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THE GEOMETRICAL DESIGN OF SIGNALISED ROAD TRAFFIC JUNCTIONS.

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ABSTRACT

Methods are provided, mainly via simulation modelling but also utilizing mathematical analysis, to enable vehicle queue lengths, delays, and saturation flow profiles to be estimated on a flared approach and on a conventional approach to a signalised road traffic junction. The models show that the provision of flared approaches can appreciably increase the vehicular capacity of a junction. Comparisons are made between simulated and observed results. Consideration is also given to the problem of optimising the design of the junction to best accommodate certain features of the traffic input pattern.

1 INTRODUCTION

The use of mathematical models to estimate capacity, delays, queue lengths, etc., at signalised junctions is a well-established practice (see, for example, Webster (1958), Allsop (1971)). More elaborate models exist to deal with the additional problems which occur in the conflicts caused by the turning movements of opposing traffic, see Allsop (1977). Several algorithms have been developed to optimise signal timings, where the criterion is the minimization of total delay to vehicles, see for example Saka (1986).

An alternative policy in the continuing search for improved junction performance is to consider changes in geometrical design, the objective being to achieve an increase in vehicular capacity, thus leading to a reduction in overall delay to vehicles. A flared approach to a set of traffic signals (or a roundabout) consists of a widened section of roadway in the proximity of the junction; the purpose is to provide extra lanes at the stop-line to enable greater numbers of vehicles to depart during the initial part of the green phase of the signals. Although it is becoming increasingly common to provide flared approaches at traffic signals and roundabouts, in many cases it is necessary to make substantial alterations to road and walkway layout (or even demolish buildings), resulting in considerable expense. Clearly such changes are only worthwhile if it can be demonstrated that they will produce a considerable improvement in the performance of the junction. Considerations relating to the design of such flares have been the subject of several investigations, see, for example, Kimber and Semmens (1982) and Griffiths and Williams (1985).

The present study is concerned with producing quantitative evidence of the advantages to be gained in providing wide flares at such junctions, and in making comparisons with standard layouts so that reasoned judgments (probably based on a cost analysis) may be made relating to the merits of installing a flared approach at a particular junction.

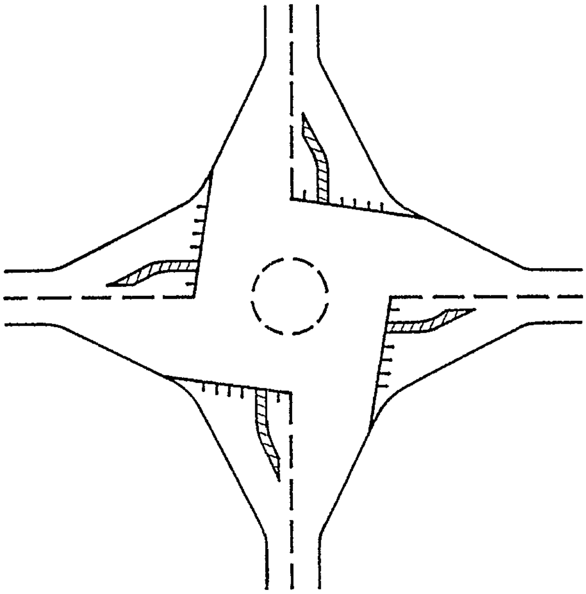
The modelling situation abounds with uncertainty. For example, what proportion of the flared area can actually be filled with vehicles under saturation flow conditions, bearing in mind that vehicles with different intended turning directions may prevent one another from accessing their desired position, thus leaving 'holes' vacant in the flare ?

Whilst it is possible to make some progress using mathematical arguments, the main emphasis of the work has been manifested in developing a realistic simulation model.

