION OF TRAFFIC PERFORMANCE VARIABLES

An important issue in assessing the usefulness of microsimulation models is the definitions of the traffic performance variables employed. Consistency of such definitions among different models is important for the user, and this issue applies equally well to both analytical or simulation models.

An essential requirement for a model is to define traffic performance measures such as "delay", "queue length" and "stops" clearly and precisely. For example, delay could mean control delay, stop-line delay, queuing delay or stopped delay, and additional types of delay such as geometric delay, queue move-up delay and major stop-start delay need to be identified for various purposes [11,13]. Distinguishing between delay based on the queue sampling method vs delay based on the path trace (instrumented car) method is also important in oversaturated conditions experienced in a time slice employed in variable-demand modelling [17].

Similarly, queue length could mean back of queue, cycle-average queue, queue at the start of green period, or overflow queue, and it is important to identify average and percentile values of each of these queue lengths. A detailed discussion of comparison of queue length estimates by two different simulation models can be found in a recent article by Trueblood [9]. *Figure 5* shows a comparison of simulated cycle-average queue values vs simulated average back of queue values for roundabouts [11].

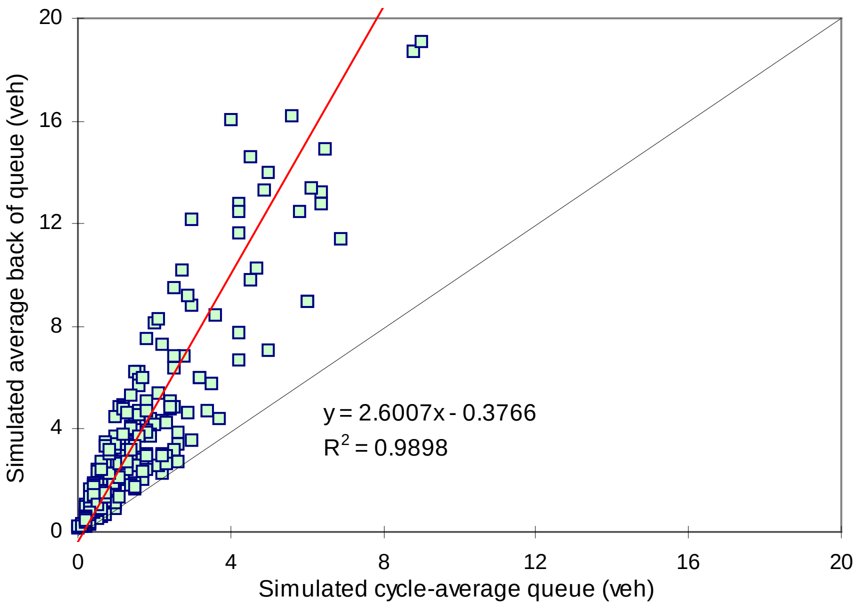

Figure 5 - Simulated cycle-average queue values vs average back of queue values for roundabouts

A stop is particularly difficult to define, and such terms as stop rate, major stop, partial stop, geometric stop and proportion stopped are relevant.

Essentially, microsimulation packages should be able to derive (or "measure") all the above performance variables although deriving some of them may be more difficult, and calculation of some types of delay would involve some level of calculation beyond direct observations of time spent in various modes of driving. Options for users to select particular definitions of the performance variables would be useful.

7 BENCHMARKING for MODEL COMPARISONS

Description of case studies for model comparison may be a useful method for benchmarking for evaluation of new and existing models. Ideally, benchmarking should be based on real-life case studies, e.g. see the roundabout case studies described in Akçelik et al [13]. It may also be useful to describe hypothetical case studies for the purpose of testing fundamental model elements (components), e.g. see Yoshii [18]. The main benefit of comparison of models, especially analytical vs microsimulation, is a verification process [18] to establish detailed issues involved in determining reasonableness and accuracy of models rather than comparing estimates produced by alternative models with the implication that one model is better than the others. With this in mind, a simple hypothetical signalised intersection case is described in the *Appendix*.

