

REPRINT

X and Y in Traffic Signal Design

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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 Though some aspects of this paper may be outdated, this reprint is provided as a record of important aspects of the SIDRA INTERSECTION software, and in order to promote software assessment and further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.

X AND Y IN TRAFFIC SIGNAL DESIGN*

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ABSTRACT

The paper proposes various improvements to the traffic signal design method with particular reference to the method used in Australia. The intersection degree of saturation, X is recommended for use as a simple measure of operating conditions instead of the intersection flow ratio, Y . The use of Y may be misleading especially in the case of alternative analysis and when one or more signal phases have green times which do not satisfy pedestrian or vehicle minimum green time requirements. A simple formula for calculating the intersection X value without the need to calculate phase green times is given. It is proposed that an acceptable maximum degree of saturation X_m is used instead of an upper limit on the intersection Y value and it is recommended that $X_m = 0.90$ is used for general signal design purposes. The paper also describes methods for (a) calculating a minimum cycle time which yields an acceptable maximum degree of saturation, (b) calculating spare intersection capacity based on the use of X_m and a maximum acceptable cycle time and (c) signal design calculations when a minimum green time constraint is not satisfied. An alternative method for calculating signal settings is also described.

NOTATION AND DEFINITIONS

a	=	Green time ratio — the proportion of the cycle which is effectively green for a particular phase ($= g/c$)
A	=	Intersection green time ratio — the ratio of available green time to cycle time ($= 1 - L/c$ = summation for the whole intersection of the a values for each phase)
c	=	Cycle time
c_o	=	Optimum cycle time — the cycle time which gives the least total delay to all traffic using the intersection
c_m	=	Minimum cycle time — the cycle time which gives a maximum acceptable degree of saturation, X_m
c_{max}	=	Maximum acceptable cycle time (120 seconds suggested)
d	=	Average delay per vehicle in seconds
D	=	Total delay per unit time ($= qd$) in vehicle-hours per hour
E_{ri}	=	Through car equivalent of an opposed right turning car (average value 2.9)

E_z	=	Average overflow (residual) queue — average number of vehicles left in the queue when the signals change to red (total queued vehicles in all lanes when there are several lanes of vehicles)
f	=	A measure of the ability of right-turning vehicles to filter through the opposing traffic flow (used in the calculation of E_{ri})
g	=	Effective green time (= actual green time + amber time - lost time)
g_m	=	Minimum acceptable green time ($= y_c/x_m$) for vehicle operations (not safety or pedestrian crossing reasons)
g_s	=	Saturated portion of green ($= yr/(1-y)$)
G	=	Actual (controller) green time
I	=	Intergreen time (amber plus all red) — time from end of the green time of the phase losing right-of-way to the beginning of the green time of the phase gaining right-of-way (effective intergreen = all red plus lost time)
k	=	Capacity — maximum flow which can just be passed through the intersection from the particular approach ($= a s$)
k_m	=	Service volume — maximum flow value used for design purposes which corresponds to a maximum acceptable degree of saturation ($= x_m k$)
l	=	Lost time — the time which is effectively lost to traffic movement in the phase because of starting delays and the falling-off of the discharge rate which occurs during the amber period
L	=	Total intersection lost time per cycle — the sum of the lost times for each phase and those periods when all signals show red (all red periods)
n	=	Number of phases, where a phase is a state of the signal during which a particular group of traffic streams receives right-of-way

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N	=	Average number of vehicles in the queue at the beginning of the green period, i.e. the maximum queue during an average signal cycle ($= E_z + qr$)
p_0	=	Proportion of cycles during which the queue is cleared
p_s	=	Proportion of oversaturated cycles ($= 1 - p_0$)
q	=	Flow — average number of vehicles passing a given point on the road in the same direction per unit of time
r	=	Effective red time ($= c - g$ = actual red time + lost time)
s	=	Saturation flow — maximum rate of discharge of the queue during the green period
tcu	=	Through car unit — a 'through' passenger car unit (used in saturation flow calculations to allow for turning vehicle effects)
x	=	Degree of saturation — the ratio of flow to capacity ($= q/k = qc/s = y/a$)
X	=	Intersection degree of saturation ($= Y/A$) — the value of x which is the same for each traffic stream representative of its phase (obtained when the effective green times are made proportional to the representative y values for each phase, as given by the formula $g = y(c - L)/Y$)
x_m	=	Maximum acceptable degree of saturation for an individual phase
X_m	=	Maximum acceptable degree of saturation for the intersection (0.90 suggested)
X_p	=	Practical (minimum) degree of saturation which corresponds to the maximum acceptable cycle time, c_{max}
y	=	Flow ratio — the ratio of flow to saturation flow ($= q/s$)
Y	=	Intersection flow ratio — summation for the whole intersection of the y values representative of each phase
α	=	Load factor — proportion of fully saturated cycles ($> p_s$ since α includes cycles which are just saturated as well as those which are oversaturated)

INTRODUCTION

1. Operational design of traffic signals involves the selection of a phasing system (definition, order and number of phases) and the calculation of signal settings (cycle time and green times, also offsets for linked signals) for given intersection geometry (number of lanes, turning radii, etc.) and flow conditions, usually considering various peak traffic conditions. Ideally, alternative signal designs should be developed and the best, i.e. the most cost-effective design should be selected to

satisfy given conditions with the safety as a fundamental constraint.

2. Flows and saturation flows are the fundamental parameters in traffic signal design. Flows are normally treated as fixed data (except when signal control is to be coupled with turn bans, one-way systems and wider route control measures), but methods of increasing saturation flows are usually sought to improve intersection operating conditions and the provision of additional lanes is the most effective way of achieving this.

3. Given a set of flow and saturation flow conditions, an optimum signal plan is calculated for a chosen phasing system. Total system (intersection) delay is a widely accepted measure of performance employed for this purpose (Webster and Cobbe 1966; ARRB 1968). Since the calculation of total intersection delay may be rather time consuming, simple and straightforward measures of operating conditions are needed for preliminary design purposes. Three simple measures which can be considered are as follows.

- Intersection flow ratio (Y):* this is the sum of flow/saturation flow ratios for critical movements. A higher value of Y indicates worse operating conditions.
- Cycle time (c):* the optimum value of the cycle time is determined by intersection flow ratio Y and lost time L . A smaller optimum cycle time indicates better operating conditions.
- Intersection degree of saturation (X):* this is a direct measure of the intersection level of service and depends on intersection flow ratio, lost time and cycle time (Y, L, c). The maximum acceptable degree of saturation corresponds to the 'service volume' concept of the U.S. *Highway Capacity Manual* (HRB 1965) and determines the minimum (practical) value of cycle time for given Y and L .

4. At present, intersection flow ratio Y is used as a simple measure of intersection operating conditions for preliminary design purposes in the Australian traffic signal design practice (ARRB 1968; CRB 1974). A value of Y less than 0.70 is recommended and 0.75 is specified as an absolute upper limit. However, it may be misleading to use Y , especially in the case of alternative analysis and when one or more signal phases have green times which do not satisfy vehicle or pedestrian minimum green time requirements. At the same time, there is undue emphasis on the value of an upper limit on Y . For the critical values of Y (those greater than about 0.65) intersection performance depends on the values of cycle time and lost time (c, L) as much as the value of Y and the acceptability of the design should not be decided on using Y as a single criterion.

5. It is proposed in this paper that the intersection degree of saturation X , rather than the flow ratio Y , is used for preliminary design purposes since it allows for the lost time and cycle time as well as the Y value and it is a quantitative measure of level of service commonly used in the design of all other road traffic facilities. A simple formula for calculating the intersection X value without the need to calculate phase green times is given. A table of level of service definitions for signalised intersections in relation to the degree of saturation X is presented. A method is proposed for signal design calculations in the case where a minimum green time requirement is not satisfied.