2 SIMULATION MODEL

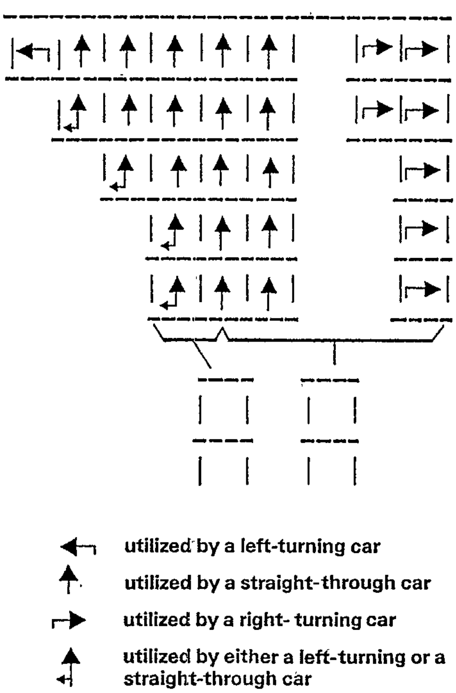
Figure 1 shows the layout of a typical flared junction. It should be noted that a central reservation area is provided to accommodate right-turning vehicles after they have crossed the stop-line. Vehicles depart from this reserved area either by seeking appropriate sized gaps in the oncoming flow of vehicles, or by waiting until the end of the green phase when they are able to complete their exit manoeuvre before the start of the next phase of lights. Several forms of gap-acceptance function were studied, notably step-functions, normal and log-normal distributions.

Activities on two opposing arms of a symmetrical junction were modelled, with each arm being fed by two approach lanes. Sufficient flexibility was built in to enable a large variety of different flare designs to be studied in addition to the non-flared situation. In essence the model provides a computer representation of the design of a flared approach (the numbers of left-turning, straight-through and right-turning lanes within the flare, the lengths of such lanes, etc), see Figure 2, and then processes vehicles on an event basis in accordance with assumptions postulated with regard to driver behaviour in choice of approach lane, filling of flare, departure process, etc. These assumptions were based on previous studies, and were verified and calibrated in several instances by reference to a film of a Test Track experiment held at Transport and Road Research Laboratory (TRRL) in 1980.



EXAMPLE OF A TYPICAL FLARED JUNCTION LAYOUT.

FIGURE 1



COMPUTER REPRESENTATION OF A FLARED DESIGN.

FIGURE 2

3 CALIBRATION OF SIMULATION MODEL

There are two major areas of the simulation model which have to be calibrated; these are described below :

3.1 Departure Headways Of Vehicles Across The Stop-line And Move-up Times Within A Queue

Data collected at 35 sites were provided by the University of Southampton (1982). Their survey measured certain parameters at traffic signals, which included departure headways of vehicles as they crossed the stop-line. The departure headway of a vehicle is defined as the time interval between the rear wheels of that vehicle crossing the stop-line and the rear wheels of the preceding vehicle crossing the stop-line. In the case of a first-in-line vehicle the time interval is measured from the start of the green phase.

The data did not contain any information about vehicle movements further back in the queue and it was not possible from the Southampton video recordings or the Test Track video recordings to obtain this information. The assumption was made that each car space which a 'delayed' vehicle has to move through takes a constant time of about 2.0 seconds. This will be referred to as the 'move-up time'.

To determine the departure headway distribution of delayed cars at the stop-line each site was analysed, using a 'Chi-Square Goodness of Fit Test' as the criterion, for each individual queueing position and for all queueing positions combined. For individual queueing positions no single distribution was a good fit, but overall the Lognormal distribution provided a statistically sound fit at just over eighty percent of the sites.

A saturation flow profile is constructed from the number of vehicles departing per six-second interval during the green period when the system is fully saturated, as described in Road Note 34 (1963). The observed saturation flow profiles from the Test Track experiment with no turning traffic were provided by TRRL. These were compared with simulation runs for each layout using the Lognormal distribution for departure headways and constant move-up times of 2 seconds. If the standard deviation of the Lognormal distribution is set at 1/8th of the mean (for the Southampton data it is approximately 2/5th of the mean), the saturation flow profiles compared well with the Test Track results for some (but not all) of the layouts.

