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REPRINT

Analysis of roundabout performance by modelling approach flow interactions

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NOTE:

This paper is related to the intersection analysis methodology used in the SIDRA INTERSECTION software. Since the publication of this paper, many related aspects of the traffic model have been further developed in later versions of SIDRA INTERSECTION. Though some aspects of this paper may be outdated, this reprint is provided as a record of important aspects of the SIDRA INTERSECTION software, and in order to promote software assessment and further research.

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Analysis of roundabout performance by modelling approach flow interactions

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ABSTRACT

An analytical method for estimating roundabout entry lane capacity and performance measures is presented. The method is based on modelling the gap acceptance process that takes place in real-life roundabout operation. Unlike past studies that treated roundabouts *as a series of T-junctions*, the method presented here allows for *approach flow interactions*. A factor is used to adjust the basic gap-acceptance capacity for the effects of the origin-destination pattern and the queueing characteristics of the approach flows. Circulating stream characteristics are determined considering the approach lane use characteristics of the traffic streams that constitute the circulating flow. The modelling of interactions amongst approach flows is important, especially in heavy and unbalanced demand flow cases. Ignoring approach flow interactions can cause serious overestimation of capacity, and underestimation of delays and queue lengths, especially for multi-lane roundabouts with unbalanced flow patterns. This is demonstrated through a case study that compares the results from the methods with and without approach flow interactions. Formulae are presented for the estimation of stop-line (control) delay, queue length, as well as proportion queued, queue clearance time and queue move up rate. The formulae were derived and calibrated using the two-term model structure based on the *overflow queue* concept as used in the well-established method for signalised intersections. The formulae also allow for the effects of any *initial queued demand* at the start of the analysis period. The difference between the *cycle-average queue* and the *average back of queue* is emphasised.

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Edward Chung graduated from Monash University in 1986 with a Bachelor degree in Civil Engineering. He studied at the University of California, Berkeley in 1989. Edward completed his Ph.D. work on modelling of roundabout performance in 1993. He worked at ARRB TR for several years before joining R.J. Nairn and Partners in 1994, and was involved in a wide range of projects. Edward rejoined ARRB TR as a Senior Research Scientist in 1996. His current research interests include traffic modelling, intelligent transport systems and public transport issues.

Mark Besley

Mark Besley is a senior research scientist at ARRB Transport Research Ltd. Mark joined ARRB TR in 1980 after studying applied mathematics at Monash University. During his time at ARRB TR, Mark has been involved in traffic data collection and analysis, software development, training courses and conference organisation. He has made a significant contribution to the development and support of the SIDRA computer package since 1982. Currently, Mark is involved with paired intersection research and the incorporation of the results of latest ARRB TR research into the SIDRA model.

INTRODUCTION

This paper presents a comprehensive method for roundabout capacity and performance estimation allowing for *approach flow interactions* rather than treating the roundabout as a series of *independent T-junctions*. The method takes into account the effects of the origin-destination demand pattern, lane usage and queueing characteristics of approach flows. The development of the method was described and various aspects discussed in recent publications which also presented a real-life case study *(1,2)*. The method was first implemented in the SIDRA 4.1 software package, and has been available in the latest version SIDRA 5 with minor refinements *(3)*. As such, the method has been used extensively in practice, with all user feedback indicating that the method has solved problems encountered with earlier methods *(2)*.

Roundabout performance models for the estimation of delay, queue length, proportion queued, queue clearance time and queue move up rate presented in this paper are based on a general two-term model structure that uses the *overflow queue* concept. Discussions of the general model structure and the overflow concept, as well as the formulae for fixed-time signals, actuated signals, two-way stop and give-way (yield) sign control can be found in the SIDRA 5 user guide *(3)* and other publications that present detailed discussions on various aspects of the capacity and performance models *(4-10)*. The roundabout performance models presented here were calibrated using the microscopic simulation program MODELC *(1,5,11,12)*.

Recently, Akçelik *(13)* extended the performance model formulations to the case with an *initial queued demand* at the start of the flow (analysis) period due to oversaturation in the previous flow period. The roundabout performance models given in this paper use this extended form.

CAPACITY AND PERFORMANCE MODELS

The formulae given in this section can be used for predicting the performance and capacity of a roundabout entry lane. A list of notations is given first. Formulae for average stop-line (control) delay, total (aggregate) delay, average back of queue, average overflow queue, cycle-average queue, percentile queue lengths, queue move-up rate, proportion queued and queue clearance time are given, followed by expressions for parameters common to the formulae.

Additional information such as the method for calculating effective stop rate expressed in terms of equivalent major stop values (ESVs), and the method to estimate gap acceptance parameters (critical gap and follow-up headway) can be found in the SIDRA user guide *(3)*.

Notations

b	A calibration parameter in the formula for estimating proportion of free (unbunched) vehicles in the traffic stream (see *Table 1*)
c	Equivalent average cycle time corresponding to the *block and unblock periods* in the circulating traffic stream ($c = r + g$)
d	Average stop-line (control) delay per vehicle as the average delay to vehicles *arriving* during the current flow period, and considering all vehicles queued and unqueued (seconds)
d_1, d_2	First and second terms of the delay formula

d_m	Minimum (average) stop-line (control) delay experienced by a vehicle at near-zero entry flow conditions (seconds)
D	Total (aggregate) delay in veh-h/h
$f_{bp\%}$	Factor for *pth* (90th, 95th, 98th) percentile back of queue
$f_{cp\%}$	Factors for *pth* (90th, 95th, 98th) percentile cycle-average queue
f_{od}	Factor to adjust the basic gap-acceptance capacity for roundabout origin-destination flow pattern and approach queueing effects
f_{qc}	A calibration parameter in the formula for the factor (f_{od}) for roundabout origin-destination flow pattern and approach queueing effects
g	Equivalent average green time corresponding to the *unblock periods* in the circulating traffic stream
g_s	Average queue clearance time (seconds)
h_{qm}	Queue move-up rate (average number of acceleration-deceleration cycles while in the queue before clearing the intersection)
k_d	Overflow term parameter in the formula for average stop-line (control) delay
k_b	Overflow term parameter in the formula for average back of queue
k_{qm}	Overflow term parameter in the formula for queue move-up rate
k_o	Parameter in the formula for average overflow queue
l	Equivalent lost time that corresponds to the unused portion of the unblock period in the circulating traffic stream (seconds) ($l = 0.5\ \beta$)
n_m	Minimum number of vehicles per minute which can enter the circulating stream under heavy flow conditions (veh/min)
N_b	Average back of queue (vehicles)
N_{b1}, N_{b2}	First and second terms of the back of queue formula
$N_{bp\%}$	*pth* (90th, 95th, or 98th) percentile value of the back of queue
N_c	Cycle-average queue (vehicles)
$N_{cp\%}$	*pth* (90th, 95th, or 98th) percentile value of the cycle-average queue
N_i	Initial queued demand as observed at the start of a flow period (vehicles)
N_j	Residual queued demand as observed at the end of a flow period (vehicles)
N_o	Average overflow queue (vehicles)
pcu	Passenger car units (used to allow for the effect of heavy vehicles in the circulating stream)
p_{cd}	Proportion of the total circulating flow that originated from the dominant approach ($p_{cd} = q_{cd} / q_c$)