6. It is proposed that an acceptable maximum degree of saturation X_m is used instead of an upper limit on the intersection Y value. It is recommended that $X_m = 0.90$ is used for general signal design purposes, which corresponds to the upper limit of level of service D . A method is also described for calculating spare intersection capacity, which is based on the use of X_m and a maximum acceptable cycle time (e.g. 120 seconds). The importance of using a small cycle time in traffic signal control is discussed and a formula to calculate a minimum cycle time c_m which yields an acceptable maximum degree of saturation is given. A table is presented which gives minimum cycle times for the intersection lost time values of 10, 15 and 20 seconds, satisfying particular level of service and delay criteria.

INTERRELATIONSHIPS OF Y, c AND X

7. Traffic signal design methods used in the United Kingdom (Webster and Cobbe 1966) and Australia (ARRB 1968) are fundamentally the same and based on Webster's (1958) Y -value method. In this method:

- the ratio of flow to saturation flow (y value) for each approach (or individually defined movement) is calculated;
- a representative flow ratio (critical y value) is determined for each phase by comparing y values of approaches (movements) which have right-of-way during that phase and choosing the largest one (overlapping phases are taken into account in this process); and
- the intersection flow ratio Y is calculated as the sum of representative y values.

Then, an optimum signal plan (cycle time and green times which approximately minimise total intersection delay) is calculated using the intersection Y value.

8. In addition to the method of estimating saturation flows, the U.K. and Australian methods differ in the calculation of an optimum cycle time. The U.K. method is based on the use of Webster's (1958) cycle time formula

$$c_o = \frac{1.5L + 5}{1 - Y} \quad (1)$$

whereas the Australian method is based on Miller's (1963) cycle time formula which is given in ARRB (1968) as

$$c_o = \frac{L + 2.2\sqrt{L/s}}{1 - Y} \quad (2)$$

where s is the lowest of saturation flows for any of the representative movements. Thus, both formulae are based on the use of the intersection Y value and lost time L (in seconds), but the ARRB formula takes into account the minimum saturation flow value (in through car units per second (tcu/s)) used in the calculation of Y . If s is in vehicles per second, then the constant in front of the square root is 2.0. A numerical analysis will show that the two formulae will give very close results if the minimum saturation flow corresponds to a single lane (values around 1700 tcu/h) whereas the ARRB formula will produce smaller cycle times in cases where the minimum saturation flow corresponds to two or more lanes.

9. The values of optimum cycle time (nearest 10 seconds) derived from the single lane minimum saturation flow value of 1700 tcu/h are presented in Table I which can be used for preliminary design purposes (lost time values of $L = 10, 15$ and 20 seconds correspond to two, three and four phase systems assuming 5 seconds lost time per phase). From Table I, it can be seen that the choice of a suitable cycle time is important for values of Y greater than about 0.65 depending on the value of L (hence the number of phases). In fact it is seen that the role of intersection lost time L is more significant than the role of Y in the critical region of Y values. It is also seen that the upper value of Y should depend on L (e.g. Y values of up to 0.82 can be allowed in design for $L = 10$ if a maximum value of $X = 0.90$ is accepted).

10. The method of calculating green times to give the least delay to all traffic using the intersection is to make the effective green times proportional to the representative y values for each phase i , as given by the formula

$$g_i = \frac{c - L}{Y} y_i \quad \text{for } i = 1, 2, \dots, n \quad (3)$$

where n = number of phases and g_i = green time for phase i .

11. The degree of saturation, x of an approach road is defined as the ratio of flow to capacity, q/k . The capacity under signal control conditions is given by

$$k = as \quad (4)$$

where $a = g/c$ is the green time ratio, i.e. proportion of the cycle which is effectively green, and s = saturation flow. Hence the degree of saturation is

$$x = \frac{q}{as} = \frac{Y}{a} \quad (5)$$

From eqn (3) the green time ratio for phase i is

$$a_i = \frac{g_i}{c} = \left(\frac{c - L}{c} \right) \frac{y_i}{Y} \quad (6)$$

and the degree of saturation for phase i is

$$x_i = \frac{y_i}{a_i} = \left(\frac{c}{c - L} \right) Y \quad (7)$$

Hence the degrees of saturation for all representative movements resulting from green settings using the method expressed by eqn (3) are equal, i.e.

$$x_1 = x_2 = \dots = x_n = X \quad (8)$$

Thus the method of calculating green times expressed by eqn (3) is an 'equal degree of saturation' method and on this basis an intersection degree of saturation can be defined as

$$X = \left(\frac{c}{c - L} \right) Y \quad (9)$$

Since $c - L = g_1 + g_2 + \dots + g_n$ = total available green time per cycle, an intersection green time ratio can be defined as the sum of phase green time ratios, i.e.

$$A = a_1 + a_2 + \dots + a_n = \frac{c - L}{c} \quad (10)$$

TABLE I

PRACTICAL OPTIMUM CYCLE TIMES (c_o)

Y	L = 10		L = 15		L = 20		Level of Service
	c	X	c	X	c	X	
0.30	30*	0.45	40	0.48	50	0.50	B
0.40	40	0.50	50	0.57	60	0.60	
0.45	40	0.56	50	0.64	70	0.63	
0.50	40	0.67	60	0.67	70	0.70	C
0.54	50	0.68	60	0.72	80	0.72	
0.58	50	0.73	70	0.74	90	0.75	
0.60	50	0.75	70	0.76	90	0.77	D
0.62	60	0.75	70	0.79	90	0.80	
0.64	60	0.77	80	0.79	100	0.80	
0.66	60	0.79	80	0.81	100	0.83	E
0.68	70	0.79	90	0.82	110	0.83	
0.70	70	0.82	90	0.84	120	0.84	
0.72	70	0.84	100	0.85	120*	0.86	F
0.74	80	0.85	110	0.86	120*	0.89	
0.76	90	0.86	120	0.87	120*	0.91	
0.78	90	0.88	120*	0.89	120*	0.94	F
0.80	100	0.89	120*	0.91			
0.82	110	0.90	120*	0.94			
0.84	120*	0.92					
0.86	120*	0.94					

c* = Minimum or maximum cycle time

c = Optimum cycle time (nearest 10 seconds)

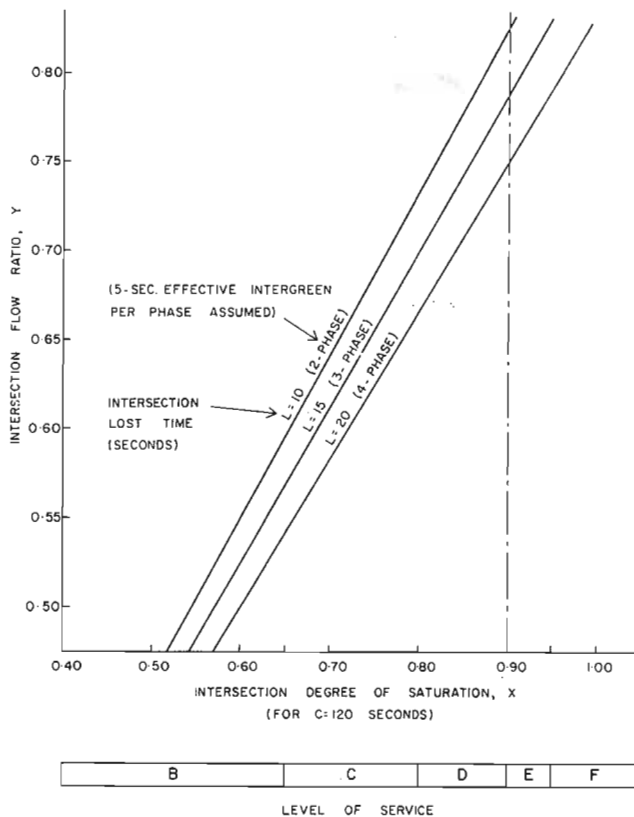


Fig. 1 — The relationship between X and Y

Then, eqn (9) for the intersection can be written as

$$X = Y/A \quad (11)$$

which is similar to eqn (5) for an individual phase or movement.