Further analysis on the Southampton data combining all sites gave clearer results regarding the departure headway distributions. A Negative Exponential distribution gave the 'best fit' for vehicles in the first queueing position, and a Lognormal or an Erlang distribution with shape parameter of about 10 for all other queueing positions. The average departure headway of the first vehicle was also noticeably smaller than any of the other positions, indicating that first-in-line drivers tended to anticipate the green light.

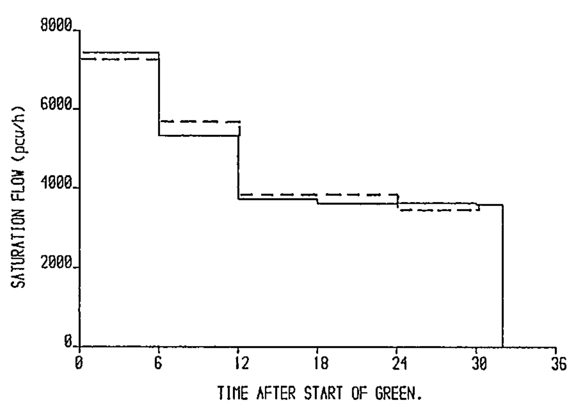
The time required to generate random Erlang variates increases considerably as the shape parameter becomes larger and although algorithms are available, see Cheng and Feast (1980), which substantially reduce this generation time it still significantly increases the c.p.u. time required by the simulation. Therefore constant departure headways for second and subsequent delayed vehicles were used in the simulation, since the Erlang distribution tends to a constant headway distribution as the shape parameter increases. In fact it was found that no improvement was obtained by using Erlang distributed variates for these departure headways.

A saturation flow profile, produced by the simulation program using a Negative Exponential departure headway distribution with a mean of 1.5 seconds for the first-in-line cars after the start of green and a constant departure headway of 2.0 seconds for the remaining vehicles, for a typical layout is shown in Figure 3. The saturation flow profiles obtained from the Test Track experiment are superimposed on the simulation results for comparison purposes. All move-up times within the queues were constant at 2.0 seconds. Turning vehicles had the same parameter settings as above except for second and subsequent headways which were a constant 2.3 seconds; this value was taken from the Track experiment data, Kimber and Semmens (1982).

1 Left Turning Lane.
1 car space.

5 Straight Through Lanes.
2 3 5 5 5 car spaces.

2 Right Turning Lanes.
2 5 car spaces.



———— represents Simulation.

- - - - - - represents Track Experiment.

COMPARISON OF SIMULATION AND TEST TRACK SATURATION FLOW PROFILE.

FIGURE 3

3.2 The Gap-Acceptance Behaviour Of Right-turning Vehicles.

The choice of suitable gap-acceptance distributions and the estimation of their associated parameters is a complicated task. For instance if the data consists of all gap lengths offered to waiting vehicles which are accepted or rejected, then the distribution is biased as a result of a number of individual drivers rejecting several short gaps before eventually accepting a longer one, Ashworth and Green (1966). Also some authors, for example Ashworth and Bottom (1977), have distinguished between gap-acceptance and lag-acceptance behaviour, finding that gaps were more likely to be accepted than lags of equivalent size. (A lag is the remaining part of a gap which is presented to a driver on his arrival at the Give Way line.)

Once the distribution and parameters have been decided on there are two distinct ways in which they can be applied within a simulation model. The first is to generate a probability of acceptance according to some predetermined distribution for each gap in the opposing stream; this can result in drivers occasionally rejecting a large gap and subsequently accepting a much smaller gap. The second method is to generate, again from a predetermined distribution, the minimum acceptable gap to a waiting driver and then reject all successive gaps in the opposing stream until a gap equal to or greater than this minimum acceptable gap occurs. Although neither method adequately describes the true 'real-life' situation, it has been suggested, see Ashworth and Bottom (1977), that the second method gives the better results of the two.

Literature exists on at least twenty different methods relating to the analysis of gap-acceptance data, some of which have been reviewed by Ashworth (1970) and Miller (1972).