p_q	Proportion queued (considering major stops or slow-downs from the approach negotiation speed)
p_{qd}	Proportion of queued vehicles on the dominant roundabout approach
q	Flow rate (veh/s or veh/h): number of vehicles per unit time passing (*arriving* or *departing*) a given reference point
q_c	Total circulating flow rate relevant to the subject entry lane (calculated in pcu/h by adjusting the flow rate for heavy vehicles)
q_{cd}	Part of the total circulating flow that originated from the dominant approach
q_e	Arrival (demand) flow rate of the entry lane (veh/s or veh/h), i.e. the average number of vehicles per unit time as measured at a point upstream of the back of queue
q_{ei}	Demand flow rate of the entry lane adjusted to take into account the initial queued demand at the start of the flow period ($q_{ei} = q_e + N_i / T_f$)
$q_e c$	Number of arrivals per cycle in the entry lane as measured at the back of the queue (vehicles)
$q_{ei} c$	Average demand (vehicles) per cycle in the entry lane corresponding to the total demand including initial queued demand
Q_e	Capacity of the entry lane (veh/h); this is the maximum arrival flow rate that can be serviced under prevailing flow conditions
$Q_e T_f$	Throughput (vehicles): maximum number of vehicles that can be discharged during a flow period of duration T_f
Q_g	Capacity estimate using the basic gap-acceptance method (veh/h) ($Q_g = sg/c$)
Q_m	Minimum entry lane capacity (veh/h)
r	Equivalent average red time corresponding to the *block periods* in the circulating traffic stream
s	Saturation flow rate (veh/h) ($s = 3600 / \beta$)
sg	Cycle capacity (veh) (s in veh/s, g in seconds): the maximum number of vehicles that can discharge during the average *unblock period*
T_f	Duration of a flow period (hours)
T_i	Time for the initial queued demand to clear (hours) ($T_i = N_i / Q_e$)
T_j	Time for the residual queued demand to clear (hours) ($T_j = N_j / Q_e$)
u	Equivalent green time ratio ($u = g / c$)
x	Degree of saturation, i.e. the ratio of entry lane (demand) flow rate to capacity ($x = q_e / Q_e$)

x'	Effective degree of saturation allowing for the effect of the initial queued demand ($x' = q_{ei} / Q_e = x + N_i / (Q_e T_f)$)
x_o	Non-zero overflow degree of saturation (the average overflow queue, queue move-up rate and the second terms of the formulae for delay and back of queue are zero for degrees of saturation below x_o)
y	Flow ratio, i.e. the ratio of arrival (demand) flow rate to the saturation flow rate, including the effect of the initial queued demand ($y = q_{ei} / s = q_{ei}\, \beta / 3600$ where q_{ei} and s are in veh/h; if there is no initial queued demand, $N_i = 0$, $y = q_e / s = q_e\, \beta / 3600$)
z	A performance model parameter used in the formulae for the average overflow queue, queue move-up rate and the second terms of the formulae for delay and back of queue ($z = x - 1 + 2\, N_i / (Q_e T_f)$)
α	Mean critical acceptance gap for the entry traffic stream (seconds)
β	Follow-up (saturation) headway of the entry traffic stream (seconds)
Δ	Intra-bunch headway, i.e. the minimum headway in the arrival headway distribution model (seconds)
Δ_c	Intra-bunch headway in the circulating traffic stream relevant to the subject entry lane (seconds)
Δ_e	Intra-bunch headway in the entry lane traffic stream (seconds)
φ	Proportion of free (unbunched) vehicles in the traffic stream
φ_c	Proportion of free (unbunched) vehicles in the circulating traffic stream
φ_e	Proportion of free (unbunched) vehicles in the entry lane traffic stream
λ	A parameter in the exponential arrival headway distribution model

Average stop-line (control) delay

(d, d_1, d_2, d_m, α, Δ_c in seconds, T_f in hours, Q_e in veh/h, q_c in pcu/h, sg, N_i in vehicles)

$$d = d_1 + d_2 \qquad (1)$$

$$d_1 = \frac{d_m\,(1 + 0.3\; y^{0.20})}{1 - y} \qquad \textit{for } x' \le 1.0 \qquad (1a)$$

$$= d_{1(x'=1)} \qquad \textit{for } x' > 1.0$$

$$d_2 = 900\, T_f \left[\, z + \left(z^2 + \frac{8\, k_d\, (x - x_o)}{Q_e T_f} + \frac{16\, k_d\, N_i}{(Q_e T_f)^2}\right)^{0.5} \right] \qquad \textit{for } x > x_o \qquad (1b)$$

$$= 0 \qquad \textit{otherwise}$$

where

$$d_m = \frac{3600\, e^{\lambda(\alpha-\Delta_c)}}{\varphi_c\, q_c} - \alpha - \frac{1}{\lambda} + \frac{\lambda\, \Delta_c^2 - 2\,\Delta_c + 2\Delta_c\,\varphi_c}{2\,(\lambda\, \Delta_c + \varphi_c)} \qquad \text{for } q_c > 0 \qquad (1c)$$

$$= 0 \qquad \text{otherwise}$$

$$k_d = 0.20\, \varphi_e\, (sg)^{1.30}\, y^{-0.40}\, (d_m\, Q_e / 3600) \qquad (1d)$$

and $d_{1(x'=1)}$ is the value of d_1 at x' = 1.0 (or at y = u); if $N_i = 0$, the condition x' = 1 corresponds to x = 1, or $q_e = Q_e$.

Total (aggregate) delay

(D in veh-h/h, d_1, d_2 in seconds, q_e, q_{ei} in veh/s)

$$D = d_1\, q_{ei} + d_2\, q_e \qquad (2)$$

Average back of queue

(N_b, N_{b1}, N_{b2}, N_i, sg in vehicles, r, c, d_m in seconds, T_f in hours, Q_e in veh/h, q_{ei} veh/s)

$$N_b = N_{b1} + N_{b2} \qquad (3)$$

$$N_{b1} = \frac{1.2\, \varphi_e^{0.8}\, q_{ei}\, r}{1 - y} \qquad \text{for } x' \le 1.0 \qquad (3a)$$

$$= 1.2\, \varphi_{e(x'=1)}^{0.8}\, q_{ei}\, c \qquad \text{for } x' > 1.0$$

$$N_{b2} = 0.25\, Q_e T_f \left[z + \left(z^2 + \frac{8\, k_b\, (x-x_o)}{Q_e T_f} + \frac{16\, k_b\, N_i}{(Q_e T_f)^2}\right)^{0.5} \right] \qquad \text{for } x > x_o \qquad (3b)$$

$$= 0 \qquad \text{otherwise}$$

where

$$k_b = 0.40\, \varphi_e\, (sg)^{1.40}\, y^{0.40}\, (d_m\, Q_e / 3600) \qquad (3c)$$

and $\varphi_{e(x'=1)}$ is the value of φ_e at x' = 1.0 (or at $q_e = Q_e - N_i / T_f$); if $N_i = 0$, the condition x' = 1 corresponds to x = 1, or $q_e = Q_e$.