12. It is therefore seen that the intersection degree of saturation can be calculated from eqn (9) without calculating individual phase green times. It should be noted that individual x values of representative movements may differ from the value of X calculated from eqn (9) by up to ± 2 per cent if green times are rounded to the nearest second for practical reasons.

13. The relationship between X and Y is illustrated in Fig. 1. Three lines representing X - Y relationships for a cycle time of 120 seconds and lost times of 10, 15 and 20 seconds are drawn. Writing eqn (9) as

$$X = \frac{Y}{1 - L/c} \quad (12)$$

it can be seen that the lowest degree of saturation, X is given by the maximum value of cycle time, c for given Y and L . Using $c_{max} = 120$ seconds as the maximum cycle time and assuming 5 seconds lost time per phase, eqn (12) can be written as

$$X = \frac{Y}{1 - n/24} \quad (13)$$

where n = number of phases. X - Y relationships given in Fig. 1 have been drawn using eqn (13) and can be used to find X for a given Y value and number of phases, n , for preliminary design purposes. In Fig. 1, levels of service corresponding to X are also illustrated. This subject is discussed below. The value of X given by eqn (13) or more generally by eqn (12) for $c = 120$ seconds, can be used for spare intersection capacity calculations which is also discussed below.

X AS A MEASURE OF SIGNALISED INTERSECTION PERFORMANCE

14. The ARRB (1968) method recognises the relationship between the Y value and the degree of saturation, but implicitly considers that green times must be calculated in order to determine its value (individual approach values, x). Then, probably with the intention of eliminating the need to calculate signal settings at the preliminary design stage, the Y value is used to decide if a signal design is reasonable. Another reason for the ARRB method not adopting the X value is the influence of the U.S. *Highway Capacity Manual* (HRB 1965) which used the 'load factor' (proportion of fully saturated cycles) concept to define level of service at signalised intersections.

15. As expressed in ARRB (1968) and Miller (1968), the U.S. *Highway Capacity Manual* (HRB 1965) describes the level of service or operating characteristics of all road traffic facilities as a function of the ratio of flow to capacity (degree of saturation) except for signalised intersections. This seems to be due to the methods of designing traffic signals developed in the U.S. which are fundamentally different from the U.K. and Australian methods (for a review, see Department of Roads and Traffic 1974). However, there is no specific reason to justify the exclusion of signalised intersections when relating levels of service (operating conditions) to the degree of saturation. Robinson (1976) suggests that the load factor values given in the *Highway Capacity Manual* to define signalised intersection service levels should be treated only as approximations due to the limited research substantiation and refers to May and Pratt (1968) who studied the relationship between delay and load factor by means of simulation. More recently, Sutaria and Haynes (1977) found in a study of driver attitudes that level of service is better related to delay than the load factor concept used in the *Highway Capacity Manual*.

16. Both Webster (1958) and Miller (1968) delay formulae show a strong relationship between delay and the degree of saturation. Fig. 2 shows the typical pattern of change in delay (calculated from the Miller formula) as a function of x for various values of c , a and sg . Rapid increases in delay at degrees of saturation above 0.80 to 0.90 is seen in Fig. 2, which is caused by the random delay component (i.e. resulting from the queueing behaviour). The random delay component in both the Webster and Miller relationship is essentially a function of the degree of saturation. In Miller's delay formula, average overflow queue, E_z , is the determining factor. In Fig. 3, graphs showing E_z as a function of x for values of $sg = 10, 20$ and 60 are given. Fig. 4 presents graphs showing the proportion of oversaturated cycles, p_s , as a function of x for the same values of sg .

17. The recommended definitions of levels of service for signalised intersections as a function of the degree of saturation are illustrated in Figs 1 to 4. The definitions are also presented in Table II together with the definitions given in CRB (1974) for 'interrupted flow conditions'. Levels of service resulting from various combinations of (c , Y , L) are indicated in Table I.

18. The relationship between X and level of service may be regarded as somewhat arbitrary and subjective

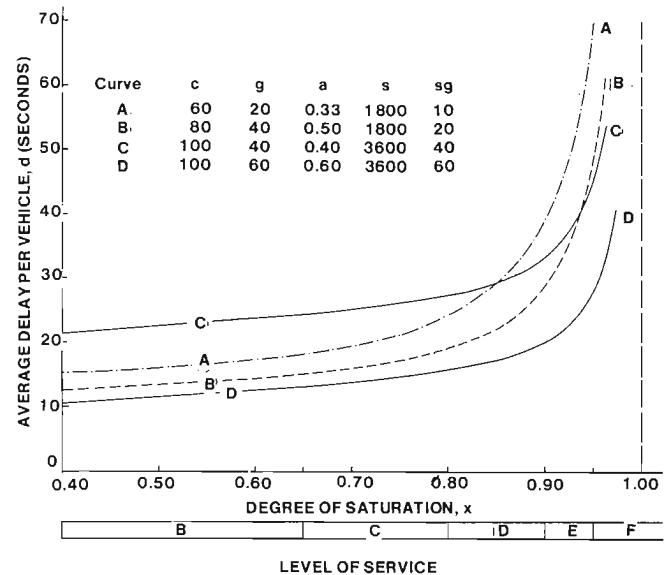


Fig. 2 — Average delay as a function of the degree of saturation

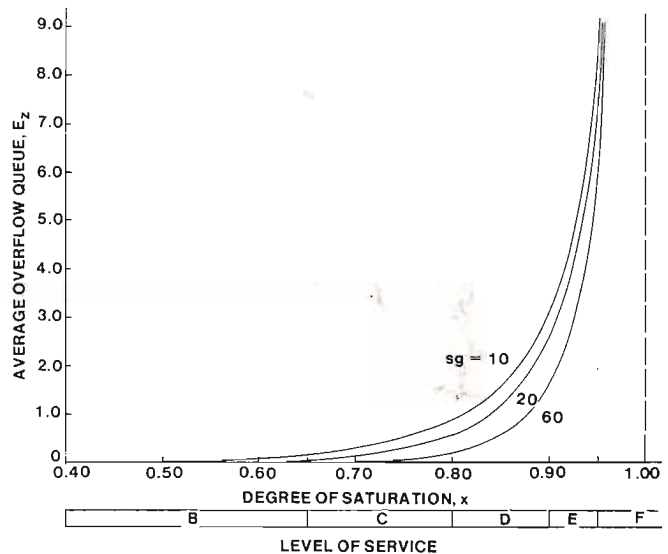


Fig. 3 — Average overflow queue as a function of the degree of saturation

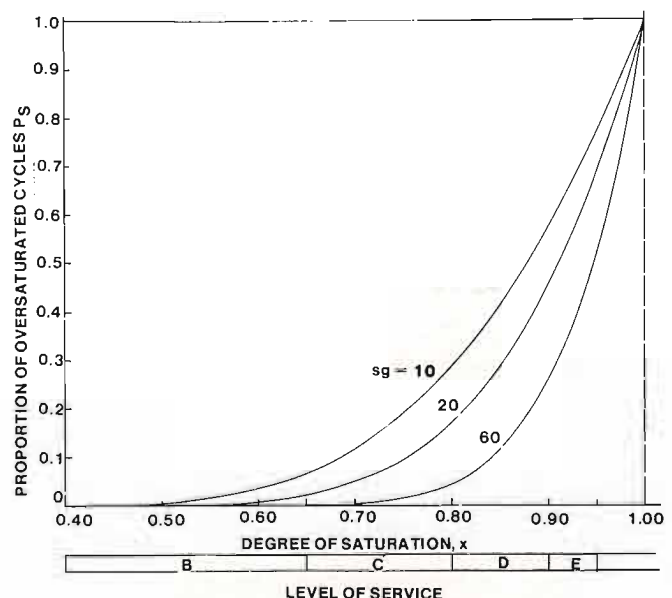


Fig. 4 — Proportion of oversaturated cycles as a function of the degree of saturation

and this is inevitable since level of service is a qualitative definition of operating conditions. However, the recommended definitions have been arrived at after careful consideration of:

- (a) the pattern of change in intersection performance characteristics such as delay, overflow queue, proportion of oversaturated cycles, etc. with respect to increasing degree of saturation; and
- (b) what combinations of signalised intersection characteristics (c, Y, L) result in certain values of X as indicated in *Table I*.