Figure 4, taken from Kimber and Semmens (1982), shows the observations made in the Test Track experiment concerning the right-turning flow versus opposing flow per cycle. The total number of right-turning vehicles were split into two operational modes:

1. those right-turners who found gaps in the opposing flow (gap-seeking),

2. those who crossed after the opposing flow has stopped during the inter-green period (reservoir clearance).

This behaviour of the right-turning vehicles was modelled within the simulation program. From the video recording of the Test Track experiment, it was observed that on average a car in the opposing stream took approximately four seconds after crossing the stop-line to pass in front of a waiting right-turning vehicle. Thus departure times of cars across the stop-line were displaced by four seconds when checking for gaps of suitable lengths.

Initially the simulation was run using a step-function as the gap acceptance criterion for the right-turning vehicles, i.e. if a gap greater than or equal to a specified time occurred, the waiting vehicle accepted it; this enabled the logic of the simulation model to be thoroughly checked.

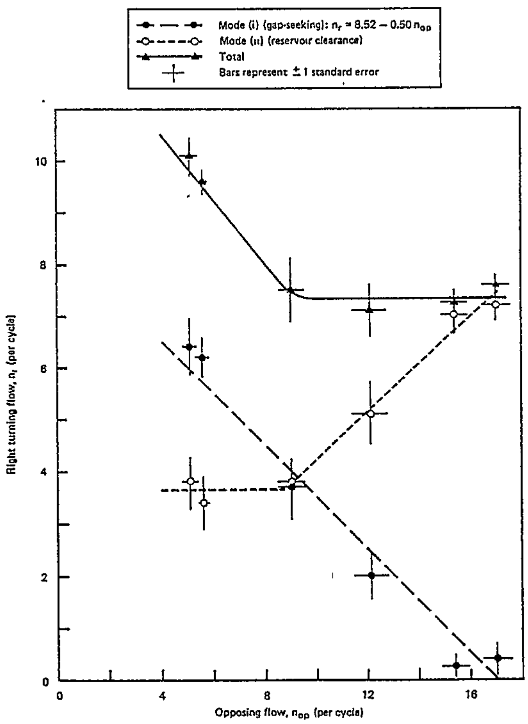
The Lognormal distribution, which has been shown to be a more realistic gap-acceptance distribution, Miller (1972), was then used with a mean value of 5.26 seconds, variance of 2.25 seconds, minimum limit of 1.6 seconds and maximum limit of 9.1 seconds. (If the gap drawn from the distribution lay outside one of these limits it was set equal to the limit.) The parameter values are those observed by Southampton University in their study of 'Saturation Flows at Traffic Signals' (1984), and are approximately in the middle of the range of values recorded elsewhere.

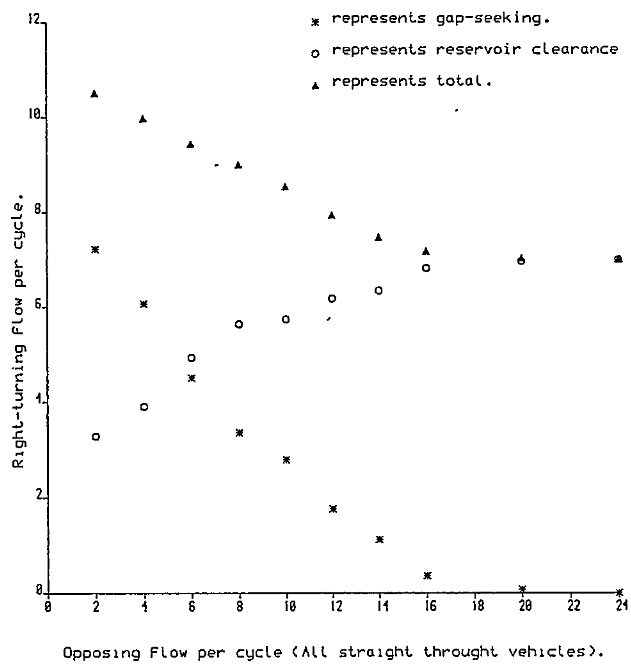
As can be seen from Figure 5, which shows results of simulation runs using the Lognormal distribution, the numbers of right-turners who gap-cross are very similar to the observed numbers in the Test Track experiment, although there is a discrepancy between the numbers of right-turners who clear during the inter-green period when the opposing flow is between 4 and 12 vehicles per cycle. This may be due to the fact that it is easier in the simulation model than in the Track experiment to saturate the junction with right-turning vehicles, and then to identify those vehicles that found a suitable gap between the last opposing vehicle and the end of green, and those vehicles that waited until the end of green to complete their right turn.

