THE CONTROL OF HIGHWAY INTERSECTIONS, ROUNDABOUTS AND SIGNALS

R Pretty
Senior Lecturer
Department of
 Civil Engineering
University of Queensland
Queensland

R Troutbeck
Senior Lecturer
Department of
 Civil Engineering
University of Technology
Queensland

ABSTRACT

At a basic level, highway intersections are controlled by priority rules, that is one road is given priority over another. Right turning vehicles also give way to opposing traffic. Instead of numerical traffic volume warrants for installing signals or roundabouts, the paper suggests that delay, stopping and excess fuel consumption be determined for each intersection being considered for signals or roundabout control. Because fuel consumption considers delay and stopping, it should be minimised for the optimum form of control. An extension of the SIDRA program would be a convenient computing technique. In a numerical example, the excess fuel consumption for signal control was found to be relatively high. At isolated intersections this form of control should be used sparingly. Further work is required to determine the optimum extent of signal control in a network of intersections.

INTRODUCTION

Where roads in a network meet or cross, the highway authority determines the form of control. The basic form is for traffic on one road to have priority over the other. If one road terminates, the traffic on the through road normally has priority. If neither road terminates, signs or markings designate the road with priority, usually the one with the greater traffic volume. With more than two roads meeting or crossing, channelisation is usually implemented, so that the priority is decided between two roads at a time. To go with priority roads, a rule is required to determine priority between two vehicles on the same road where one or both are turning onto another road. In this case, right turning vehicles usually give way to opposing traffic.

The basic system of priority breaks down when traffic volumes become too high. This is because the gaps or time intervals between successive priority vehicles may be, on average, too short to allow sufficient non-priority vehicles to enter. In that case, long delays for the latter stream will occur leading to congestion and frustration and perhaps risk-taking by drivers in this group. To solve the problem, engineers have available roundabouts and traffic signals. Roundabouts separate the many conflicting movements at an intersection into simple T-junctions by requiring traffic to circulate around a central island.

Signals allocate priority alternately to one stream and then the other. If after one of these treatments, there is still excessive delay, the intersection approaches can be widened, so that more vehicles may proceed together. If this is impractical, grade separation is the ultimate step with one road passing over the other and with ramps between the two for turning movements.

Until now, an engineer had available a <u>warrant</u> or minimum traffic volumes before signals might be considered. Although these volumes were arbitrary, at least there was some guidance unlike the case of roundabouts. Now, following extensive research, the consequences of installing a roundabout can be determined in terms of traffic capacity, delay, excess fuel consumption and safety. The aim of this paper is to set out definitive guidelines for the use of roundabouts in a similar style to those for traffic signals.

The paper presents formulae with relevant parameters, and graphs and tables to enable the capacity and delay of a roundabout to be calculated for any intersection geometrical characteristics and traffic volumes. The formulae use gap acceptance parameters and are backed by extensive field

studies. From delay and stops, excess fuel consumption can be predicted. For the same intersection, the same results can be obtained for signals using SIDRA (Akçelik, 1987). Both procedures can be run on a personal computer, or both can be adapted to a pocket calculator by using an earlier Australian Road Research Board procedure for signals.

WARRANTS FOR SIGNALS AND ROUNDABOUTS

Highway authorities are probably familiar with the concept of a 'warrant' for a particular type of control. In Queensland, for example (Main Roads Department 1979), a warrant is the set of ..... conditions (established by long experience) and "may comprise quantitative figures or other general requirements."

The use of warrants is to ensure that

(a) control devices are installed where the need has been proven, and only in such conditions;

(b) the most efficient treatment is provided for any given set of conditions; and

(c) standard treatment is employed in similar situations.

The document concedes that some elements which may justify a particular device may be incapable of being expressed in quantitative terms. Nevertheless there is an attempt to write down quantitative traffic volumes to justify installing traffic signals. There is no justification for roundabouts.

The Manual in Queensland puts stop and give way signs as the basic form of intersection control in that stop or give way signs are used where the warrant for traffic signals is not satisfied. Quantitative traffic volumes are then given for signal installation.

They are based on vehicular volumes on the intersecting roads - a minimum of 600 veh/h on the major road and 200 veh/h on the highest-volume approach on the minor road for each of any four hours of an average day. There are also alternative warrants such that the satisfaction of any one may justify the installation of signals. These alternatives are:

(a) interruption of continuous traffic whereby there may be a hazard to minor road traffic, in this case the volumes become 900 veh/h and 100 veh/h respectively;

(b)   a pedestrian volume with 150 pedestrians/hour replacing the 200 veh/h on the minor road in the original warrant;

(c)   accidents coming to 3 or more reported casualty accidents over 3 years together with only 80 percent of the relevant volume warrants.

The Manual concedes that warrants cannot be used alone but only to separate cases into likely effective and ineffective sites. Cost benefit analysis is recommended.