The property of a rapid increase in any of the operating characteristics at degrees of saturation above 0.80 to 0.90 as seen in *Figs 2 to 4* has been used to choose a maximum acceptable degree of saturation for design purposes. This is discussed below.

SELECTION OF A MAXIMUM ACCEPTABLE DEGREE OF SATURATION

19. The capacity condition for a signalised intersection is usually expressed as $X < 1.0$, but in practice there is an acceptable maximum degree of saturation, X_m which is less than 1.0 because traffic conditions become unstable, speeds drop and long queues develop when flow approaches capacity. Given the relationship between level of service and X , the choice of the value of X_m depends on the acceptable design level of service. This corresponds to the service volume concept of the U.S. *Highway Capacity Manual* (HRB 1965). It is recommended that such a value of X_m rather than a specified global value of an upper limit on Y is used for signal design purposes for the following reasons.

- (a) The intersection degree of saturation X is a direct measure of intersection performance as manifested by delay — X , proportion of oversaturated cycles — X , overflow queue — X , etc. relationships.
- (b) The intersection flow ratio, Y alone is not a sufficient measure of intersection performance as discussed above and illustrated by means of numerical examples given in the Appendix.

20. Webster and Cobbe (1966) used a value of $X_m = 0.90$ in the calculations of (a) reserve intersection capacity and (b) cycle time for the critical intersection in a network of linked signals. Recently, Logie (1977) reported a signal network design model in which a value of $X_m = 0.80$ was used with a fixed cycle time. The value of 0.90 corresponds to the upper limit of level of service D , i.e. the onset of unstable flow conditions as described in *Table II*. However, a slightly higher value can sometimes be used, e.g. at a critical intersection which forms part of a linked signal system, or a lower value may be chosen if a somewhat better level of service is aimed for. A range of X from 0.80 to 0.90 can be considered for design purposes. It is recommended that the value of 0.90 is adopted as a general purpose maximum acceptable degree of saturation and under special circumstances up to a value of 0.95 (upper limit of level of service E as an absolute maximum value) could be chosen. Allsop's (1976) computer program SIGCAP which allows for the use of different values of maximum acceptable degrees of saturation for different approach roads (or movements) should be mentioned here.

21. Due to the relationship between X and cycle time c as given by eqn (9), the value of X_m determines the minimum value of cycle time as discussed below. The choice of $X_m = 0.90$ as a general purpose design value has been arrived at after consideration of the minimum cycle time requirements (discussed below) as well as level of service requirements mentioned above.

22. The ARRB method of defining an upper limit on the Y value is based on the probability of clearing queues (ARRB 1968, p. 6). However, there seems to be a confusion in the interpretation of the load factor of the U.S. *Highway Capacity Manual* (HCM). Miller (1968, p. 8) states that HCM recommends the use of $p_o = 0.85$, where p_o is the probability of the queue being exhausted. The HCM (HRB 1965, p. 17) defines the load factor as 'the ratio of the total number of green intervals that are fully utilised by traffic during the peak hour to the total number of green intervals for that approach during the same period'. Obviously, this is the proportion of fully saturated cycles which can be approximated by $p_s = 1 - p_o$, rather than p_o . In fact, p_s is less than the load factor: p_o corresponds to cycles in which the overflow queue is zero ($g \geq g_s$, where g_s is the saturated portion of green time) and p_s corresponds to oversaturated cycles ($g < g_s$). In other words, p_o includes cycles where the overflow queue is zero but the queue has been just cleared, i.e. $g = g_s$ (fully utilised green interval). Assuming that a proportion β of p_o represents such saturated cycles, the load factor is

$$\alpha = p_s + \beta p_o = \beta + (1 - \beta) p_s$$

and the proportion of 'oversaturated' cycles is

$$p_s = (\alpha - \beta) / (1 - \beta)$$

which is less than the load factor, α . In a later publication, Miller (1969) proposed the formula $\alpha = \exp(-1.3\phi)$, where $\phi = \sqrt{sg} (1-x)/x$, as a good approximation for the load factor using the simulation results of May and Pratt (1968) which were for $sg = 10$. A comparison of this formula with Miller's formula for the proportion of oversaturated cycles, $p_s = 1 - p_o = \exp(-1.58\phi)$ indicates that the difference between p_s and α is in the range from 5 to 60 per cent for x from 0.96 to 0.50 (smaller difference for higher values of x).

23. The HCM (HRB 1965, p. 130) states that 'theoretically a load factor of 1.0 would represent capacity ...' but in practice 'a load factor range of 0.7 to 1.0 is more realistic'. It then recommends the use of 0.85 for isolated intersections as a general purpose design

value. The corresponding p_s value can be estimated assuming a range of β , e.g. for $\beta = 0.20$ to 0.50, $p_s = 0.81$ to 0.70, or using the Miller formula, $p_s = 0.82$ is found. It can be seen from *Fig. 4* that this corresponds to rather high degrees of saturation. On the other hand, the use of a small load factor, hence low values of p_s , e.g. $p_s = 0.15$ as effectively used in Miller (1968, p. 8), would lead to adopting small values of maximum acceptable degree of saturation and unduly long cycle times would result. It should also be noted that different degrees of saturation correspond to a given load factor, or p_s value depending on the number of vehicles which can be served per cycle. For example, for $p_s = 0.40$, x is about 0.85 for $sg = 10$, whereas it is about 0.93 for $sg = 60$. This means that for narrow approaches where few vehicles are served per cycle (single-lane minor roads), a lower degree of saturation should be used than

TABLE II

**DEFINITIONS OF LEVELS OF SERVICE FOR SIGNALISED
INTERSECTIONS AS A FUNCTION OF THE DEGREE OF SATURATION**

Level of Service	Description	Degree of Saturation	
		Recommended	CRB (1974)*
A	Free flow (almost no delay)	< 0.40	0.45-0.60
B	Stable flow (slight delay)	0.40-0.65	0.60-0.70
C	Stable flow (acceptable delay)	0.65-0.80	0.70-0.80
D	Approaching unstable flow (tolerable delay)	0.80-0.90	0.80-0.90
E	Unstable flow (intolerable delay)	0.90-0.95	0.90-1.00
F	Forced flow (congested)	Not meaningful (> 0.95)	Not meaningful (> 1.00)

* Specified for 'interrupted flow conditions'

for multi-lane approaches, if the load factor is utilised for the design of signals. This does not necessarily lead to an optimum design. An inspection of Fig. 3 shows that the average overflow queue is quite small for degrees of saturation less than 0.90. If it is accepted for design purposes that a maximum overflow queue of about two vehicles per phase (on average) corresponds to the upper limit of tolerable delay then this would give maximum acceptable degrees of saturation between 0.87 and 0.91 (for $sg = 10$ to 60). It is seen from Figs 3 and 4 that the recommended general purpose design value of $X_m = 0.90$ corresponds to p_s values in the range from 0.20 to 0.60 (hence load factors of about 0.25 to 0.70) and it permits higher proportions of saturated cycles and longer overflow queues for minor approach roads. It is seen that this is opposite to the result of specifying a 'load factor' for design purposes. However, it is not necessary to use the same X_m value for all movements. A method is described below which allows for the use of a different maximum acceptable degree of saturation for each phase, the choice of which depends on particular design objectives.