4 SIMULATION RESULTS

Figure 6 shows the average number of vehicles queueing on one arm of the junction at the beginning of the green period, given by the simulation model for a typical layout, plotted against the vehicle total arrival rate per hour on that arm. The flow consists of 10% left-turning and 20% right-turning vehicles, with the right-turning vehicles gap-crossing against an opposing flow rate of .31 vehicles per second (1100 veh/hr), which is a medium sized flow level. For vehicle flow rates below capacity (the vehicle arrival rate at which the number queueing begins to increase to an unacceptable level), the simulation results are steady, but become extremely sensitive and dependent on the duration of the simulation when capacity flow rates are approached.

Comparing delays at flared and non-flared layouts, waiting times were found to be similar at traffic flows up to levels approaching the capacity of the non-flared junction. An example relating to a typical layout is given in Figure 7. Clearly the increased capacity of the flared situation prevents significant delays from building up until a much later stage.





RIGHT-TURNING FLOW VERSUS OPPOSING FLOW FOR A TYPICAL LAYOUT.

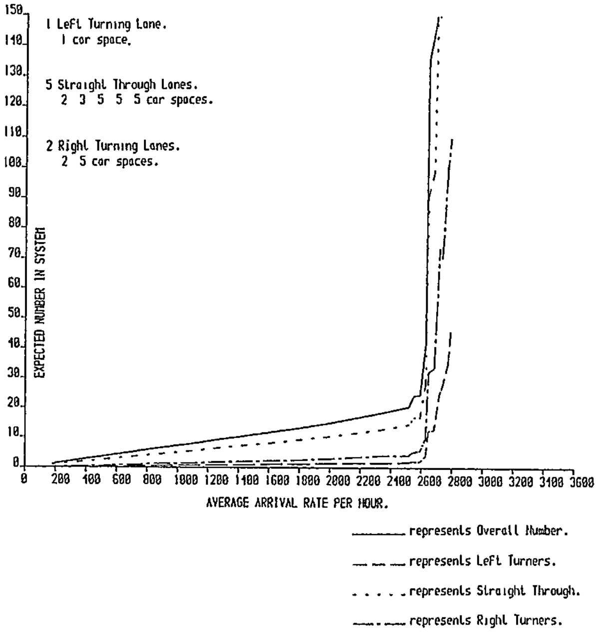
TEST TRACK RESULTS

FIGURE 4

SIMULATION RESULTS

FIGURE 5

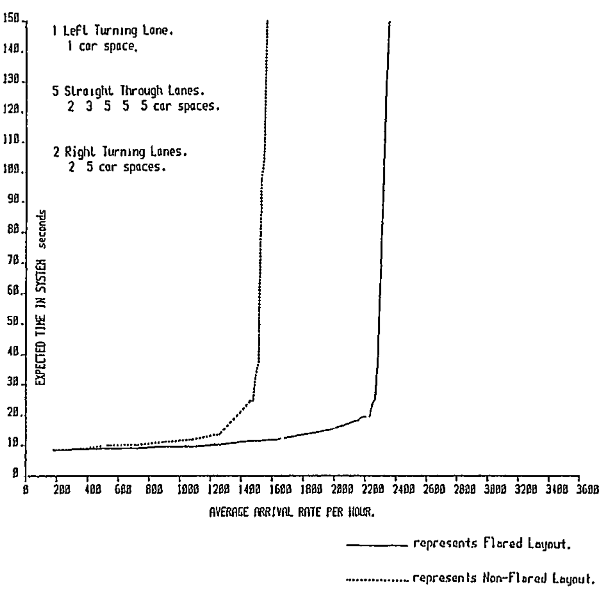
10% Left Turners, 20% Right Turners.
23sec Red, 2sec Red/Amber, 18sec Green, 3sec Amber.