For roundabouts, the appropriate design guide (NAASRA 1986) is even more circumspect. It is stated that there are so many factors needing to be considered that it is not possible to specify whether a roundabout should or should not be installed. Nevertheless a list of situations where a roundabout may or may not be appropriate is given. The most important of these was from an evaluation of delay. Roundabouts are favoured when they reduce delays compared with the base system of priority rule control and even more than from signalisation. It was then pointed out a roundabout could be particularly appropriate where there are high proportions of right turning traffic, where it was desirable to reduce speeds or where the geometry of the intersection led to difficulties in defining priority or signal phasing. The inappropriate sites included places where a limited signal system would give better service or where different operation was required in the peak period compared with off-peak.

The warrants therefore are incomplete and lack detail. Surely it would be better practice to write down the consequences in terms of each type of intersection control. These include the capacity of a movement to enter or cross the intersection, the delay and the amount of stopping. A secondary consequence is the amount of excess fuel consumption. In the next section, the consequences of priority rule operation are given. This is followed by sections on signals and roundabouts.

PRIORITY RULE OPERATION OF INTERSECTIONS

A simple model for capacity, delay and stopping is that proposed by Troutbeck (1989). It is assumed that minor stream drivers consider all major stream drivers and vehicles to be identical. For convenience the major stream can be divided into two or more streams representing for example each direction of flow. It is also assumed that minor stream drivers are consistent and homogeneous and will cross if the time gap between the major stream vehicles is greater than a critical acceptance gap T. Several

326

minor stream vehicles can enter the intersection at headways of T_0, the 'follow-on' time, if there is a large gap in the major stream traffic.

The headway model for the major stream chosen is also the same as that used by Troutbeck, namely Cowan's (1975) model. This has the following cumulative distribution of headways:

$$F(t) = 1 - \alpha \, e^{-\lambda(t-\Delta)} \qquad t \geq \Delta \qquad (1)$$
$$\text{and} \qquad F(t) = 0 \qquad t < \Delta$$

where α is the proportion of free vehicles (These vehicles have headway greater than Δ)

λ is a decay constant given by:

$\lambda = \alpha q /(1-\Delta q)$ $\qquad (2)$

q is the flow of vehicles (number per unit time) ($q = q_1 + q_2$), for two streams 1 and 2)

and Δ is the minimum headway in the stream.

Then Troutbeck showed that the entry capacity for two lanes is,

$$Q = \frac{(q_1 + q_2) \, \alpha' \, e^{-\lambda' (T-\Delta)}}{1 - e^{-\lambda' T_0}}$$

$\qquad (3)$

where

$$\alpha' = \frac{\alpha_1 \, q_1 \, (1-\Delta q_2) + \alpha_2 \, q_2 \, (1-\Delta q_1)}{q_1 + q_2}$$

$\qquad (4)$

$$\lambda' = \lambda_1 + \lambda_2 \qquad (5)$$

Within the major streams there are usually right turning vehicles which in turn give way to opposing vehicles (Fig 1). An adjusted major stream flow is given by (Bennett 1984).

With four approaches, the work by Bennett (1984) can be extended to give

$$q_{1a} = q_1 + q_{1r} - [\ln P_{o1}]/T \qquad (6)$$

$$q_{2a} = q_2 + q_{2r} - [[\ln P_{o2}]/T \qquad (7)$$

$$q_a = q_{1r} + q_{2r} - [\ln P_{o1}]/T - [[\ln P_{o2}]/T \tag{8}$$

where $\quad P_{o2} = 1 - q_{2r}/Q_{2r}$ $\tag{9}$

Q_{2r} = capacity to turn right from stream 2

$$Q_{2r} = \frac{q_1 \, \alpha_1 \, e^{-\lambda_1(T_r - \Delta)}}{1 - e^{-\lambda_1 T_{0r}}}$$

$\tag{10}$

q_{2r} = flow turning right from stream 2

T_r = critical gap for right turning
T_{0r} = follow on headway for right turning

Similarly P_{o1}, q_{1r} and Q_{1r} can be written for stream 1. Then q_a is used in (3), (13) and (14) in place of $(q_1 + q_2)$, q_{1a} for q_1, and q_{2a} for q_2 in eqs (4), (14) and (15).
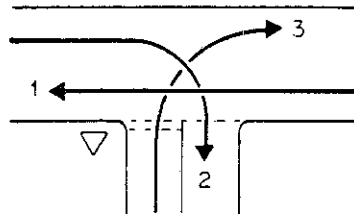

Fig.1. Three levels of priority at a T intersection where stream 2 must wait for acceptable gaps in stream 1 and where stream 3 must wait for acceptable gaps in streams 1 and 2.