VARIOUS SIGNAL DESIGN CALCULATIONS BASED ON THE USE OF X_m

MINIMUM CYCLE TIME AND AN ALTERNATIVE METHOD FOR SIGNAL SETTINGS

24. Webster and Cobbe (1966) define the minimum cycle time as

$$c_m = \frac{L}{1 - Y} \quad (14)$$

This is an absolute minimum cycle time which corresponds to a degree of saturation $X = 1.0$, i.e. to a theoretical capacity value and therefore should not be adopted for use in signal design practice. The correct value of minimum cycle time should correspond to the maximum acceptable degree of saturation, X_m as described below. From eqn (9), the value of the cycle time which yields a degree of saturation x is given by

$$c_x = \frac{xL}{x - Y} = \frac{L}{1 - Y/x} \quad (15)$$

Since $x < X_m$ by definition of X_m , the minimum value of cycle time corresponds to the maximum value of x ,

i.e. X_m . The minimum practical cycle time is therefore given by

$$c_m = \frac{X_m L}{X_m - Y} = \frac{L}{1 - Y/X_m} \quad (16)$$

and for the recommended value of $X_m = 0.9$

$$c_m = \frac{0.9 L}{0.9 - Y} = \frac{L}{1 - Y/0.9} \quad (17)$$

25. It can be seen from eqn (16) that the required minimum value of cycle time, c_m increases as the maximum acceptable degree of saturation, X_m decreases. In Figs 5 and 6, graphs are given which show cycle times for $X_m = 0.80, 0.85$ and 0.90 as well as optimum cycle times from eqn (2) using one-lane and three-lane minimum saturation flow values of 1700 and 5000 tcu/h, respectively. It is seen that minimum cycle times required for achieving $X_m = 0.80$ and 0.85 are larger than the optimum cycle time for the critical values of Y (those greater than about 0.65) most of the time. On the other hand, c_m for $X_m = 0.90$ exceeds the optimum cycle time only for low values of L and high values of minimum saturation flow and this corresponds to only a limited range of operating conditions. The recommendation of $X_m = 0.90$ rather than smaller values has been based on this consideration in addition to the relation of X to the level of service discussed above (in particular the use of 0.80 would result in unduly long cycle times, hence increased delays to all traffic).

26. A lower limit on cycle time is particularly useful when small values of cycle time are desirable. Advantages of using small cycle times are:

- delays to pedestrians are decreased;
- delays to minor vehicle movements are decreased;
- delays to buses are decreased where bus only lanes are available and bunching of buses is decreased in general;
- total delay to traffic in a network of co-ordinated signals is decreased if the cycle time at the critical intersection (which is used as the common cycle time) is set to its minimum value, or double-cycling is used for minor intersections;
- queue lengths are decreased or dynamic queue storage capacities are increased; and
- effective saturation flows of lanes of limited length, e.g. right-turn slots are increased and better lane utilisation is achieved.

TABLE III

 PRACTICAL MINIMUM CYCLE TIMES (c_m)

Y	L = 10	L = 15	L = 20
0.30	30	30	40
0.40	30	40	50
0.45	30	40	50
0.50	30	50	60
0.54	40	50	60
0.58	40	50	60
0.60	40	50	70
0.62	40	60	70
0.64	50	60	70
0.66	50	60	80
0.68	50	70	80
0.70	60	70	90
0.72	60	80	100
0.74	60	90	110
0.76	70	100	
0.78	80	110	
0.80	90		
0.82	110		

27. Table III gives practical minimum cycle times c_m for various values of Y and L. The values given in Table III (rounded to the nearest 10 seconds) satisfy two criteria:

- the resulting degree of saturation $X \leq 0.90$; and
- the ratio of minimum cycle time to optimum cycle time, $c_m/c_o \geq 0.75$ to assure that total delay is not increased significantly. The latter criterion is based on Webster's (1958) work. If the minimum saturation flow for any of the representative movements (s in eqn (2)) corresponds to more than one lane, say 3000 tcu/h, then the cycle times given in Table III can be used as practical optimum cycle times for preliminary design purposes (i.e. instead of those given in Table I which correspond to single-lane minimum saturation flow value of 1800 tcu/h).

28. In some cases, it may be desirable to use a maximum acceptable degree of saturation x_m for an individual phase, which is different from the intersection value X_m . In order to generalise this, define a minimum acceptable green time g_m (e.g. see Webster and Cobbe (1966) for its use in designing linked signal systems):

$$g_m = a_m c \quad (18)$$

where $a_m = y/x_m$ is the minimum green time ratio required; y and x_m are the flow ratio and the maximum acceptable degree of saturation. Then the total green time required is

$$\sum g_i = \sum a_i c = \sum \frac{y_i}{x_{mi}} c = c - L \quad (19)$$

where g_i, a_i , etc. are for phase i ($= 1$ to n). Solving this equation for c, the cycle time is found

$$c = \frac{L}{1 - \sum a_i} = \frac{L}{1 - \sum \frac{y_i}{x_{mi}}} \quad (20)$$

It is seen that this is a generalisation of eqn (16). Once the cycle time is determined, the green time for each phase can then be calculated from eqn (18) using ($a_i = y_i/x_{mi}$) ratios already calculated for eqn (20). The method described here for calculating cycle time and green times could be used as an alternative to the Webster (or ARRB) method described by eqns (1) to

(3). This method also simplifies calculations if one or more phases are subject to minimum green constraints. In this case, fixed green time values are used for such phases in eqn (19). This method is illustrated in Example 2 in the Appendix.

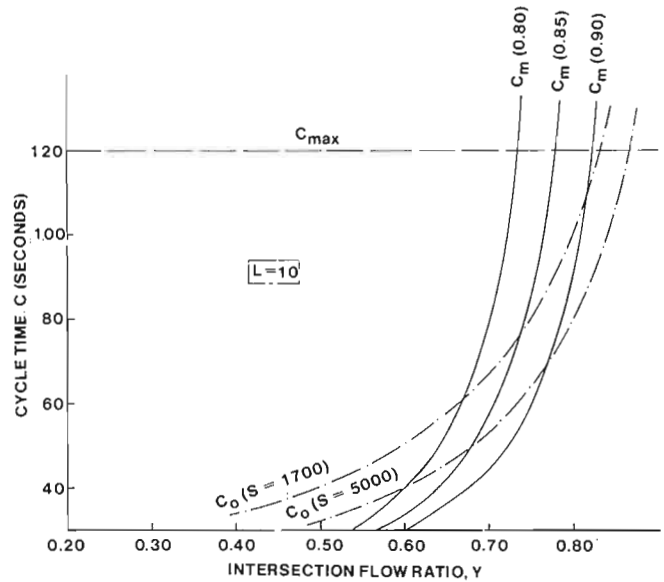


Fig. 5 — Comparison of various minimum and optimum cycle times (c_m and c_o) for 10-second intersection lost time