GRAPH OF EXPECTED NUMBER IN SYSTEM
AT START OF GREEN PHASE.

FIGURE 6

0% Left Turners, 0% Right Turners.
23sec Red, 2sec Red/Amber, 18sec Green, 3sec Amber.



COMPARISON OF TIME IN SYSTEM FOR FLARED
AND NON-FLARED LAYOUTS.

FIGURE 7

Table 1 gives the approximate capacities, λ_{max} (veh/sec), for various combinations of vehicle turning proportions under the following conditions ; no extra delay within the junction to right-turning vehicles; all headways and move-up times are constant at 2.0 seconds; three different cycle times.

TABLE 1.

Capacity on one arm of the junction produced by Simulation Model.

With (i) No extra delay within junction for right-turning vehicles.

(ii) All headways and move-up times 2.0 seconds (constant).

34 sec. cycle time, (12 second green).

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop.	.0	.65	.71	.77	.75
of	.1	.66	.72	.77	.76
left-	.2	.60	.65	.69	.72
turners.	.3	.52	.57	.62	.63

46 sec. cycle time, (18 second green).

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop.	.0	.61	.66	.69	.70
of	.1	.62	.67	.71	.71
left-	.2	.57	.62	.65	.66
turners.	.3	.53	.55	.60	.60

70 sec. cycle time, (30 second green).

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop.	.0	.57	.60	.63	.63
of	.1	.57	.60	.63	.65
left-	.2	.54	.58	.58	.59
turners.	.3	.51	.55	.57	.57

Changing the departure headway distribution and move-up times in accordance with the earlier discussion, the results of simulation runs for a 34 second cycle time and a 46 second cycle time are given in Table 2. The runs for a longer cycle time of 70 seconds were not repeated as it is obvious from Table 1 that the larger designs are more advantageous when used in conjunction with shorter cycle times.

TABLE 2.

Capacity on one arm of the junction produced by Simulation Model.

With (i) No delay to right-turning vehicles within junction.

(ii) First headways at start of green Neg. Exp., mean 1.5 seconds, rest constant 2.0 or 2.3 seconds if turning.

34 sec. cycle time, (12 second green).

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop. of left-turners.	.0	.69	.75	.80	.78
	.1	.69	.74	.81	.77
	.2	.62	.66	.71	.71
	.3	.54	.60	.65	.63

46 sec. cycle time, (18 second green).

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop. of left-turners.	.0	.64	.68	.73	.72
	.1	.65	.68	.73	.73
	.2	.60	.63	.65	.68
	.3	.55	.57	.62	.61

TABLE 3.

Capacity on one arm of the junction produced by Simulation Model.

With (i) Right-turners delayed in junction until end of green.

(ii) First headways at start of green Neg. Exp., mean 1.5 seconds, rest constant 2.0 or 2.3 seconds if turning.

34 sec. cycle time, (12 second green).

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop. of left-turners.	.0	.69	.76	.79	.70
	.1	.69	.75	.79	.70
	.2	.63	.68	.71	.68
	.3	.56	.58	.63	.60

46 sec. cycle time, (18 second green).

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop. of left-turners.	.0	.65	.68	.69	.53
	.1	.65	.69	.70	.53
	.2	.59	.63	.65	.53
	.3	.54	.58	.60	.52

Table 3 contains results for the simulation model when right-turning vehicles are forced to wait within the junction until the end of the green phase. Tables 2 and 3 combine to provide upper and lower limits for the capacity on one arm of the junction. Allowing right-turning vehicles to gap-cross will give a junction capacity between these limits, the exact value depending on the magnitude of the opposing flow.