The same model was used by Troutbeck to estimate the average delay D, to a minor stream, at an intersection with two major streams as

$$D = D_{min} \left[1 + \frac{x}{1 - x} \right]$$

$\tag{11}$

where

$$x = q_e/Q$$

(12)

q_e = entry flow

$$D_{min} = \frac{e^{\lambda'(T-\Delta)}}{\alpha'(q_1+q_2)} - T - \frac{1}{\lambda'} + \frac{\lambda'\Delta^2+2\alpha'\Delta-2\Delta+2\beta\Delta^2-4/3\lambda'\Delta^3\beta}{2\lambda'\Delta+2\alpha'-2\beta\Delta^2\lambda'}$$

(13)

$$\beta = \frac{q_1 q_2}{q_1 + q_2}$$

(14)

D_{min} is called Adams' delay and represents the average delay when the minor stream flow is very small. It is also the average delay to pedestrians who need not queue behind each other but may cross together.

The third consequence of an intersection is the fact that a proportion of vehicles are required to stop. For this Troutbeck (1988) gives the expression

$$P_d^* = 1 - (1-\Delta q_1)(1-\Delta q_2) e^{-(\lambda_1+\lambda_2)(T-\Delta)}$$

(15)

for minimal minor stream flow. A derived consequence is the excess fuel consumption caused by delay and stopping. For this an elemental full consumption model (Bowyer, Akcelik and Biggs 1985) can be used as shown in the Appendix.

On the matter of priority of other streams, it is assumed that pedestrians are given priority by turning vehicles. For this paper, right turning vehicles are assumed to give way to opposing, through and left-turning vehicles. If such left-turning vehicles are required to give way to the right turners (as in Victoria), the model can take this into account by identifying another level of priority.

SIGNAL OPERATION

At traffic signals priority is shared in time between the various competing streams instead of remaining with the major streams. At the end of each green period for a stream, there is a possibility of an overflow queue developing. The average overflow queue for a stream is given by (Akçelik 1981):

329

$$N_o = \frac{Q\,T_f}{4}\left\{ z + z\sqrt{\frac{12(x - x_o)}{Q\,T_f}} \right\}$$

$$N_o = 0 \qquad\qquad \begin{array}{l} \text{for } x > x_o \\ \text{otherwise} \end{array} \qquad (16)$$

where
Q = capacity in vehicles per hour = $s(g/c)$
s = saturation flow (veh/h)
g = green time (s)
c = average cycle time (s)
T_f = flow period (hours) during which the average flow rate, q, persists. (The default value for T_f is 1)
x_o = $0.67 + sg/600$ (17)
z = $x - 1$, with x from eq (6) (18)

The average delay per vehicle is

$$d \approx \frac{c(1-u)^2}{2(1-y)} + \frac{N_o\,x}{q} \qquad\qquad \text{(s)} \qquad (19)$$

where
u = g/c, the green time ratio, and
y = q/s

The total delay per stream, $D = q\,d$ (20)

The number of stops per vehicle is

$$h = 0.9\left[\frac{1-u}{1-y} + \frac{N_o}{qc}\right] \qquad\qquad (21)$$

with the number of stops per stream, $H = q\,h$. Again a secondary measure of performance is fuel consumption given by the elemental fuel consumption model.

The signals would then be set to operate at an average cycle time, c_o, optimised according to Akçelik (1981)

$$c_o = \frac{1.6\,L + 6}{1 - Y} \qquad\qquad \text{(s)} \qquad (22)$$

where
L = intersection lost time in seconds

330

$L = \Sigma\ l$, the lost times for the critical movements (the movements determining the timing requirements).

Y = intersection flow ratio, the sum of the ratios (q/s) for the critical movements.

In this form, the cycle time is set to approximate minimum user cost. For the critical movements, the green times are distributed according to

$$g = \frac{c - L\,u}{U}$$

(23)

where u and U are the movement and intersection green time ratios for the critical movements ($U = \Sigma\ u$). A movement is defined as a separate queue leading to the intersection, characterised by its direction, lane allocation and right-of-way provision.

Pedestrians constitute a movement and may influence timings because of the need to provide a safe street crossing time. Akçelik (1981) suggests (5 + $D_s/1.4$) s; where D_s is the street width in metres.

ROUNDABOUT OPERATION

As for priority rule and signal operation, the design traffic volumes should be distributed to the entry lanes. The number of entry lanes will depend on the width of the approaching roadways. The average lane width is also required. Drivers turning right should be in the right hand lane, if there are two and those turning left will be in the left lane. Through vehicles could be in either lane (to minimise their journey times). The circulating volume for a roundabout is found by adding the volume from the movements passing each entrance. Pedestrians are assumed to cross from the kerb side to the splitter island without interrupting entering vehicles.

The number of circulating lanes is required even though these may not be marked on the roadway. The number is a function of the circulating road width i.e. $n_c = 1$ for cw < 10 ; $n_c = 2$ for 10 < cw < 15; $n_c = 3$ for 15 < cw (cw in metres).