The purpose of the signalised intersection case described in the *Appendix* is to enable assessment of two basic traffic model components, namely queue discharge flow rate and lane flow distribution. While there is a large number of factors that affect queue discharge characteristics at signalised intersections, the example is set only to test how alternative models allow for the turning vehicle, heavy vehicle and road grade effects.

8 CONCLUDING REMARKS

In-depth studies of the correspondence between key traffic parameters used in simulation and analytical methods are recommended as outlined in this paper. Other fundamental traffic parameters such as speed-flow-density relationships, vehicle spacings, headways, occupancy and space times, and detector occupancy ratios (time and space) should also be studied [14,19]. At a more sophisticated level, parameters used for fuel consumption, pollutant emissions and operating costs should be subject to comparative studies [11].

Microsimulation software packages should provide facilities to calibrate the models in simple traffic engineering terms as discussed in this paper in order to increase their usefulness.

The need for clarification of the definitions of performance variables used in various software packages, and effort towards standardisation or provision of options for users to choose their preferred definitions is emphasised.

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APPENDIX -
A Signalised Intersection Case for Testing Some Basic Aspects of Simulation and Analytical Models

Description

This signalised intersection example has been set for testing some basic aspects of simulation and analytical models. The Highway Capacity Manual metric version of the aaSIDRA software was used for setting this example, including HCM default values of road and traffic parameters, and the HCM delay and queue models [10,11].

The purpose is to test the following aspects of signalised intersection modelling:

(i) comparison of queue discharge flow characteristics of left, through and right turn movements,

(ii) effect of road grade and heavy vehicles on queue discharge flow characteristics, and

(iii) estimation of lane flows for an approach road with shared lanes, including the possibility of a de facto exclusive lane.

The test case is kept as simple as possible. For this purpose, it is assumed that:

- there are no slip lanes or short lanes,
- there are no opposed (permitted) turns,
- there are no Right Turns On Red,
- there are no pedestrians,
- there are no parking manoeuvres or buses stopping,
- there is no peaking of demand flows,
- all lanes have the same width,
- in the analytical model, all movements have the same basic (ideal) saturation flow, start loss and end gain characteristics, but actual queue discharge flow rates will be lower due to the factors discussed below.

The only factors that affect queue discharge characteristics are:

- turning vehicle effects,
- heavy vehicle effects, and
- road grade effects.

The intersection characteristics are as follows:

- Driving on the right-hand side of the road applies.
- A cross intersection with divided two-way legs (South, East, North and West).
- All approaches have four lanes. Intersection geometry including lane configurations is shown in *Figure A.1*.
- Controlled by pretimed (fixed-time) signals.
- There is no signal coordination, therefore random arrivals are assumed.

- A four-phase system is used where each approach operates separately, and is served in clockwise order ("split-approach" phasing). Signal Phasing is shown in *Figure A.2*.
- All signal phases have the same intergreen time, which is I = 5 s (yellow time = 4 s, all-red time = 1 s).
- The cycle time is 100 s. Equal green splits are used, i.e. the green time is 20 s, or the phase time (green plus intergreen) is 25 s, for each approach.
- Basic saturation flow rate is 1900 passenger car units per hour per lane (HCM [10,11] default value). The corresponding queue discharge parameters, including acceleration characteristics are shown in *Table 1*.
- Start loss and end gain values for all movements are 3.0 s (these differ from the HCM [10,11] default value of 2 s in order to be consistent with other queue discharge parameters given in *Table 1*, but this has little effect on results).
- All lane widths are 3.60 m (HCM [10] default).
- Median widths are 2.0 m for all intersection legs.
- The analysis period is 15 minutes. Demand volumes measured during the 15-minute peak period are used, therefore Peak Hour factor = 1.0. Demand flow rates in vehicles *per hour* derived from these values are given in *Figure A.3*. The demand flow rate is constant during the 15-min analysis period.
- Heavy vehicles exist on East approach (10 per cent for all movements) and West approach (10 per cent for right-turn movement only).
- All intersection legs are level (zero grade) except North leg that has 10 per cent uphill grade.
- Approach and intersection negotiation distances and speeds are given in *Figure A.4*.