Average overflow queue

(N_o, N_i, sg in vehicles, d_m in seconds, T_f in hours, Q_e in veh/h)

$$N_o = 0.25\, Q_e T_f \left[z + \left(z^2 + \frac{8\, k_o\, (x-x_o)}{Q_e T_f} + \frac{16\, k_o\, N_i}{(Q_e T_f)^2}\right)^{0.5} \right] \qquad \text{for } x > x_o \qquad (4)$$

$$= 0 \qquad \text{otherwise}$$

where

$$k_o = 0.30\, \varphi_e\, (sg)^{1.10}\, (d_m\, Q_e / 3600) \qquad (4a)$$

Cycle average queue

(N_c in vehicles)

$$N_c = D \tag{5}$$

Percentile back of queue

(N_b, $N_{bp\%}$ in vehicles)

$$N_{bp\%} = f_{bp\%} \, N_b \tag{6}$$

where

$$f_{b90\%} = 1.9 + 0.7 \, e^{-N_b/8} \tag{6a}$$

$$f_{b95\%} = 2.5 + 0.7 \, e^{-N_b/8} \tag{6b}$$

$$f_{b98\%} = 3.0 + 0.7 \, e^{-N_b/8} \tag{6c}$$

Percentile cycle-average queue

(N_c, $N_{cp\%}$ in vehicles)

$$N_{cp\%} = f_{cp\%} \, N_c \tag{7}$$

where

$$f_{c90\%} = 1.6 + 0.7 \, e^{-N_c/8} \tag{7a}$$

$$f_{c95\%} = 1.8 + 0.8 \, e^{-N_c/8} \tag{7b}$$

$$f_{c98\%} = 1.9 + 1.5 \, e^{-N_c/8} \tag{7c}$$

Proportion queued

(sg in vehicles)

$$p_q = \frac{0.78 \, \varphi_e \, (sg)^{0.40} \, (1 - u)}{1 - y} \qquad \textit{subject to } p_q \le 1.0 \tag{8}$$

Queue move-up rate

(sg, N_i in vehicles, c, d_m in seconds, T_f in hours, Q_e in veh/h, q_e veh/s)

$$h_{qm} = \frac{0.25 \, Q_e T_f}{q_e \, c} \left[z + \left(z^2 + \frac{8 \, k_{qm} \, (x - x_o)}{Q_e T_f} + \frac{16 \, k_{qm} \, N_i}{(Q_e T_f)^2}\right)^{0.5} \right] \qquad \textit{for } x > x_o \tag{9}$$

$$= 0 \qquad \textit{otherwise}$$

where

$$k_{qm} = 0.40 \, \varphi_e \, (sg)^{1.15} \, (d_m \, Q_e / 3600) \tag{9a}$$

Queue clearance time

(sg in vehicles, g_s, r in seconds)

$$g_s = \frac{0.78\ \varphi_e\ (sg)^{0.40}\ y\ r}{1 - y} \qquad \textit{subject to } g_s \le g \qquad (10)$$

Capacity

(sg in vehicles, c, α, β, Δ_c in seconds, q_c in pcu/h, Q_e, Q_g, Q_m, q_e in veh/h, n_m in veh/min)

$$Q_e = \textit{max}\ (f_{od}\ Q_g,\ Q_m) \qquad (11)$$

$$Q_g = \frac{sg}{c} = \left(\frac{3600}{\beta} - \frac{\Delta_c}{\beta}\ q_c + 0.5\ \varphi_c\ q_c\right) e^{-\lambda(\alpha-\Delta_c)} \qquad (11a)$$

$$Q_m = \textit{min}\ (q_e,\ 60\ n_m) \qquad (11b)$$

$$f_{od} = 1 - f_{qc}\ (p_{qd}\ p_{cd}) \qquad (11c)$$

Single-lane stream circulating flow:

$$f_{qc} = 0.04 + 0.00015\ q_c \qquad \textit{for } q_c < 600 \qquad (11d)$$
$$= 0.0007\ q_c - 0.29 \qquad \textit{for } 600 \le q_c \le 1200$$
$$= 0.55 \qquad \textit{for } q_c > 1200$$

Multi-lane stream circulating flow:

$$f_{qc} = 0.04 + 0.00015\ q_c \qquad \textit{for } q_c < 600 \qquad (11e)$$
$$= 0.00035\ q_c - 0.08 \qquad \textit{for } 600 \le q_c \le 1800$$
$$= 0.55 \qquad \textit{for } q_c > 1800$$

Common parameters

(sg, N_i in vehicles, c, g, r, α, β, Δ_e, Δ_c in seconds, T_f in hours, Q_e in veh/h, q_e, q_{ei}, q_c, s in veh/s)

$$x_o = 0.18\ (sg)^{0.60} \qquad \textit{subject to } x_o \le 0.95 \qquad (12)$$

$$y = q_{ei} / s = \beta\ q_{ei} \qquad (13a)$$

$$x = q_e / Q_e \qquad (13b)$$

$$x' = q_{ei} / Q_e = x + \frac{N_i}{Q_e T_f} \qquad (13c)$$

$$z = x - 1 + \frac{2N_i}{Q_e T_f} \qquad (13d)$$

$$sg = g / \beta \qquad (14)$$

$$\varphi_e = e^{-b\ \Delta_e\ q_e} \qquad (15a)$$

$$\varphi_c = e^{-b\,\Delta_c\,q_c} \qquad (15b)$$

$$\lambda = \frac{\varphi_c\,q_c}{1-\Delta_c\,q_c} \qquad \textit{subject to } q_c \le 0.98/\Delta_c \qquad (16)$$

$$c = \frac{e^{-\lambda(\alpha-\Delta_c)}}{\varphi_c\,q_c} \qquad \textit{for } q_c > 0 \qquad (17a)$$
$$= 0 \qquad \textit{otherwise}$$

$$g = \frac{1}{\lambda} + 0.5\,\beta \qquad \textit{for } q_c > 0 \textit{ (hence } \lambda > 0) \qquad (17b)$$
$$= c \qquad \textit{otherwise}$$

$$r = \frac{e^{-\lambda(\alpha-\Delta_c)}}{\varphi_c\,q_c} - \frac{1}{\lambda} - 0.5\,\beta \qquad \textit{for } q_c > 0 \textit{ (and } l > 0) \qquad (17c)$$
$$= 0 \qquad \textit{otherwise}$$

$$u = g\,/\,c = (1 - \Delta_c\,q_c + 0.5\,\beta\,\varphi_{c.}q_c)\,e^{-\lambda(\alpha-\Delta_c)} \qquad (17d)$$

Circulating Stream Characteristics

The values of the intra-bunch headway (Δ) and the calibration parameter (b) in the formula to calculate the proportion unbunched (φ) for circulating flows and entry lane arrival flows at roundabouts are given in *Table 1*. A flow-weighted average of Δ_c is used when the streams contributing to the circulating flow are different in terms of being *single-lane* or *multi-lane* (using contributing flow rates in pcu/h). This is determined by inspecting the effective *approach lane use* characteristics of the flows that constitute the circulating stream. Thus, the value of Δ_c may be in the range 1.2 to 2.0. In the example shown in *Figure 1*, the circulating flow for the South approach is 900 pcu/h which consists of through flow from the West approach (600 pcu/h in two lanes, hence Δ_c = 1.2 s) and left-turn flow from the North approach (300 pcu/h in one lane, hence Δ_c = 2.0 s). The intra-bunch headway for South approach lanes is calculated as Δ_c = (600 x 1.2 + 300 x 2.0) / 900 = 1.47 s.