The first parameter required is the inscribed diameter in metres (Figure 2). This is twice the radius of the largest arc that can be drawn inside the kerb line of the roundabout. The diameter may be different for each entry of a non-circular roundabout

331

D = Inscribed diameter (m)

cw = circulating roadway width (m)

e = entry width (m)

r_e = entry radius

ecw = extra circulating roadway width
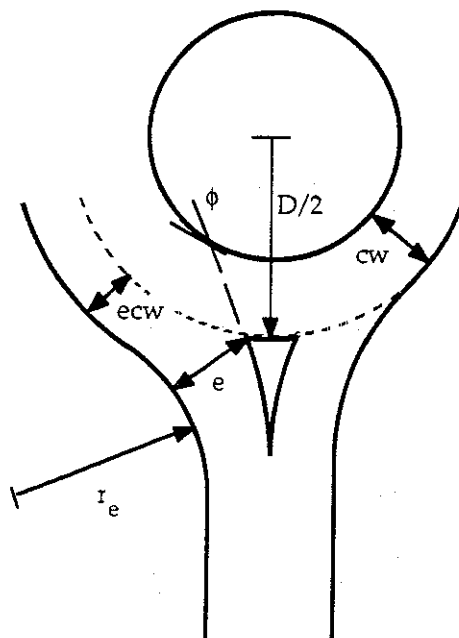
ϕ = conflict angle

Fig.2. Description of the roundabout entry terms

Drivers in different entry lanes behave differently. Generally there is one lane in which drivers tend to dominate. These drivers enter the roundabout with less regard to those in other entry lanes of the same approach. Conversely drivers in other entry lanes watch the circulating vehicles' manoeuvres and those of drivers in the dominant stream

Knowing the diameter, Table I is entered to find the dominant entry stream follow-on time adjusted according to Table II.

332

Table I   Dominant stream follow-on times (Initial values)

Inscribed diameter (m)	Circulating flow (veh/h)						
	0	500	1000	1500	2000	2500	3000
20	2.99	2.79	2.60	2.40	2.20	2.00	1.81
25	2.91	2.71	2.51	2.31	2.12	1.92	1.72
30	2.83	2.63	2.43	2.24	2.04	1.84	1.64
35	2.75	2.55	2.36	2.16	1.96	1.77	1.57
40	2.68	2.48	2.29	2.09	1.89	1.70	1.50
45	2.61	2.42	2.22	2.02	1.83	1.63	1.43
50	2.55	2.36	2.16	1.96	1.76	1.57	1.37
55	2.49	2.30	2.10	1.90	1.71	1.51	1.31
60	2.44	2.25	2.05	1.85	1.65	1.46	1.26
65	2.39	2.20	2.00	1.80	1.61	1.41	1.21
70	2.35	2.15	1.96	1.76	1.56	1.36	1.17
75	2.31	2.11	1.92	1.72	1.52	1.33	1.13
80	2.27	2.08	1.88	1.68	1.49	1.29	1.09
85	2.24	2.05	1.85	1.65	1.46	1.26	1.06
90	2.22	2.02	1.82	1.63	1.43	1.23	1.04
95	2.20	2.00	1.80	1.61	1.41	1.21	1.01
100	2.18	1.98	1.79	1.59	1.39	1.19	1.00

Source: Troutbeck (1989)

Table II   Adjustment factors for the dominant stream follow-on time

Number of circulating lanes	Number of entry lanes		
	1	2	3
1	0.00	0.39	—
2	-0.39	0.00	0.39
3	—	-0.39	0.00

Note: Add or subtract these factors from the initial values from Table II

Source: Troutbeck (1989)

The dominant stream is the one with the greatest flow after allocation of the through traffic to appropriate lanes. Compared with other streams (sub-dominant), the ratio of the flows, dominant to sub-dominant affects the sub-dominant stream follow-on times. These are obtained from Table III.

Table III Sub-dominant stream follow-on times

Dominant stream follow-on time (s)	Ratio of flows Dominant flow/Sub-dominant flow				
	1.0	1.5	2.0	2.5	3.0
1.5	2.05	1.99	1.94	1.89	1.84
1.6	2.10	2.07	2.05	2.02	1.99
1.7	2.15	2.15	2.15	2.15	2.15
1.8	2.20	2.23	2.25	2.28	2.30
1.9	2.25	2.30	2.35	2.40	2.46
2.0	2.30	2.38	2.46	2.53	2.61
2.1	2.35	2.46	2.56	2.66	2.76
2.2	2.41	2.53	2.66	2.79	2.92
2.3	2.46	2.61	2.76	2.92	3.07
2.4	2.51	2.69	2.87	3.05	3.23
2.5	2.56	2.76	2.97	3.17	3.38
2.6	2.61	2.84	3.07	3.30	3.53
2.7	2.70	2.92	3.17	3.43	3.69
2.8	2.80	3.00	3.28	3.56	3.84
2.9	2.90	3.07	3.38	3.69	4.00
3.0	3.00	3.15	3.48	3.82	4.15

The critical acceptance gaps for the dominant and sub-dominant streams are obtained from Table IV. They are a function of the average entry lane width, the number of circulating lanes, the circulating flow and the follow-on times.