  All intersection legs are Class II urban streets with free-flow speed (or speed limit) of 65 km/h in both directions and approach distance of 500 m (see HCM2000 Exhibits 10-5 and 10-6) [10]. As seen in *Figure A.4*, approach distance is the mid-block distance from one intersection to the next and does not include the intersection negotiation distances. This differs from the stop line to stop line distance that includes the intersection negotiation distance (see *Figure A.4*). Therefore, the stop line to stop line distance varies according to the destination of each movement. The approach distance of 500 m represents roughly two intersections per kilometre (or three intersections per mile).

  Negotiation distances and speeds affect the travel time through the intersection, and contribute to the overall delay experienced by each vehicle. Safe negotiation speeds given in *Figure A.4* are for unqueued vehicles (18 km/h for right-turning vehicles, 65 km/h for through vehicles, and 25 km/h for left-turning vehicles). For unqueued through vehicles, intersection negotiation speeds are assumed to be the same as the approach speeds. For unqueued turning movements, safe negotiation speed depends on the turning radius. The speeds in *Figure A.4* are based on 10 m radius for right-turning movements, 20 m radius for left-turning movements).
- Queue (jam) space is 7.60 m per light vehicle and 14.00 m per heavy vehicle. Assuming a jam gap distance of 2.8 m, the implied vehicle lengths are 4.80 m per light vehicle and 11.20 m per heavy vehicle.

The Results

It is expected that the microsimulation models will be able to predict the following characteristics:

(i) Capacities (or saturation flows) for turning vehicles are lower relative to through vehicles on each approach.

(ii) Capacities are lower, and delays and queues are longer with 10 per cent heavy vehicles (compare West approach vs South approach).

(iii) Capacities are lower, and delays and queues are longer with 10 per cent uphill grade (compare North approach vs South approach).

(iv) On West approach:

   (a) left turn lane is a defacto exclusive lane, i.e. effectively a "left turn only" lane as no through vehicles use it, although the lane arrangement is specified as shared left and through (the defacto exclusive lane should be predicted by the model, not specified as input); and

   (b) only a small proportion of through traffic uses the right lane due to the lower capacity resulting from slower right-turn speeds (longer right-turn vehicle headways), and the queue lengths in the through only lanes (two middle lanes) are longer than the queue length in the shared right-turn and through lane.