Factor for Origin-Destination Pattern and Approach Queueing

The basis of the model for estimating the capacity of a roundabout entry lane (Q_e) is to use a factor (f_{od}) to reduce the basic gap-acceptance capacity (Q_g) to allow for the effects of the *origin-destination pattern* and *approach queueing* characteristics of traffic that constitute the circulating stream as seen from *Equations (11) to (11e)*. The two variables in the factor (f_{od}) to reduce the basic gap-acceptance capacity are:

(i) the proportion of the total circulating stream flow that originated from the *dominant approach* ($p_{cd} = q_{cd} / q_c$), and

(ii) the proportion queued for that part of the circulating stream that originated from the *dominant approach* (p_{qd}).

The *dominant approach* is determined as the approach that has the highest value of ($p_{qd}\ p_{cd}$) considering all approaches that contribute to the circulating flow ($p_{qd}\ p_{cd}$ is the proportion of the total circulating stream flow that originated from and were queued on the dominant approach). For multi-lane approach roads that contribute to the circulating flow, the value of ($p_{qd}\ p_{cd}$) is calculated as a flow-weighted average of individual lane values considering the lanes used by the relevant movements (using contributing flow rates in pcu/h).

For the purpose of calculating parameter f_{qc}, the total circulating flow rate is used for both single-lane and multi-lane circulating streams. In the case of multi-lane circulating roads, f_{qc} is calculated as a flow-weighted value of the *single-lane* and *multi-lane* values (using contributing flow rates in pcu/h) determined by inspecting the effective *approach* lane use characteristics of the flows that constitute the circulating stream. In the example discussed above (using the total circulating flow, q_c = 900 pcu/h, we find f_{qc} = 0.0007 x 900 – 0.29 = 0.340 for the single-lane stream case, and f_{qc} = 0.00035 x 900 – 0.08 = 0.235 for the two-lane stream case. The average value to be used in *Equation (11c)* is then calculated as f_{qc} = (600 x 0.340 + 300 x 0.235) / 900 = 0.305.

The factor f_{od} decreases (therefore the entry capacity decreases) as the proportion of the total circulating stream flow that originated from and were queued on the *dominant approach* increases. The amount of reduction also increases with increasing flow levels (in the range 4 per cent at low flows to 55 per cent at high flows). This method is particularly useful for analysing the cases of unbalanced flow patterns and heavy flow levels. *Equations (11c) to (11e)* were calibrated using results from MODELC *(1,5,11,12)*.

Example

Consider the roundabout shown in *Figure 1* but with single-lane approaches and circulating roads. The basic gap-acceptance capacity and minimum capacity of the South approach entry lane are Q_g = 650 veh/h and Q_m = 60 veh/h. Total circulating flow is q_c = 1200 pcu/h which consists of through flow from the West approach (900 pcu/h) and left-turn flow from the North approach (300 pcu/h). Thus, the proportions of the total circulating flow that originated from the West and North approaches are p_{cW} = 900 / 1200 = 0.75 and p_{cN} = 300 / 1200 = 0.25. The proportion queued on the West and North approach lanes are p_{qW} = 0.80 and p_{qN} = 0.70. Since ($p_{qW}\ p_{cW}$) = 0.80 x 0.75 = 0.60 and ($p_{qN}\ p_{cN}$) = 0.70 x 0.25 = 0.175, the dominant approach is the East leg, ($p_{qd}\ p_{cd}$) = 0.60. Since q_c =1200, *Equation (11d)* gives f_{qc} = 0.55 (single-lane stream) and *Equation (11c)* gives f_{od} = 1 – 0.55 x 0.60 = 0.67. Therefore, the capacity of the South leg is found from *Equation (11)* as Q_e = *max* (0.67 x 650, 60) = 436 veh/h.

Performance Estimates

The average stop-line (control) delay calculated from *Equation (1)* does not include the *geometric delays*. For detailed discussions of the subject of geometric delay and various components of the stop-line (control) delay (queueing delay, queue move-up delay, stopped delay, etc.), refer to the SIDRA User Guide *(3)* and a more recent publication *(14)*.

Comparisons of the estimated vs simulated values of capacity, average stop-line (control) delay, average back of queue and proportion queued are shown in *Figures 2 to 5*. Simulated values were obtained from the MODELC program *(1, 5, 11, 12)*.

Back of Queue vs Cycle-Average Queue

The difference between the *cycle-average queue length* and the *back of queue* is emphasised. Traditional gap-acceptance and queueing theory models predict the cycle-average queue length whereas the average back of queue is a more useful statistic relevant to short lane capacities and to the blocking of upstream intersections. The models presented in this paper fill the gap in modelling queue length for roundabouts.

The *90th, 95th and 98th percentile* queue lengths are useful for the design of queue spaces (turn slots, parking bans, etc.). A *percentile* queue length is a value *below which* the specified percentage of the average queue length values observed for individual cycles fall. For example, the 95th percentile queue length is the value below which 95 per cent of all observed cycle queue lengths fall. Note that percentile queue lengths ($N_{bp\%}$, $N_{cp\%}$) calculated from *Equations (6) and (7)* are time-dependent although the factors $f_{bp\%}$ and $f_{cp\%}$ are independent of the flow period. This assumes that the flow period is long enough for the random effects to be valid. Therefore, the method should not be used for very short flow periods ($T_f \geq 15$ minutes is recommended).

The cycle-average queue length incorporates all queue states including zero queues. The back of queue is more relevant to the design of appropriate queueing space. The back of queue is also relevant to the prediction of such statistics as the queue clearance time and proportion queued, is used in modelling short lane capacities. It is recommended that the back of queue is used for all practical purposes. The cycle-average queue is for academic interest, and is useful for comparing the back of queue estimates with cycle-average queue length estimates from other methods such as the Highway Capacity Manual Chapter 10 *(15)* and the AUSTROADS *(16)* methods.

A comparison of the simulated values of average back of queue and cycle average queue (each point with the same entry and circulating flow characteristics) is shown in *Figure 6*. It is seen that these two types of queue length are very different. A particular case when the difference is large occurs when the entry flow rate is very high and the circulating flow rate is very low. This is characterised by a short red time and a large green time. In this case, the back of queue is large due to high entry lane demand flow rate, whereas the cycle-average queue length is small as is the case for average stop-line (control) delay.

Initial Queued Demand

The first terms of the performance formulae, as well as the method for estimating capacity must reflect the effect of *initial queued demand.* For example, entry lane capacity will be lower if there is an initial queued demand on an approach lane that contributes to the relevant circulating stream. This effect is approximated through the use of an adjusted entry lane demand flow rate, q_{ei}, is calculated from:

$$q_{ei} = q_e + N_i / T_f \qquad (18)$$

The adjusted entry lane demand flow rate is used to calculate the flow ratio, y, and an effective degree of saturation, x', as seen in *Equations (13a) and (13c).*

The overflow delay formula (*Equation 1b*) is based on the definition of stop-line (control) delay as the average delay to vehicles *arriving* during the current flow period (see *Figure 7*). Various parameters shown in *Figure 7* can be calculated from the following formulae.