The intra-bunch headway, Δ is set to 1 s for multi-lane circulating sections and to 2 s for single circulating lane roundabouts. The proportion of free vehicles is then obtained from Table V. The values may need adjustment if there is platooning from nearby intersections.

With all the parameters obtained, similar formulae to those used for priority rule operation are applied, by setting q to q_1 and $q_2 = 0$. In the equations that result, T and T_0 take values appropriate to the dominant and sub-dominant streams.

The average extra delay incurred by the geometry of the roundabout is

$$D_{geom} = d_{decel} + d_{neg} + d_{accel} \qquad (27)$$

where D_{geom} = geometric delay

d_{decel} = delay occurring when decelerating from the approach speed to the negotiation speed.

d_{neg} = delay occurring when travelling around the circulating lanes at the negotiation speed.

d_{accel} = delay occurring when accelerating from the negotiation speed to the departure speed.

The stopped delay is less than the gap acceptance delay by the time taken for the deceleration and acceleration of an entering vehicle and the time lag between the passage of a circulating vehicle and the next entering vehicle. If τ is the amount of the time lag

$$d_{stop} = \Delta - \tau \tag{49}$$

where d_{stop} is the average stopped time of all vehicles on an approach. As further research is required to establish appropriate relationships for D_{geom} and τ, this paper will calculate excess fuel consumption from D and P_d^*.

Table IV Ratio of the critical acceptance gap to the follow-on time

Average entry lane width (m)	3	4	5	3	4	5
Number of circulating lanes	1	1	1	2	2	2
Circulating flow (veh/h)						
0	2.32	1.98	1.64	2.04	1.70	1.36
200	2.26	1.92	1.58	1.98	1.64	1.30
400	2.19	1.85	1.52	1.92	1.58	1.24
600	2.13	1.79	1.45	1.85	1.51	1.18
800	2.07	1.73	1.39	1.79	1.45	1.11
1000	2.01	1.67	1.33	1.73	1.39	1.10
1200	1.94	1.60	1.26	1.67	1.33	1.10
1400	1.88	1.54	1.20	1.60	1.26	1.10
1600	1.82	1.48	1.14	1.54	1.20	1.10
1800	1.75	1.42	1.10	1.48	1.14	1.10
2000	1.69	1.35	1.10	1.41	1.10	1.10
2200	1.63	1.29	1.10	1.35	1.10	1.10
2400	1.57	1.23	1.10	1.29	1.10	1.10
2600	1.50	1.16	1.10	1.23	1.10	1.10
2800	1.44	1.10	1.10	1.16	1.10	1.10
3000	1.38	1.10	1.10	1.10	1.10	1.10

Table V   Proportion of free vehicles, α

No. of circulating lanes	one	more than one
Intra-bunch headway (s)	2.0	1.0
Flow (veh/h)		
0	0.8	0.8
400	0.6	0.7
800	0.4	0.6
1200	0.2	0.5
1600		0.4
2000		0.3
2400		0.2

LAND RESUMPTION

In comparing all three methods of control, the one certain way of increasing capacity and reducing delay is to widen the entrances to the intersection. In the case of an intersection under priority rules, widening will permit simultaneous entry of side-street traffic. For signals more vehicles can proceed at once on a green signal providing there is room to exit the intersection. For a roundabout, the number of sub-dominant streams can increase, along with the circulating roadway width, to increase the capacity.

In this paper, we will consider only those situations in which land resumption should not be necessary.

NUMERICAL EXAMPLE

An example is taken from NAASRA (1986). The arriving flows are shown in Figure 3.

In the Appendix, the capacity, average delay and fuel consumption are evaluated for the three types of control: priority rule, signal and roundabout. The results are summarised in Tables VI, VII and VIII below.

336
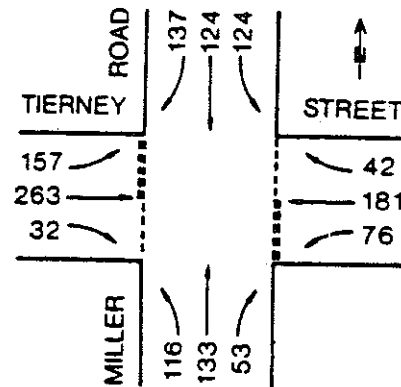
Fig.3 Arrival flows for the numerical example

Table VI Delays, stops and fuel consumption for test intersection under Priority Rules

Appr.	Flows	Av. delay (s)	Prob. delay	Stops/h	Total delay (veh.h/h)	Excess fuel cons. (L/h)
1	137	1.16	0.272	37	0.044	neglig
2	55	1.07	0.270	15	0.016	neglig
3	299	3.30	0.735	219	0.274	5.04
4	452	8.57	0.735	332	1.076	7.61
Total				603	1.410	12.65

337

Table VII Delays, stops and fuel consumption for test intersection under Signal Control