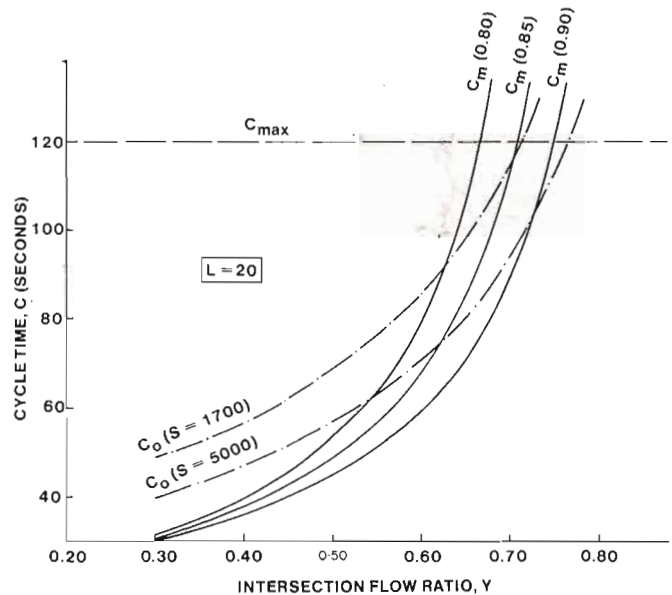


Fig. 6 — Comparison of various minimum and optimum cycle times (c_m and c_o) for 20-second intersection lost time

SPARE INTERSECTION CAPACITY

29. Percentage spare capacity (PSC) of the intersection as a whole is a useful measure of its life before the design level of service is reached. It can be calculated from

$$PSC = \left(\frac{X_m - X_p}{X_p} \right) 100 \quad (21)$$

where X_m = maximum acceptable degree of saturation and X_p = practical minimum degree of saturation for given intersection conditions. As discussed before, X_p corresponds to the maximum cycle time for given Y and L values. From eqn (12)

$$X_p = \frac{Y}{1 - L/c_{max}} \quad (22)$$

Using the recommended values $X_m = 0.90$ and $c_{max} = 120$ seconds, percentage spare capacity is

$$PSC = \left(\frac{0.9}{X_p} - 1 \right) 100 \quad (23)$$

where

$$X_p = \frac{Y}{1 - L/120}$$

(Values of X in Fig. 1 for given Y and L values are equivalent to X_p given by this formula.) Fig. 7 can be used to calculate percentage spare capacities. It should be noted that eqn (24) or Fig. 7 gives percentage spare capacities which are the same as those given by Webster and Cobbe's (1966) formula which uses a practical Y value.

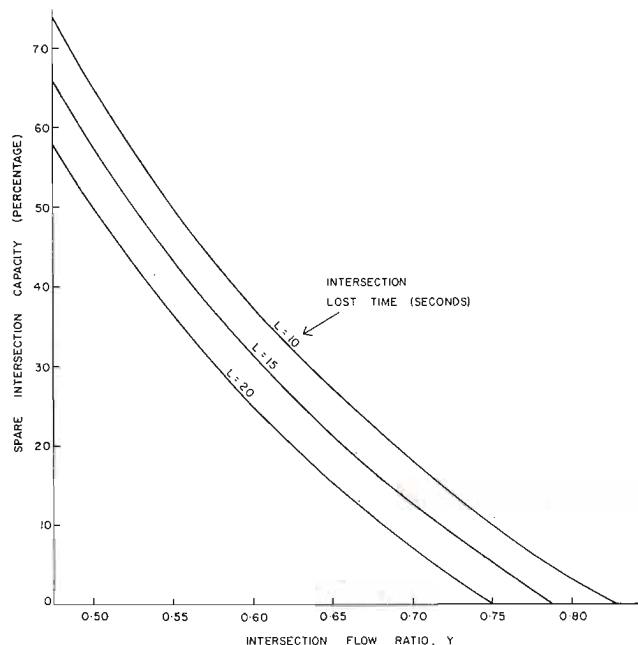


Fig. 7 — Percentage spare capacities

SIGNAL SETTINGS IN THE CASE OF A MINIMUM GREEN TIME

30. The use of intersection flow ratio Y as a measure of performance becomes misleading, particularly when one or more signal phases have green times which do not satisfy vehicle or pedestrian minimum time requirements. This is because the extra green time required to satisfy minimum green time requirement for a phase acts as a lost time for all other phases. Vehicle minimum green time may be 5 to 8 seconds and if a representative flow ratio y is less than 0.10, the green time of that phase is likely to have a small green time and it should be checked to ensure that it satisfies $g \geq g_{min}$. Pedestrian minimum green time depends mainly on the clearance distance. The green time for pedestrians crossing the road during that phase should be long enough to provide a minimum of 5 to 8 seconds 'Walk' period and a clearance ('Flashing Don't Walk') period which may be calculated from $(D/1.4)$ seconds, where D is the kerb-to-kerb crossing distance and 1.4 m/s is the pedestrian speed during clearance. Where the end of 'Flashing Don't Walk' period coincides with the end of green period, i.e. there is no overlap with amber period, the pedestrian minimum green time is $(5 \text{ to } 8 + D/1.4)$.

31. The method proposed for calculating signal settings when a minimum green time constraint is not satisfied is as follows. Suppose $g < g_{min}$ for a phase which has a representative flow ratio y' . In this case the green time for this phase should be set equal to g_{min} and signal settings should be calculated excluding this phase, i.e. the cycle time and green times for other phases should be calculated using modified intersection flow ratio and lost time values given by

$$Y' = Y - y' \quad , \quad L' = L + g_{min} \quad (24)$$

32. In cases where $g < g_{min}$ but g is close to g_{min} , the green time equal to g_{min} may not be long enough for that phase because of the longer cycle time resulting from the use of eqn (24). For this reason, a minimum acceptable green time should be calculated from eqn (18) using y' and x_m which are for the phase subject to minimum green setting (use $x_m = 0.90$ under normal circumstances) and c = cycle time resulting from calculations using eqn (24). If $g_{min} < g_m$ then g_{min} should be set equal to g_m and the calculations using eqn (24) should be repeated. This process should be reiterated as necessary. The method described in this section can be generalised using g_m from eqn (18) to replace g_{min} in eqn (24) as a 'set' green time (a value specified in terms of x_m , but still a function of c). To specify a 'fixed' green time in order to avoid iterations, a maximum cycle time can be used in eqn (18). For example, using $c_{max} = 120$ and $x_m = 0.90$, $g_m = 133.3 y'$ is obtained as a general formula (this means that $x < x_m$ for that phase if the resulting cycle time, c is less than c_{max}).

33. It should be noted that the method described in CRB (1974), which requires increasing all green times by a factor (g_{min}/g) when $g < g_{min}$, results in unduly long cycle times and it is difficult to justify this method. Example 2 in the Appendix is given to illustrate numerically the method proposed in this paper and compare it with the method suggested in CRB (1974).

TRAFFIC SIGNAL DESIGN PROCEDURE

34. Traffic signal design calculations should normally be based on the average weekday peak hour traffic flows in the design year. Separate calculations should be carried out for a.m. and p.m. peak periods for intersections in urban areas because traffic patterns differ significantly due to the directional nature of traffic movements (home-to-work in the morning and work-to-home in the evening). In particular, at T-junctions where an overlapping phase system is utilised significant difference should be expected between a.m. and p.m. peak conditions as indicated in Example 1 in the Appendix.

35. If the purpose of calculations is to adjust signal timings at an existing intersection, the current year should be taken as the design year. Actual flow counts should be used in this case. For purposes of intersection improvements through minor modifications, design year may be five to ten years hence. Existing traffic flows should be increased by allowing for normal traffic growth. Changes in traffic flow pattern resulting from expected changes to the adjacent road network should also be taken into account. If a completely new intersection is being designed, a design year 15 to 20 years hence

may be appropriate. Predicted peak hour flows from traffic assignment results should be used in this case.