Residual queued demand at the end of the flow period (vehicles):

$$N_j \;=\; \mathit{min}\ [0, N_i + (q_e - Q_e)\ T_f\ ] \qquad (19a)$$

Time for the initial queued demand to clear (hours):

$$T_i \;=\; N_i \,/\, Q_e \qquad (19b)$$

Time for the residual queued demand to clear (hours):

$$T_j \;=\; N_j \,/\, Q_e \qquad (19c)$$

Duration of oversaturation, i.e. the time for the total demand during the current flow period to clear (hours):

$$T_o = \begin{cases} \dfrac{N_i}{Q_e - q_e} & \text{for } q_a < Q_e \\ \mathit{indefinite} & \text{for } q_a \geq Q_e \end{cases} \qquad (19d)$$

Equations (19c) and (19d) assume that capacity of the current flow period is valid after the current flow period until the residual queued demand clears. Therefore, T_o does not necessarily represent the actual duration of oversaturation as it needs to be revised during the calculations for the next flow period using the capacity calculated for that flow period.

CASE STUDY

One of the real-life cases studied during the development of a method that allows for the effects of origin-destination pattern and queueing characteristics of approach flows on the entry flow capacities was described in detail in previous publications *(1,2)*. Another real-life case study is presented here. This is the intersection of Parkes Way, Kings Avenue and Moreshead Drive, which is a large roundabout in Canberra, Australia's capital city. The intersection geometry and morning peak traffic flows are shown in *Figures 8 and 9*. Note that traffic drives on the left-hand side of the road in Australia. The circulating flow values in *Figure 9* (as calculated by SIDRA) include the effect of capacity constraint due to oversaturation on the Southeast leg.

The *Before* case represents the operation of this roundabout with two lanes for the through movement on the Moreshead Drive (Southeast) approach. There is only a single exit lane on the Parkes Way approach. Accidents had occurred when two vehicles side by side from Moreshead Drive tried to exit into Parkes Way (Nortwest leg). To prevent this problem, it was proposed to linemark the Moreshead Drive approach allowing for one through lane only (the *After* case). Predictions using the SR 45 / AUSTROADS method *(16, 17)* which was implemented in an earlier version of SIDRA indicated that the modified design would work satisfactorily. The scheme was implemented, but contrary to the predictions, "queues up to 3 km long" and long delays were observed on Moreshead Drive. The method presented in this paper predicts this oversaturation case well. The predictions for the *Before* and *After* cases using the two methods are summarised in *Table 2*. The delays in *Table 2* do not include geometric delays. The queue length estimate from SIDRA 5 (95th percentile back of queue) represents the longest queue in any lane.

This case presents a problem of unbalanced flow caused by the heavy right-turn flow from Parkes Way (950 veh/h) which operates effectively as a single-lane stream. This movement was observed to operate at capacity which is predicted accurately by the SIDRA 5 method. This dominant flow reduces the capacity of the Moreshead Drive approach, causing extensive queueing and long delays in the through lane (single lane) in the *After* case. The SR 45 / AUSTROADS method fails to indicate that there is a problem at this intersection in the *After* case. It also underestimates delays and degrees of saturation in the *Before* case failing to predict that the right-turn movement from Parkes Way operates at capacity. Note that, for this example, the intra-bunch headway, critical gap and follow-up headways are generally higher for this example using the method presented in this paper. For detailed discussions on the prediction of these parameters, refer to previous publications *(1, 2, 3)*.

CONCLUDING REMARKS

The method presented in this paper is based on analytical modelling of roundabout entry lane capacity and performance measures in accordance with the gap acceptance process that takes place in real-life roundabout operation. However, this is not a simple gap acceptance process as assumed in past studies of roundabouts as a series of T-junctions. The modelling of interactions amongst approach flows is important, especially in heavy and unbalanced demand cases as demonstrated through simulation studies and analysis of real-life cases as reported in this paper and previous publications *(1,2)*.

The method reported in this paper also differs from the more commonly used models based on simple gap-acceptance and queueing theory approaches. The capacity and performance models given in this paper make use of the *overflow queue* concept and signal analogy as discussed in previous papers *(7,8)*. Recently, a comprehensive capacity and performance survey method was developed for signalised intersections using the concepts of the models given in this paper. The method was implemented at an intersection in Melbourne successfully. A demonstration task was undertaken applying the same survey method to a gap-acceptance case using arrival and departure time data generated by simulation, with results supporting the validity of the signal analogy concept. Details of the survey method will be published elsewhere.

Another real-life case studied during the development of the method described in this paper was the intersection of Fitzsimons Lane and Porter Street in Melbourne. This was a two-lane roundabout with very long delays on several approaches during peak periods. This intersection was redesigned as a three-lane roundabout after extensive evaluation of alternative design options using SIDRA. Extensive field surveys were carried out to measure delays under old and new design conditions. The new design was found to reduce the delays to satisfactory operating levels. The method presented in this report has been found to predict the operating conditions for the old and new roundabout design satisfactorily. The SIDRA evaluation and survey results for this case will be published elsewhere.

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Table 1

Parameter values for estimating the proportion of free (unbunched) vehicles in a traffic stream

	Δ	b	φ
Single-lane circulating stream	$\Delta_c = 2.0$	2.5	$\varphi_c = e^{-5.0\, q_c}$
Multi-lane circulating stream	$\Delta_c = 1.2$	2.5	$\varphi_c = e^{-3.0\, q_c}$
Approach entry lane	$\Delta_e = 1.5$	0.6	$\varphi_e = e^{-0.9\, q_e}$

Table 2

Results for the BEFORE and AFTER cases for the example shown in Figures 8 and 9

		BEFORE		*AFTER*	
		SR 45 / AUSTROADS *(16, 17)*	SIDRA 5 *(3)*	SR 45 / AUSTROADS *(16, 17)*	SIDRA 5 *(3)*
Delay (sec)	SE_Through	2.3	44.5	2.6	**638.5**
	SE_Right turn	1.8	45.5	1.2	7.3
	NW_Through	9.1	38.4	9.1	28.6
	NW_Right turn	6.7	51.1	6.7	33.5
Degree of saturation	SE_Through	0.361	0.923	0.555	**1.337**
	SE_Right turn	0.361	0.923	0.037	0.090
	NW_Through	0.782	0.986	0.782	0.967
	NW_Right turn	0.790	1.004	0.790	0.985
95% Back of queue (m)	SE_Through	–	224	–	**2013**
	SE_Right turn	–	224	–	4
	NW_Through	–	210	–	174
	NW_Right turn	–	313	–	243