Appr.	Flow (veh/h)	Sat flow (veh/h)	y	u	Total delay D (veh.h/h)	Total stops (H/h)	Excess fuel cons. (L/h)
1	385	1980	0.19	0.38	1.066	265	7.0
2	299	2650	0.11	0.38	0.753	87	4.9
3	302	2450	0.12	0.38	0.770	191	5.0
4	452	2890	0.16	0.38	1.207	300	7.9
Total					3.796	943	24.8

Table VIII Delays, Stops and fuel consumption for test intersection as a Roundabout

Appr.	Circ flow q (veh/h)	λ (veh/h)	Entry capac. Q (veh/h)	Entry vol (veh/h)	Min. delay Dmin (s)	Av. delay (s)
1	360	315	913	302	3.72	5.56
2	228	183	853	452	3.01	6.40
3	348	302	927	385	3.65	6.24
4	293	245	989	299	3.33	4.77

Table VIII continued

Appr.	Prob. delay	Stops/h	Total Delay (veh.h/h)	Excess Fuel Consumption (L/h)
1	0.390	118	0.466	3.21
2	0.254	115	0.804	4.00
3	0.378	145	0.667	4.04
4	0.322	96	0.393	2.92
Total			2.330	14.17

338

DISCUSSION OF RESULTS

Although this example is taken from NAASRA (1986) and so should be favourable for the roundabout case, in fact the total delay and excess fuel consumption are slightly greater than in the priority rule case. Thus on the basis of delay and fuel consumption, the intersection would perform better under priority rules than under roundabout or signal control. This points to the desirability of having available rigorous methods to compare the three forms of control.

Even with the technique described for roundabout control, there is the minor problem of how to estimate the excess fuel consumption due to extra travel around the central island and how to calculate geometric delay from acceleration, deceleration and negotiation times at the roundabout. Some more field work is required.

CONCLUSIONS, FINDINGS AND RECOMMENDATIONS

This paper has used theoretical methods to derive delay, stopping and excess fuel consumption for highway intersections under control by priority rules, roundabouts or signals. It is suggested that, as excess fuel consumption takes into account delay and stopping, a reasonable decision on the form of control is to use the form which minimised excess fuel consumption. This criterion is subject to land resumption and safety considerations. The warrant for installation of a particular form of control should be the combination of minimum traffic volumes to produce the least excess fuel consumption. However, the present simple volume warrant for signals is not satisfactory. The theoretical relationships should be incorporated into an extension of a computer programs such as SIDRA or INSECT (Tudge 1988) This will enable the user to find the optimum control for any given demand

It has been shown that the excess fuel consumption for signal control can be quite high. In this connection, it must be remembered that signals are often installed in networks. With co-ordinated signal control, delay and stopping may be less than for an isolated intersection. An important research task related to the theoretical work reported here is to determine the appropriate boundary for a signal network.

Further work is required, to establish improved techniques to compare the effects of different intersection control. This paper discusses the basis for further work and outlines a preliminary technique.

Further research is also needed to establish values for geometric delay at a roundabout together with the time lag between the passage of a circulating vehicle and the next entering vehicle.

REFERENCES

Akçelik, R (1981) Traffic Signals - Capacity and Timing Analysis. Australian Road Research Board Research Report, ARR 123.

Akçelik, R (1987) SIDRA Version 2.2 Input and Output. Australian Road Research Board Technical Manual, ATM 19

Bennett, D W (1984) Elementary traffic flow theory and applications. In *"Traffic Engineering Practice"*. K W Ogden and D W Bennett, eds. 3rd ed. Monash University, Melbourne.

Bowyer, D P, Akçelik, R and Biggs, D C (1985) Guide to Fuel Consumption Analyses. Australian Road Research Board Special Report, SR 32.

Cowan, R J (1975) Useful headway models, *Transportation Science*, Vol. 9(6), pp371-376

Main Roads Department, Queensland (1983) Manual of Uniform Traffic Control Devices. Rev. ed.

NAASRA (1986) Roundabouts - A Design Guide. National Association of Australian State Road Authorities, Sydney.

NAASRA (1988) Guide to traffic Engineering Practice, Part 5 - Intersections at Grade. National Association of Australian State Road Authorities, Sydney.

Troutbeck, R J (1988) The Performance of Roundabouts on Urban Arterial Roads. Ph.D. Thesis, Univ. of Qld.

Troutbeck, R J (1989) Evaluating the Performance of a Roundabout. Australian Road Research Board Special Report (in press).