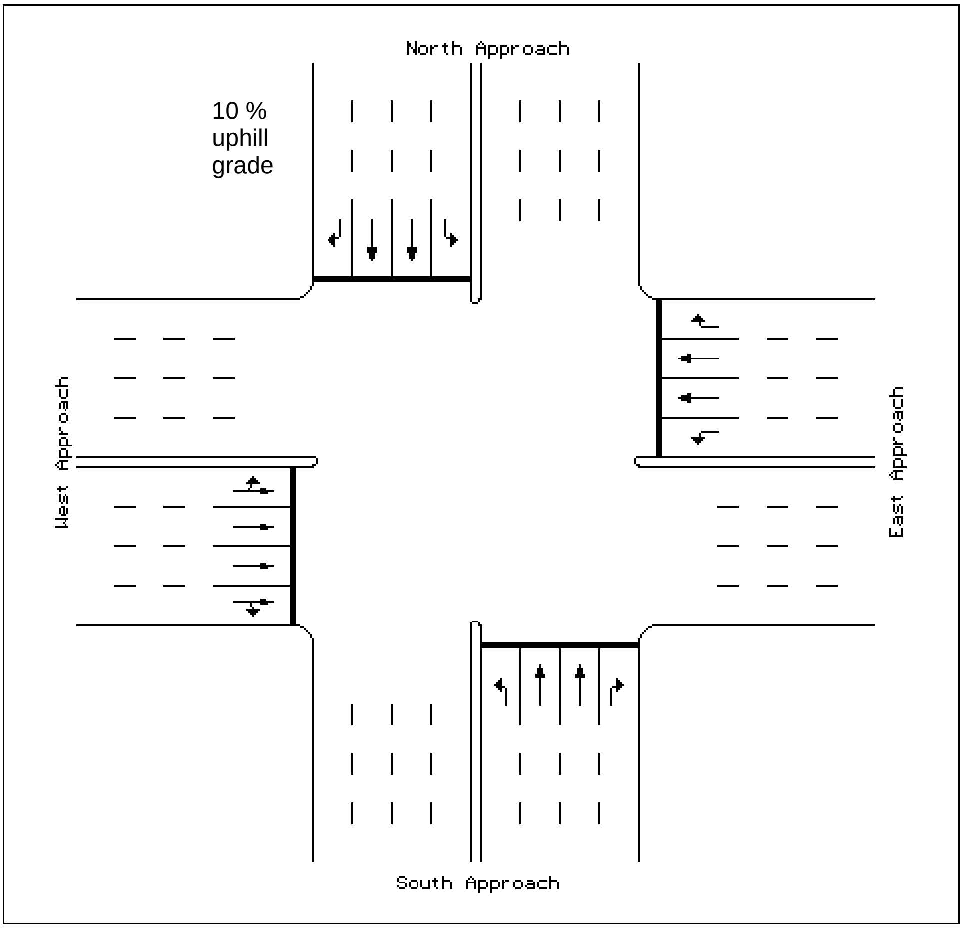

Figure A.1 - Intersection geometry

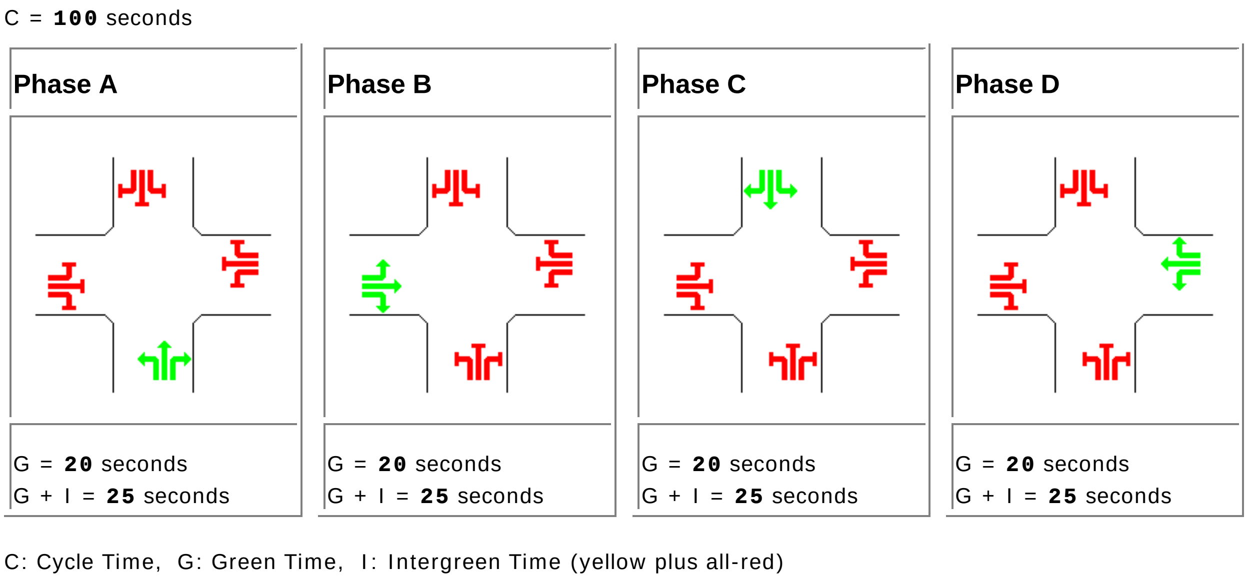

Figure A.2 - Signal phasing and timing

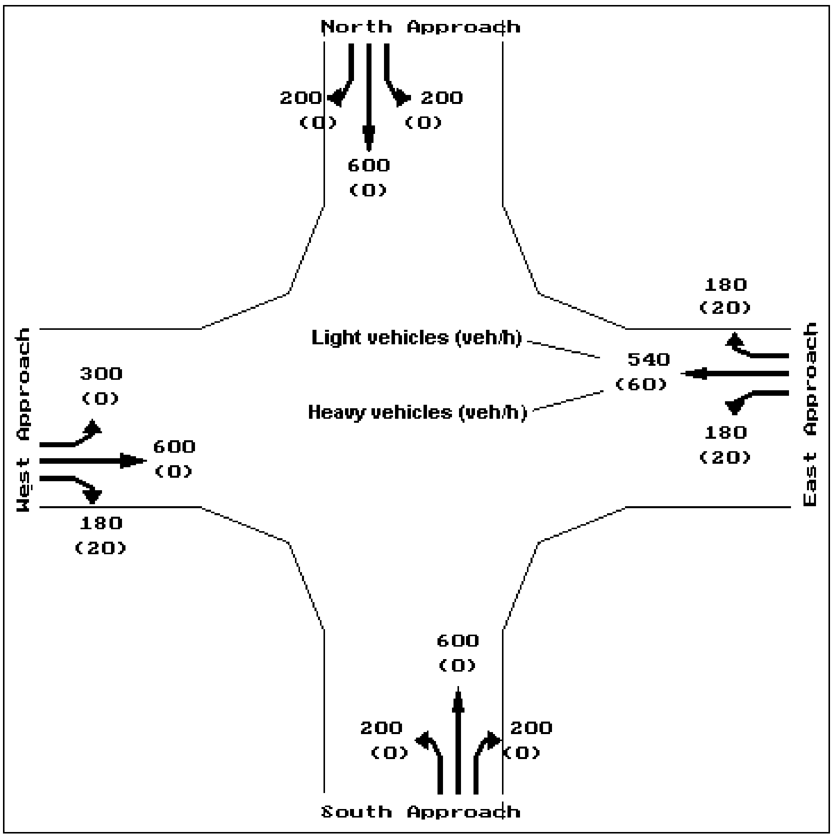