When the opposing flow had 10% left- and 20% right-turning vehicles, the capacities (for three different levels of flow) were found to be as shown in Table 4. No significant changes in capacities were found when using different turning proportions on the opposing arm.

TABLE 4.

Capacity on one arm of the junction produced by Simulation Model.

With (i) Right-turning vehicles gap-crossing, accepting gaps drawn from a Lognormal Distribution.

(ii) First headways at start of green Neg. Exp., mean 1.5 seconds, rest constant 2.0 or 2.3 seconds if turning.

(iii) Cycle time 46 seconds (18 second green).

Opposing Flow = .16 veh/sec. (10% left-, 20% right-turning)

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop. of left-turners.	.0	.64	.69	.73	.70
	.1	.65	.69	.73	.70
	.2	.61	.63	.67	.67
	.3	.54	.58	.61	.60

Opposing Flow = .31 veh/sec. (10% left-, 20% right-turning)

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop. of left-turners.	.0	.64	.69	.72	.62
	.1	.65	.69	.73	.62
	.2	.61	.62	.66	.61
	.3	.54	.57	.60	.58

Opposing Flow = .70 veh/sec. (10% left-, 20% right-turning)

	λ_{max} veh/sec	Prop. of right-turners. .0	.1	.2	.3
Prop. of left-turners.	.0	.64	.68	.71	.52
	.1	.64	.69	.71	.52
	.2	.60	.63	.65	.52
	.3	.54	.57	.59	.52

A problem of some importance is to choose a flare design (i.e the number of flare lanes devoted to particular turning movements, the number of car spaces in each flare lane, etc) to maximise the capacity of an arm. Often physical constraints reduce the feasible flare patterns to a relatively small number. The simulation program can thus be used as an optimisation tool to check through these flare layouts, producing results similar to Table 4, and the design giving the largest capacity for the specified turning proportions chosen.

5 REPRESENTATION OF THE SIMULATION RESULTS IN ALGEBRAIC FORM

5.1 Capacity Estimation For One Arm Of The Junction.

The estimation of capacity for one arm of a junction, when there is no extra delay to right-turning vehicles caused by opposing vehicle flow, can be derived using regression analysis techniques.

Let

$$\lambda_{max} = (a_1 L + a_2 R + a_3 L^2 + a_4 R^2 + S_s + L_s) / \bar{y} \qquad (1)$$

where L = proportion of left-turning vehicles,

R = proportion of right-turning vehicles,

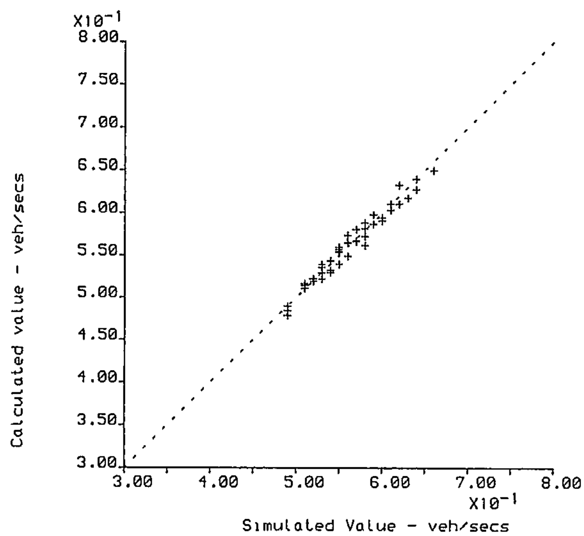
S_s = number of straight-through car spaces in flare,

L_s = number of vehicles from the approach lanes that depart during the green phase,

$\bar{y}$ = cycle time in seconds,

and $a_1, \ldots, a_4$ are the parameters provided by the regression analysis; these parameter values are different for each layout. Results of the calculated λ_{max} for one layout have been plotted against the simulated λ_{max} for all combinations of left- and right-turning proportions and cycle times, see Figure 8. (Each x represents a separate combination of left- and right-turning proportions.) It can be seen that the equation provides an excellent predictor for the capacity when headways and move-ups are constant.