Tudge, R (1988) INSECT - The calibration and validation of an Intersection Simulation Model. In *"Intersections without Traffic Signals"*. W. Briton (Ed) Springer Verlag

CONTROL OF HIGHWAY INTERSECTIONS

APPENDIX

The Elemental Fuel Consumption Model

Bowyer, Akçelik and Biggs (1985) proposed that the excess fuel consumption per stop in mL, f_h is given by

$$f_h = F_a + F_d - f_c\,(x_d + x_a) + 0.444\,t_i \tag{29}$$

assuming the cruise speeds are the same before and after passing through the intersection. In this equation

F_a = acceleration fuel consumption, mL

= $0.444\,t_a + [30 + 0.0075\,k_1\,v_c + 108\,E_k + 54\,k_2\,E_k^2 + 10.6\,G]\,x_a$ (30)
  or $0.444\,t_a$, whichever is larger

v_c = cruise speed, km/h

E_k = $0.3858 \times 10^{-4}\,v_c^2\,/x_a$, J/kg m (31)

k_1 = $0.616 + 0.000544\,v_c$ (32)

k_2 = $1.376 + 0.00205\,v_c$ (33)

G = per cent grade (negative downhill)

x_a = acceleration distance, km

= $m_a\,v_c\,t_a\,/3600$ (34)

m_a = $0.467 + 0.00200\,v_c$ (35)

t_a = $v_c\,/\,(2.08 + 0.12\,\sqrt{v_c})$ (36)

F_d = deceleration fuel consumption, mL

= $0.444\,t_d + [30\,k_x + 0.0075\,k_y\,k_1^2\,v_c + 108\,k_a\,E_k + 10.6\,k_x\,G]\,x_d$ (37)
  or $0.444\,t_d$, whichever is larger

k_x = $0.129 + 0.00421\,v_c + 0.0544\,G$ (38)

k_y = $k_x^{0.75}$ (39)

k_a = $k_x^{3.81}\,(2 - k_x^{3.81})$ (40)

k_1 = $0.621 + 0.000777\,v_c$ (41)

x_d = $m_d\,v_c\,t_d\,/3600$, deceleration distance (42)

m_d = $0.473 + 0.00155\,v_c$ (43)

t_d = $v_c\,/\,(1.71 + 0.238\,\sqrt{v_c})$ (44)

$$f_c = 1600/v_c + 30 + 0.0075\,v_c^2 + 108\,k_{E_1}\,E_k^+ + 1171.2\,E_k^{+2} + 10.6\,k_G\,G \tag{45}$$

k_{E_1} = $12.5/v_c + 0.000013\,v_c^2$ (46)
  or 0.63, whichever is smaller

E_k^+ = $0.258 - 0.0018\,v_c$, J/kg m (47)
  or 0.10, whichever is larger

341

$$k_G = 1 - 2.1 \, E_k^+ \text{ for } G < 0$$
$$= 1 - 0.3 \, E_k^+ \text{ for } G \geq 0$$

$$t_i = \text{stopped time, s}$$
$$= D - (1 - m_d)t_d - (1 - m_a)t_a$$

Despite the lengthy specification of this model, it is easily incorporated into computer programs such as SIDRA (Akcelik 1987)

Priority Rule Operation Example

(a)   Capacity for Entry

Stream 1 is from the North
Stream 2 is from the South

$q_1 = 248$ veh/h
$q_2 = 249$ veh/h
$T_r = 5$ s
$T_{or} = 3$ s, from NAASRA (1988:13)
$\alpha = 0.7$, from Troutbeck (1989)
$\lambda_1 = 0.0559$ veh/s, from (2)
$\lambda_1 = 0.0562$ veh/s
$Q_{1r} = 1385$ veh/h, from (8)
$Q_{2r} = 1388$ veh/h
$q_{1r} = 137$ veh/h (right turners)
$q_{2r} = 53$ veh/h (right turners)
$P_{o1} = 0.901$, from (7)
$P_{o2} = 0.960$
$T = 8$ s, from NAASRA (1988)
$q_a = 0.175$ veh/s, from (6)
$\alpha' = 0.573$, from (4)
$\lambda' = 0.155$ veh/s, from (5)
$Q_3 = Q_4 = 0.0736$ veh/s, from (3)

The entry capacity is therefore about 265 veh/h from either minor road approach.

(b)   Average Delay to the Minor Streams

$T = 8$ s, from NAASRA (1988)

$\beta = 0.0452$, from (14)
$D_{min} = 1.5$ s, from (13)
$x_3 = 0.546$,
$x_4 = 0.825$, from (12)
$D_3 = 3.30$ s
$D_4 = 8.57$ s, from (11)
Also, the opposed right turners on the major road have minimum delays D_{min1} and D_{min2} and the average delays D_1 and D_2.
$D_{min1} = 1.053$ s
$x_1 = 0.0989$
$D_1 = 1.16$ s
$D_{min2} = 1.03$ s
$x_2 = 0.0396$
$D_2 = 1.072$ s
Each approach delay is then multiplied by the respective flows to give total delay in veh h/h. (Table VI).

(c) Stops

$P_{d3} = 0.735$, from (15).

Similarly for the other approaches. The results are also in Table VI.

(d) Fuel Consumption

If the vehicle on the minor road is delayed it slows from the cruise speed of 60 km/h to zero and accelerates again to the cruise speed. If it is not delayed it decelerates to only 20 km/h and accelerates again. We require the excess fuel consumption for each manoeuvre.