THE PROCEDURE

36. The recommended signal design procedure is as follows.

- (a) Take the peak hour flows which the intersection is to be designed to accommodate (repeat the procedure for both a.m. and p.m. peak hour conditions).
- (b) Take a trial intersection layout and signal phasing.
- (c) Calculate tcu factors from flow data to allow for traffic composition and turning flows (if there are opposed right-turners, use $E_{rr} = 2.9$ as a first approximation).
- (d) Estimate saturation flows in tcu/h. Calculate saturation flows in veh/h using tcu factors calculated in (c). The use of flows and saturation flows in vehicles rather than tcu's (or pcu's) is suggested since this has the advantages of being directly relevant to the real-life counts and preventing confusion caused by bus occupancy figures when bus priority is allowed for in signal calculations. If possible, use actual saturation flows measured in the field for existing signalised intersections. When estimating saturation flows, consider cases of lane under-utilisation carefully (parked vehicles, bus stops, limited lane lengths, exclusive turn lanes at exit side, etc.).
- (e) Calculate the flow ratio y for each movement (approach). Select the representative value of y for each phase and calculate the intersection flow ratio Y as the sum of representative y values (consider overlapping phases in this process).
- (f) Calculate total lost time per cycle.
- (g) Calculate the practical minimum degree of saturation for given Y, L values:

$$X_p = Y / (1 - L/120)$$

- (h) Compare X_p with X_m (acceptable maximum degree of saturation) using $X_m = 0.90$, if $X_p > 0.90$ intersection operating conditions are not satisfactory, take measures for improvements (see below).
- (i) If $X_p < 0.90$, calculate percentage spare capacity. If the spare capacity is low, search for measures for improvement.
- (j) In order to determine the intersection cycle time, c , calculate optimum cycle time, c_o and/or a practical minimum cycle time, c_m to satisfy $X \leq 0.90$ and $c_m/c_o \geq 0.75$. If there are reasons to use a cycle time smaller than c_o , choose a cycle time which satisfies $c_o > c \geq c_m$. In all cases use a maximum cycle time of 120 seconds (absolute maximum = 140 seconds for intersections under stress and with no queue storage and lane under-utilisation problems). The cycle time value may be approximated to the nearest 10 seconds for convenience.
- (k) In the case of network signal control, calculate X_p for all intersections and choose the intersection with the largest X_p as the critical one. Calculate the minimum cycle time c_m at this intersection and determine the common network cycle time c on

this basis. If double-cycling is to be used at minor intersections, calculate the degree of saturation X for these intersections using a cycle time of $c/2$, where c = common network cycle time, and check if $X \leq X_m$ ($= 0.90$) is satisfied.

- (l) For the intersection cycle time chosen, calculate optimum green times (approximated to the nearest second).
- (m) Check green times against vehicle and pedestrian minimum green time constraints. If one or more minimum green time constraints are not satisfied, calculations of signal settings should be repeated using modified values of intersection flow ratio and lost time Y' and L' .
- (n) Check assumed E_{rr} values for opposed right turns. Repeat calculations if values differ significantly. Also check that the right-turn demand is satisfied ($x < X_m$ for the right-turn movement where x is calculated using the effective saturation flow, s/E_{rr}).
- (o) Check queue storage capacities and modify queue storage distances if required (increase length of right-turn slots, introduce parking restrictions, etc.) so that lane under-utilisation is minimised and/or the assumed saturation flows can be realised.
- (p) Calculate traffic performance characteristics (degree of saturation, spare capacity, total delay, etc.) for the purposes of alternative analysis and cost-benefit analysis. In this context, the procedure should be repeated for several alternative designs (intersection layouts and phasing systems) and the best design should be selected. The choice of the best design should normally be based on cost-effectiveness criterion.

MEASURES FOR IMPROVING INTERSECTION OPERATING CONDITIONS

37. The following are various means of improving intersection operating conditions:

- (a) provision of additional lanes, including turning lanes;
- (b) prohibition of one or more right turns;
- (c) provision or omission of a separate right-turn phase.
- (d) prohibition of parking or standing on approaches or exits;
- (e) suitable locations for bus stops and use of bus bays;
- (f) improvements to intersection geometry (increased geometric design standards, e.g. turning radii, and use of traffic islands and pavement markings);
- (g) provision of double left-turn lanes under signal control for heavy left-turn flows; and
- (h) better signal phasing and timing arrangements.

SUMMARY AND RECOMMENDATIONS

38. In this paper various improvements have been proposed to traffic signal design method with particular reference to the method used in Australia. A comprehensive treatment of the subject has been presented

with the objective of being of direct use to the practising traffic engineer. It has been recommended that:

- (a) the intersection degree of saturation X rather than the flow ratio Y is used as a simple measure of intersection performance for preliminary design purposes; and
 - (b) a maximum acceptable degree of saturation X_m is used instead of an upper limit on Y in traffic signal design. A value of $X_m = 0.90$ has been suggested based on the definitions of service levels as a function of X and the minimum cycle time requirements of various X_m values. The recommended definitions of service levels have been based on analyses of the patterns of change in various intersection performance characteristics, namely delay, overflow queue and the proportion of oversaturated cycles with respect to increasing values of X and the consideration of what combinations of signalised intersection characteristics (c , Y , L) result in certain values of X .
39. A simple formula has been given for calculating the intersection X value without the need to calculate phase green times. Methods have been described for:
- (a) calculating a minimum cycle time which yields an acceptable maximum degree of saturation (also subject to a delay criterion);
 - (b) calculating spare intersection capacity based on the use of X_m and a maximum cycle time (120 seconds); and

- (c) signal design calculations when a minimum green time constraint is not satisfied.

40. A traffic signal design procedure has also been described which incorporates the improvements proposed in this paper. Numerical examples are given in the Appendix in order to illustrate the issues discussed in the paper. Example 1 presents a case where the intersection flow ratio Y is greater than 0.75 but the intersection operating conditions are satisfactory. It also illustrates the method for calculating green settings in the overlap (split) phase case where a common mistake is made in practice. Example 2 demonstrates the proposed method for signal setting calculations when there is a phase subject to minimum green time constraint due to pedestrians. It introduces a case where the intersection Y value is lower than that in Example 1 (0.68 against 0.77) but the resulting intersection performance values are very similar. Example 3 presents an alternative analysis which involves a detailed look at signal phasing, lane arrangement and cycle time options in traffic signal design. It is suggested that the interested reader should note various results of Example 3 which have both practical and theoretical importance (not discussed in the text).

41. It is recommended that the methods proposed and the procedure described in this paper are incorporated in traffic signal design practice in Australia and elsewhere as relevant. The method for calculating signal settings described in para. 28 is also recommended for use in practice because of its various advantages.

APPENDIX A

NUMERICAL EXAMPLES

EXAMPLE 1: Y GREATER THAN 0.75

42. Consider the three-phase system for a T-junction illustrated in *Fig. 8a*. Flow ratios for traffic movements 1 to 4 are

$$y_1 = 0.19, y_2 = 0.58, y_3 = 0.13, y_4 = 0.25.$$

The minimum saturation flow used in the calculation of these values is 1700 tcu/h and 5 seconds lost time per phase (effective intergreen) is assumed. Overlapping phase (A + B) must be taken into account to calculate the intersection flow ratio:

$$y_{A+B} = y_2 = 0.58, y_A + y_B = y_3 + y_4 = 0.38$$

and since $y_{A+B} > y_A + y_B$ this is effectively a two-phase system.

Therefore

$$Y = y_{A+B} + y_C = 0.58 + 0.19 = 0.77$$

On the basis of accepted practice this system would be rejected since Y is above the absolute limit 0.75 specified in ARRB (1968). However, it can be seen that the system operates satisfactorily. Total lost time is $L = 10$ seconds (two phases effectively) and optimum cycle time is $c_o = 88$ ($s = 1700$ tcu/h). The cycle time which yields 90 per cent saturation is 69 seconds whereas the minimum cycle time if $X_m = 0.80$ was accepted would

be 267 seconds, i.e. any cycle time less than this value would be unacceptable since the resulting degree of saturation would be greater than 80 per cent. Spare intersection capacity (based on $c_{max} = 120$, $X_m = 0.90$) is 7 per cent. Choosing a cycle time of $c = 90$ seconds, the resulting degree of saturation would be $X = 0.87$ which corresponds to a level of service D .