SE: Moreshead Drive, NW: Kings Avenue

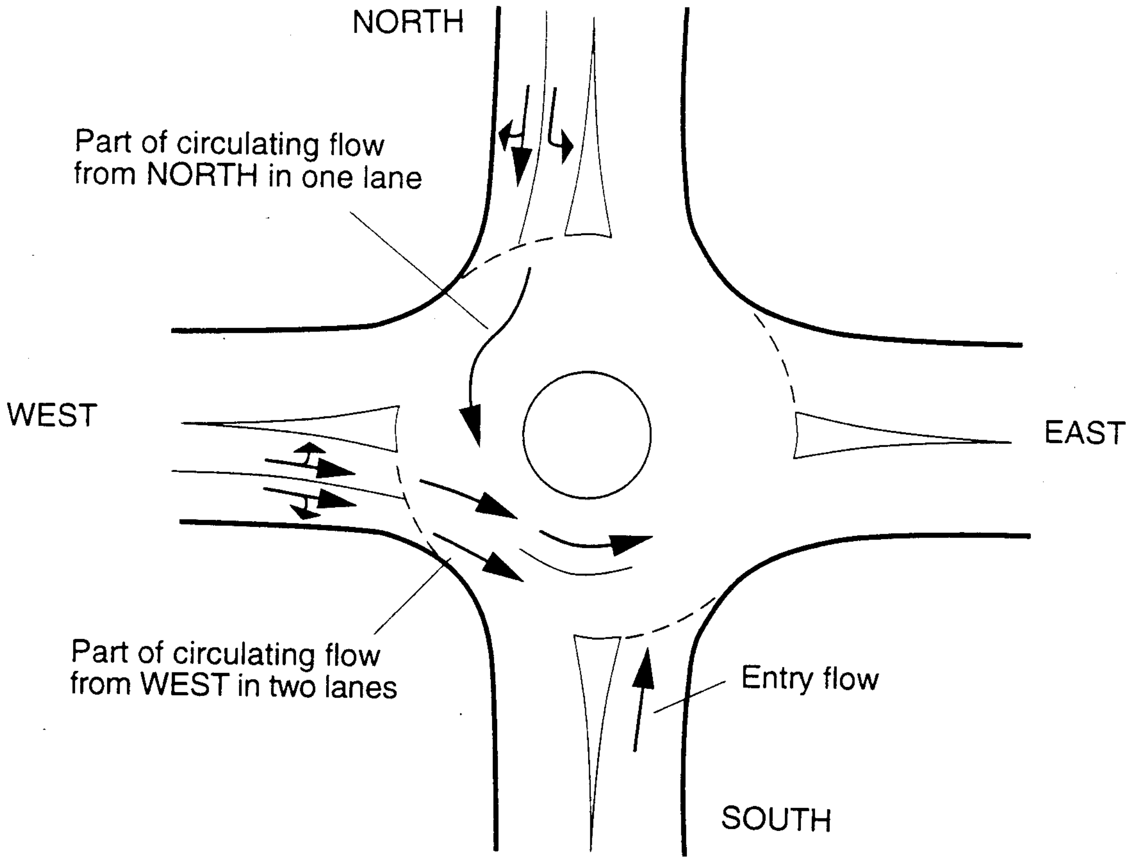

Fig. 1 – Approach lane use effect on circulating stream characteristics (an example)