(i) Delayed Vehicles

$v_c = 60$
$m_a = 0.587$, from (35)
$t_a = 19.6$ s, from (36)
$x_a = 0.192$ km, from (34)
$E_k = 0.723$ J/kg m, from (31)
$k_1 = 0.649$, from (32)
$k_2 = 1.499$, from (33)
$F_a = 40.9$ mL, from (30)
$k_x = 0.382$, from (38)

343

$k_y = 0.486$, from (39)
$k_a = 0.0505$, from (40)
$k_1 = 0.668$, from (41)
$m_d = 0.566$, from (43)
$t_d = 16.9$ s, from (44)
$x_d = 0.159$ km, from (42)
$E_k = 0.8735$ J/kg m, from (31)
$F_d = 10.0$ mL, from (37)
$E_k^+ = 0.150$ J/kg m, from (47)
$k_{E_1} = 0.255$, from (46)
$f_c = 91.65$ mL/km, from (45)
$t_i = 0$, from (49)
$f_h = 18.7$ mL, from (29)

(ii) Undelayed Vehicles

$m_a = 0.545$, from (35)
$t_a = 15.9$ s, from (36)
$x_a = 0.190$ km, from (34)
$E_k = 0.650$ J/kg m, from (31)
$k_1 = 0.572$, from (32)
$k_2 = 1.391$, from (33)
$F_a = 35.3$ mL, from (30)
$k_x = 0.4336$, from (38)
$k_y = 0.534$, from (39)
$k_a = 0.08114$, from (40)
$m_d = 0.5386$, from (43)
$t_d = 13.2$ s, from (44)
$x_d = 0.158$ km, from (42)
$E_d = 0.7813$ J/kg m, from (31)
$F_d = 8.3$ mL
$f_h = 11.7$ mL

These two rates of excess fuel consumption per vehicle are applied to the proportion which are delayed and the remainder which are undelayed. The results are summarised in Table VI.

Signal Operation Example

Two phases will be sufficient, one for each street with each approach marked in two lanes.

L = intersection lost time, assumed to be 10 seconds, 5 seconds per phase

> $Y =$ intersection flow ratio, the sum of the ratios (q/s) for critical movements.

If the base saturation flows are 1700 (through car units per hour or tcu/h) for the left lane and 1850 tcu/h for the right lane, the approach saturation flows are 3550 tcu/h. These are then adjusted for the proportion of turning traffic and the equivalent through cars for left and right turning vehicles, 1.25 and 3.00 respectively. Heavy vehicles are ignored. This is the procedure in Akcelik (1981). Pedestrian times are required to cross each street and these are assumed to be 16 seconds.

With this done, $Y = y_1 + y_4$, the two representative approaches. Hence $Y = 0.35$ for this example. The optimum cycle time is also the minimum, 42 s.

> $x < x_0$, for all approaches

Hence $N_0 = 0$, from (17) The results are shown in Table VII

The excess fuel consumption is obtained by the method in Akçelik (1981):

$$E = f_2 D + f_3' H$$

where   E is the excess fuel consumption per stop or slow down.
   D is the total stop-line delay in veh-h/h
   H is the total number of complete stops per hour
   f_2 is the fuel consumption rate while idling in L/veh.-h
   $f_2 = 1.60$ (or 0.444 mL/veh-s as before)
   $f_3' = 0.02$ L/stop (c.f. 0.0187 mL as before)

Roundabout Operation Example

Inscribed diameter is 32 m and approximately the same for each entry. The number of circulating lanes is 1. The circulating and entering volumes are shown in Fig 4.

From Table I, the dominant (only) entry stream follow-on time is approximately 2.7 s for each entry with no adjustment from Table II.

The average entry lane width is 4 m. For circulating flows in the range 200 to 400 veh/h, the ratio of critical acceptance gap to the follow-on time is approximately 1.9 from Table IV. Hence T is about 5.1 (s). The intra-bunch headway is set to 2 s for the single circulating lane. The proportion of free vehicles is therefore about 0.7 (Table V).
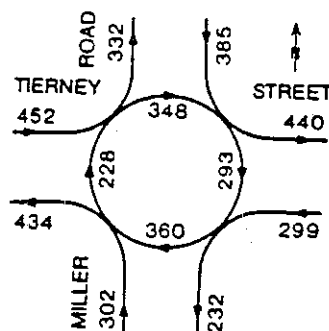
345

Fig. 4 Roundabout flows

From equation (2) the values of λ were calculated. From λ and q for each approach, Q, D_{min}, D and P_d^* were calculated. The values are shown in Table VIII.

In calculating the excess fuel consumption, similar assumptions were made as in priority rule case. The undelayed vehicles were assumed to slow down to 40 km/h to circulate at the roundabout. In this case, the excess fuel consumption was only 5.5 mL per vehicle compared with 11.7 mL for slowing to 20 km/h.