All move-ups are constant 2 seconds.



CAPACITY ESTIMATION USING REGRESSION EQUATION, TYPICAL LAYOUT.

FIGURE 8

A useful equation which will provide a means of converting from the capacities given by the simulation model when departure headways etc. are constant, shown in Table 1, to the more realistic capacities, shown in Table 2 is :

$$\lambda_{max} = 1.04 \quad \lambda_{const} \text{ (veh/sec)} \qquad (2)$$

where λ_{max} is the capacity shown in Table 2,

and λ_{const} is the capacity shown in Table 1.

It is now necessary to determine a method of calculating the capacity when right-turning vehicles are gap-crossing against varying levels of opposing flow. The controlling influence in the level of the capacity is obviously the number of right-turning vehicles that can depart per cycle. This can be calculated from equation (3), which has been developed from simulation results, some of which are shown in Figure 5.

$$\underset{\text{(veh/sec)}}{\lambda_r =} \begin{cases} (\bar{g}/2+1)/\bar{y} - \lambda_o(\bar{g}/2+1-n_{rs})/(.35\,\bar{y}) & \lambda_o < .35\text{veh/sec} \\ n_{rs}/\bar{y} & \lambda_o \geqslant .35\text{veh/sec} \end{cases} \qquad (3)$$

where λ_r = right-turning capacity (veh/sec),

λ_o = opposing straight-through flow per second,

n_{rs} = number of right-turning spaces in central reservation area,

$\bar{g}$ = effective green time in seconds,

$\bar{y}$ = cycle time in seconds.

The capacity on one arm of the junction is thus given by :-
Minimum of (equ.(3)/R and equ.(2))

where R is the proportion of right-turning vehicles.

This process can now be repeated for each arm on the junction enabling the total capacity for the four armed flared signal junction to be estimated.

6 CONCULSIONS

Methods have been provided, via simulation modelling and mathematical analysis, to enable saturation flow profiles, queue lengths and delays to be estimated on a flared approach to a signalised junction. In particular the effect on right-turning traffic of opposing vehicle streams has been studied.

The analysis shows that the provision of flared approaches can considerably increase the capacity of the junction, and the following factors have been identified as being the most important in the determination of the capacity :-

1. the size of the flared approach to the junction,
2. the cycle time,
3. the proportions of left- and right-turning vehicles in the approach flow.

The importance of condition 2 lies in the fact that any straight-through vehicles which enter the flare after the start of the green phase use only the two outermost straight-through lanes of the flare (as observed from the Test Track Experiment), thus reducing the throughput to that of a two-lane junction after the flare area has cleared. Hence any increase in capacity is derived from the first part of the green phase, making it beneficial to use flared approaches in conjunction with shorter cycle times.

For given percentages of left- and right-turning traffic, it is possible to choose the best flare design to suit those circumstances.

Comparisons between flared and non-flared junctions indicate that delays and queue lengths would be similar for flow levels below the capacity of the non-flared junction. Above this flow level the increased capacity of the flared situation prevents delays building up until a much later stage.

7 ACKNOWLEDGEMENTS

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Prof. Jeff Griffiths obtained a B.Sc. from University of Wales in 1961, the Diploma in Mathematical Statistics in 1963, and his Ph.D. in 1972. The subject of his Ph.D. thesis was Road Traffic Queues, and this topic has been the main focus of his research interests over the years. He has undertaken a large number of consultancies and contracts for several Government Establishments and private firms. Since 1984 he has been Head of the Department of Mathematics at the University of Wales Institute of Science and Technology, Cardiff, UK. His current interests lie in road traffic theory, epidemiology, and general queueing problems, and he has published extensively in these fields.

Dr. Janet Williams obtained a B.Sc.(Tech) from University of Wales in 1979. She successfully submitted her Ph.D. thesis entitled 'Stochastic Models of Road Traffic' in 1987 and has been involved in a number of studies on road traffic management. Currently she is employed as a Research Associate within the Department of Mathematics at the University of Wales Institute of Science and Technology, Cardiff, UK.

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