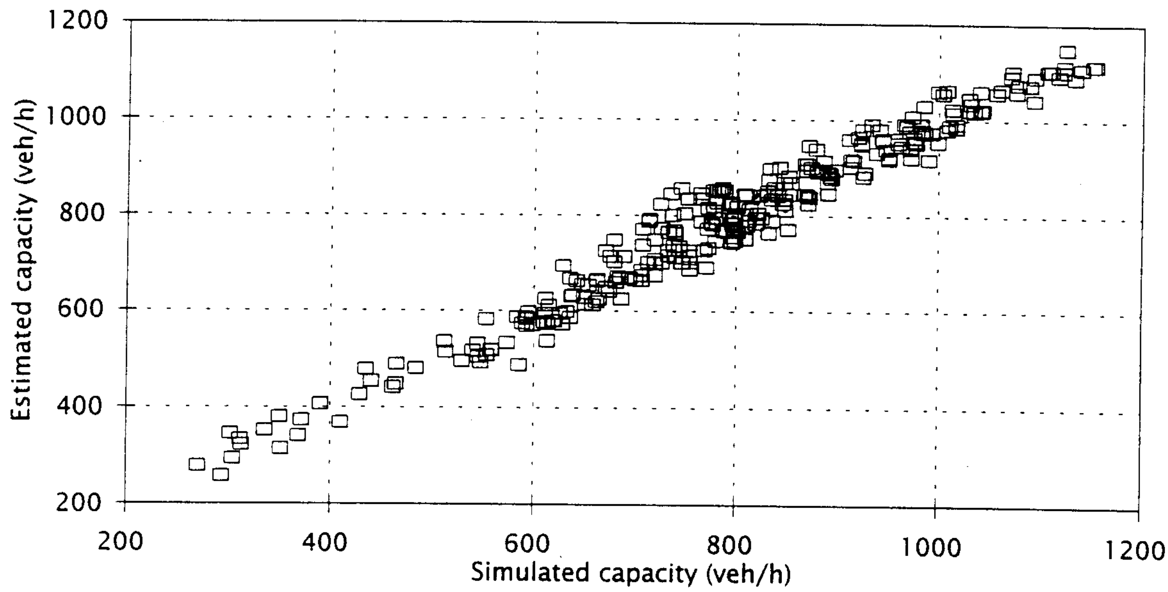

Fig. 2 – Estimated entry lane capacity vs simulated capacity

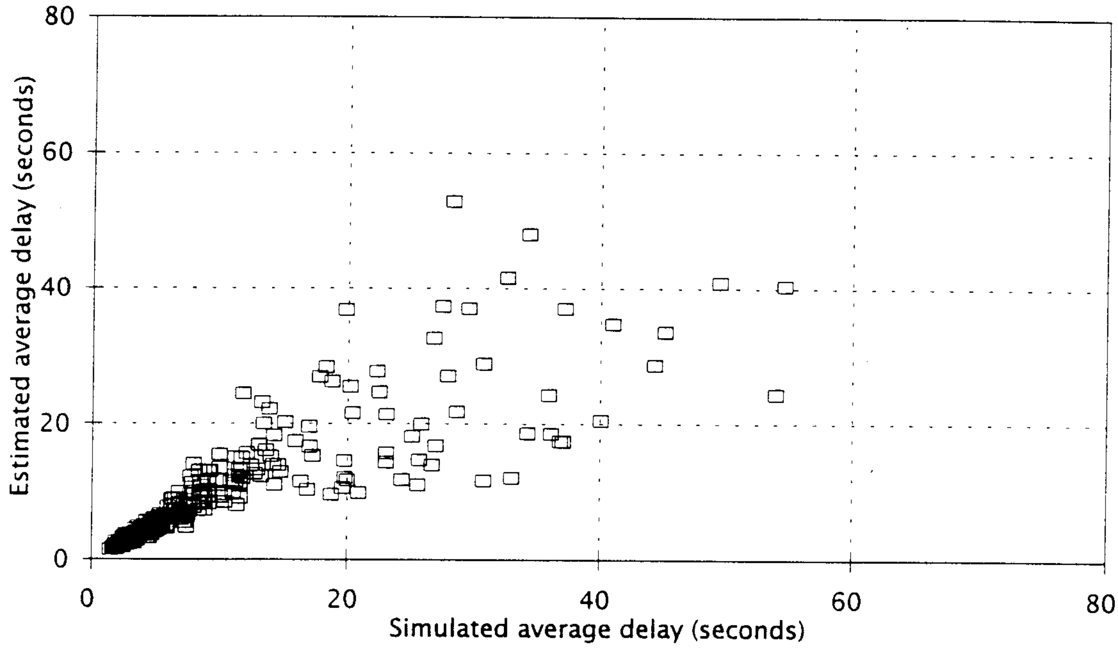

Fig. 3 – Estimated average delay vs simulated delay

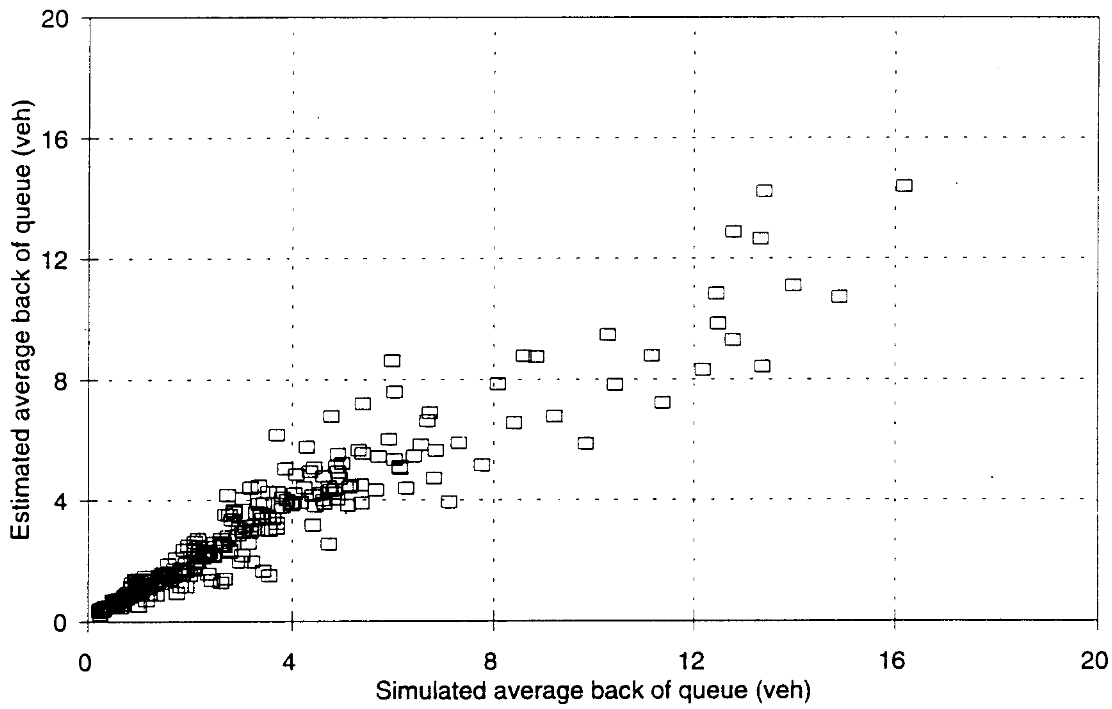

Fig. 4 – Estimated average back of queue vs simulated average back of queue

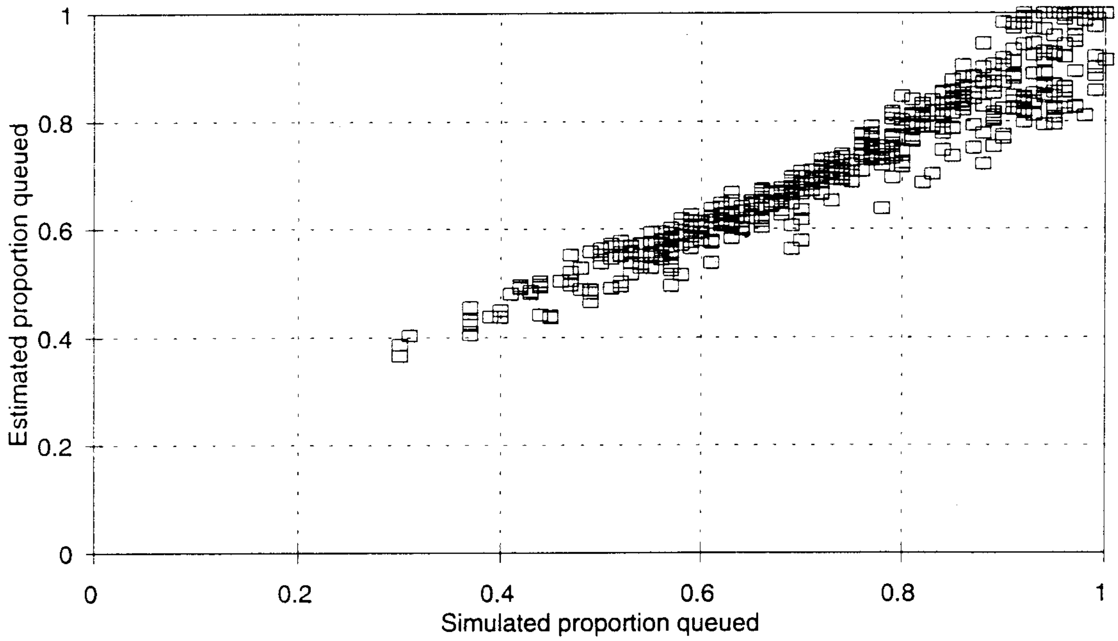

Fig. 5 – Estimated proportion queued vs simulated proportion queued

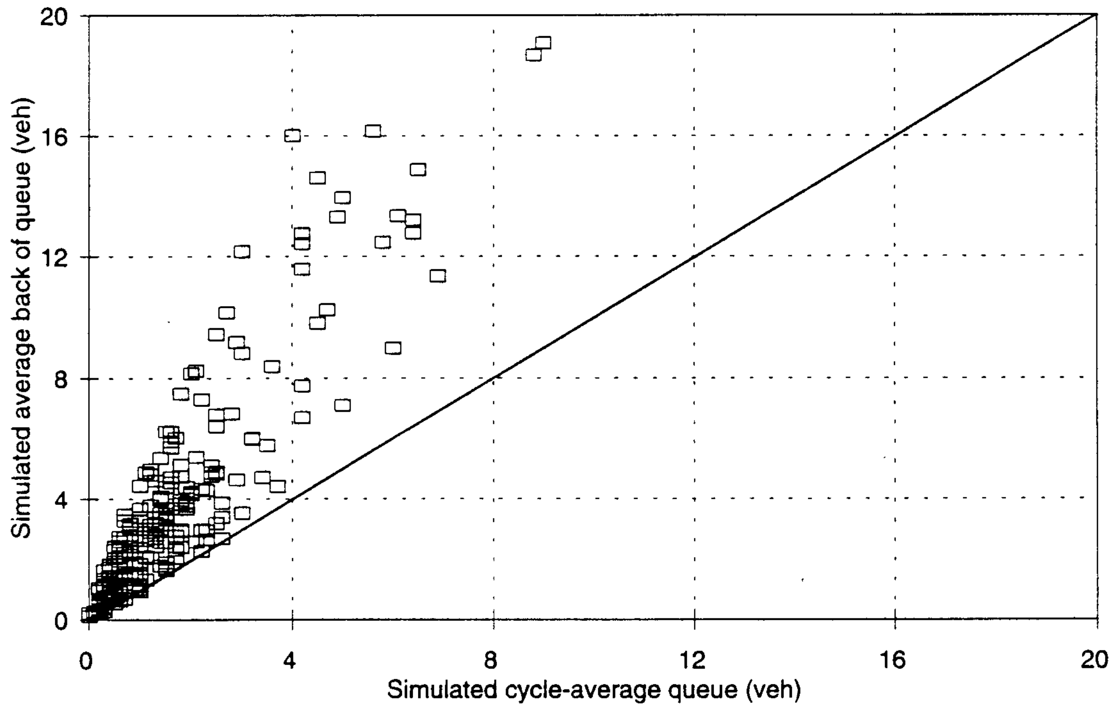