Figure A.3 - Demand volumes (divide the veh/h values shown by 4 for 15-min peak volumes)

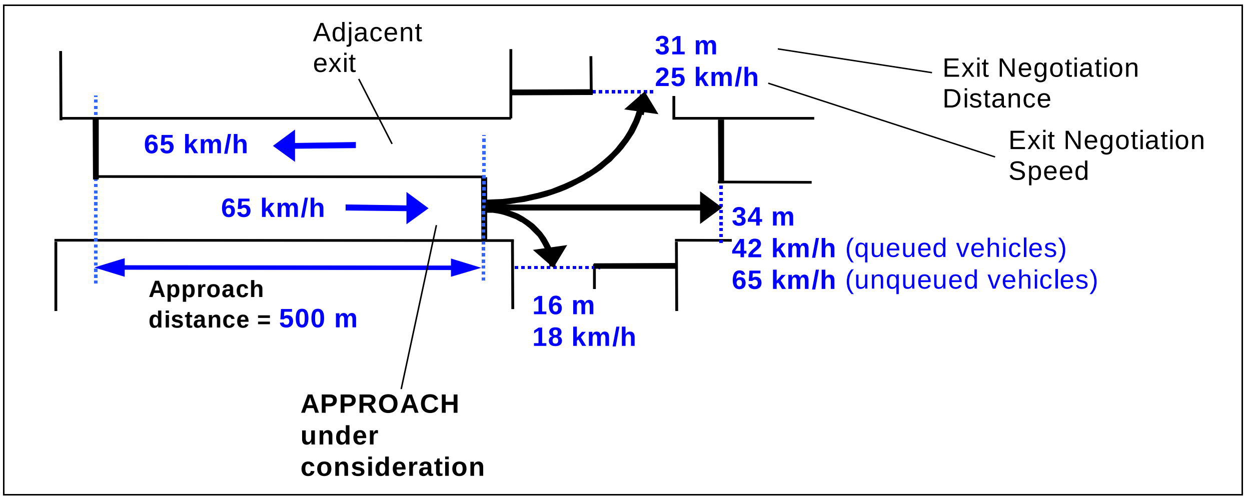

Figure A.4 - Definition of approach and intersection negotiation distances and speeds