Fig. 6 – Simulated average back of queue vs simulated cycle-average queue

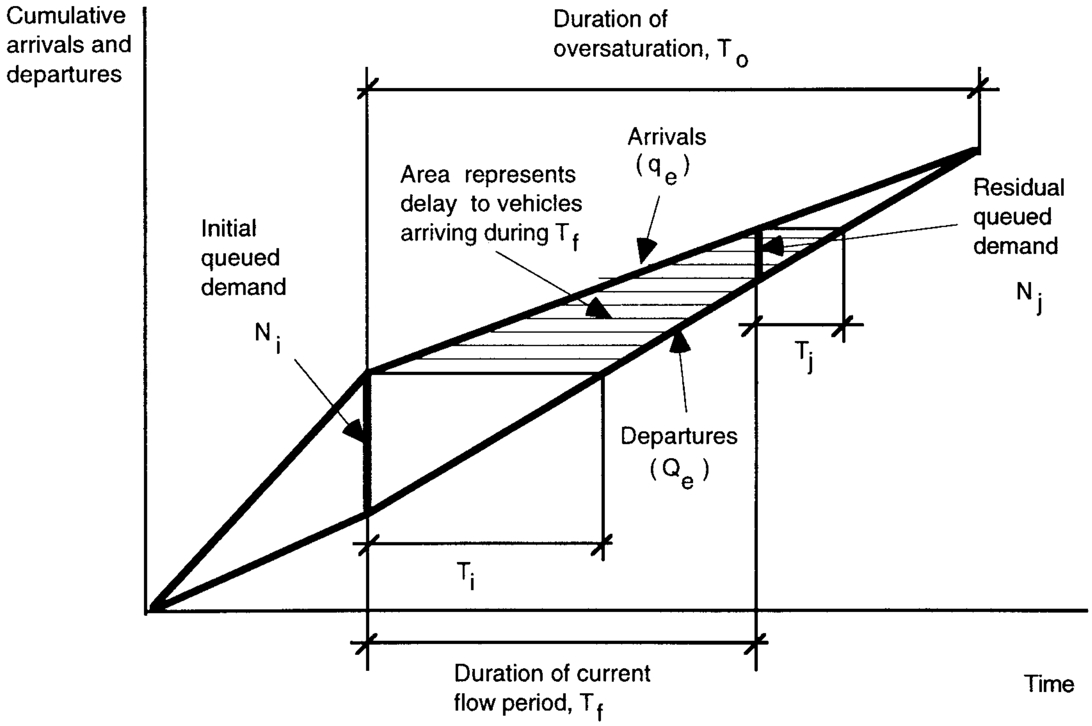

Fig. 7 – Parameters in the derivation of delay and back of queue formulae for the case with initial queued demand

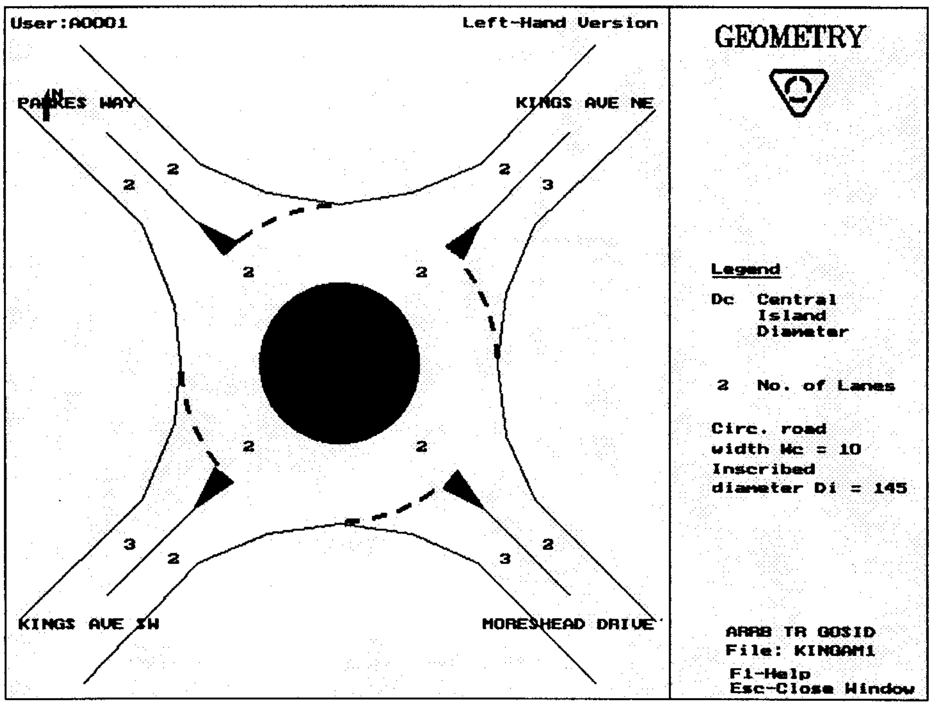

Fig. 8 – Real-life example for unbalanced flows: intersection geometry

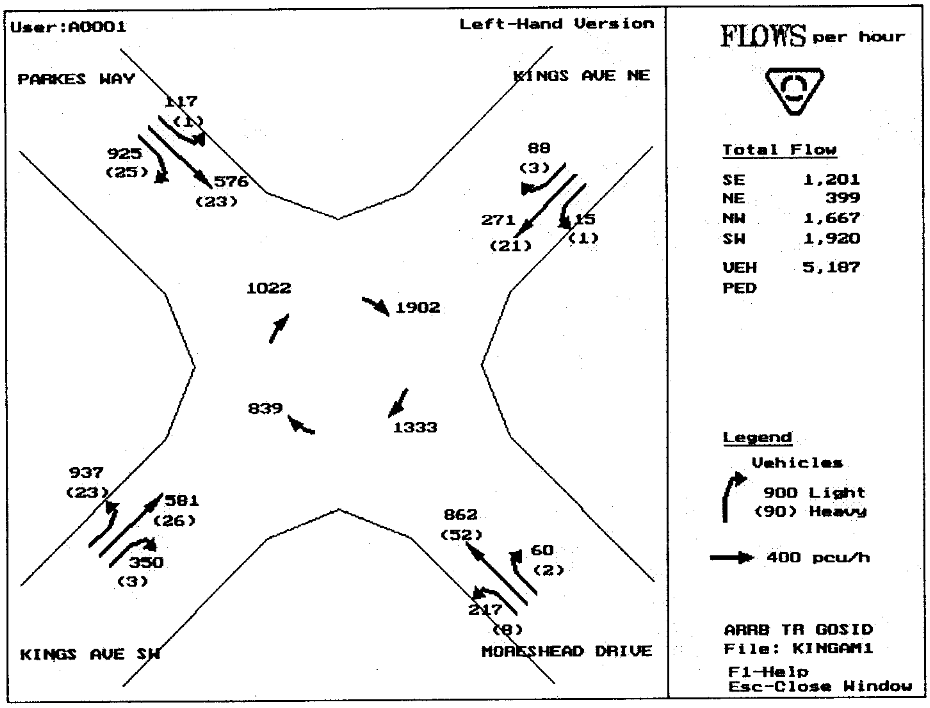

Fig. 9 – Real-life example for unbalanced flows: morning peak traffic flows

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