43. Green settings are calculated as follows. Total available green time is $(c - L) = 90 - 10 = 80$, therefore $g_{A+B} = (80/0.77) 0.58 = 60.3$ and $g_C = 19.7$ is found. Choose $g_{A+B} = 60$ seconds and $g_C = 20$ and split g_{A+B} between phases A and B . Total available green time is $g_{A+B} - 5 = 55$ (due to the lost time in changing from A to B). Using $y_A + y_B = 0.38$ as the total flow ratio, $g_A = (55/0.38) 0.13 = 18.8$ and $g_B = 36.2$ is found. Choose $g_A = 19$ and $g_B = 36$. The results are shown for individual movements in *Table IV*. Effective and actual signal timings are illustrated in *Figs 8b* and *8c* using two different methods of presentation. Actual green times have been calculated assuming 2 seconds starting/stopping lost time per phase. An inspection of the degrees of saturation for individual movements in *Table IV* indicates the differences from $X = 0.87$. Movement 1 is due to the rounding of green time. Movements 3 and 4 are due to the spare time available because the heavier movement 2 determines the green times for movements 3 and 4.

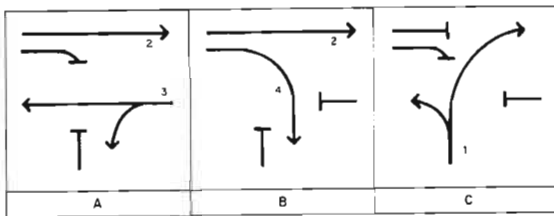


Fig. 8 (a) — Signal phases and vehicle movements

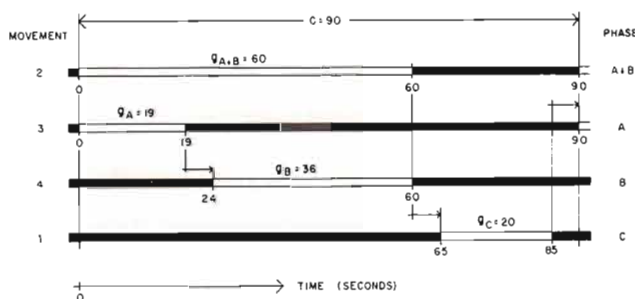


Fig. 8 (b) — Effective signal timings

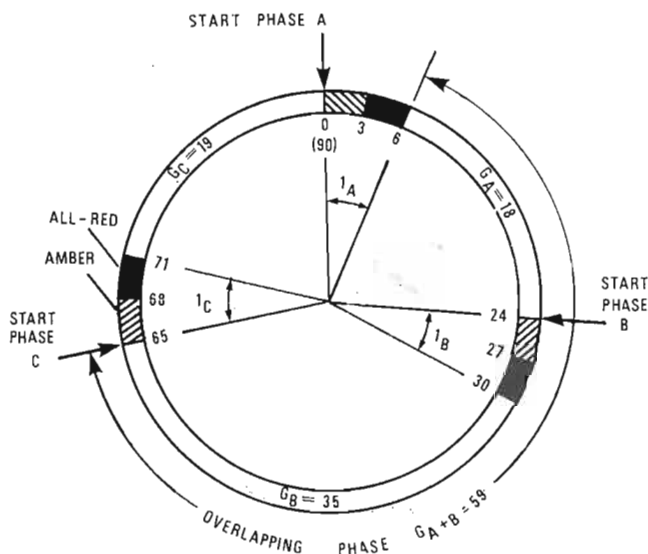


Fig. 8 (c) — Actual signal timings

Fig. 8 — Signal phasings and timings for Example 1

TABLE IV

RESULTS FOR EXAMPLE 1

<i>Movement</i>	<i>Phase</i>	<i>Flow Ratio</i> y	<i>Green Time</i> g	<i>Green Ratio</i> $a = g/c$	<i>Degree of Saturation</i> $x = y/a$
1	C	0.190	20	0.222	0.86
2	A + B	0.580	60	0.667	0.87
3	A	0.130	19	0.211	0.62
4	B	0.250	36	0.400	0.63

44. As a note on the importance of repeating signal design calculations for both a.m. and p.m. peak conditions, assume that the flows represented by movements 2 and 3 above are reversed and the resulting flow ratios are $y_2 = 0.13$ and $y_3 = 0.58$ and other flow ratios remain the same. In this case $y_{A+B} = 0.13 < (y_A + y_B) = 0.83$, hence this is effectively a three-phase system with $Y = y_A + y_B + y_C = 1.02$ and $L = 15$ seconds. Assuming a

cycle time of 120 seconds, the degree of saturation $X = 1.17$ which indicates grossly over-saturated conditions (obviously Y alone is sufficient to indicate how bad conditions are in this case).

EXAMPLE 2: MINIMUM GREEN TIME

45. Consider a three-phase system with $y_A = 0.43$, $y_B = 0.20$ and $y_C = 0.05$ and a total lost time of 15 seconds. Minimum saturation flow used in the calculation of these flow ratios is 2700 tcu/h. Minimum green time required for pedestrians crossing during phase C is 14 seconds. The method described in paras 31 and 32 is implemented below for calculating signal settings and the resulting performance values.

46. Using $Y = 0.68$, $L = 15$ and $s = 0.750$ tcu/s, the optimum cycle is found $c_o = 78$ seconds and the minimum cycle time to yield $X_m = 0.90$ is found: $c_m = 61$ seconds. Choose a cycle time of 70 seconds due to the existence of pedestrians. Green times are then $g_A = 34$, $g_B = 17$ and $g_C = 4$ seconds. However, since $g_C < g_{Cmin} = 14$, calculations must be repeated by setting $g_C = 14$. Using $y' = y_C$ and $g_{min} = g_C$ in eqn (24),

$$Y' = Y - y_c = 0.68 - 0.05 = 0.63$$

and

$$L' = L + g_C = 15 + 14 = 29 \text{ seconds}$$

Using Y' and L' , the optimum and minimum cycle times are $c_o = 115$ and $c_m = 97$ seconds. Choosing $c = 110$ seconds, the total available green time (for phases A and B) is $c - L' = 81$ and the green times are $g_A = 55$ and $g_B = 26$ seconds (and $g_C = 14$ minimum). The resulting intersection degree of saturation is $X = Y'/(1 - L'/c) = 0.86$. Using $X_p = Y'/(1 - L'/120) = 0.831$ and $X_m = 0.90$, the spare intersection capacity is 8.3 per cent. Compared with the results of Example 1 in terms of Y , X and PSC it can be seen that although the system in Example 2 has a lower Y value (0.68 against 0.77) the resulting X and PSC are very close. In order to check if $g_{min} = 14$ is sufficient with $c = 110$, the minimum acceptable green time is calculated as $g_m = (0.05/0.90) 110 = 6.1$. Since $g_{min} > g_m$, this is satisfactory.

47. If the CRB (1974) method for calculating revised signal settings due to minimum green time g_C was used, the following results would be obtained. Since $g_{Cmin}/g_C = 14/5 = 2.8$, $g_A = 2.8 \times 34 = 95$ and $g_B = 2.8 \times 17 = 48$ and this would result in a cycle time of $c = 95 + 48 + 14 + 15 = 172$ seconds. This example demonstrates that the CRB (1974) method is not a valid one.

48. In order to illustrate the method described in para. 28, assume that different maximum acceptable degrees of saturation are required for phases A and B, e.g. 0.85 and 0.92, respectively. Phase C is subject to the minimum green constraint of 14 seconds as above. Green time ratios required for phases A and B are $0.43/0.85 = 0.506$ and $0.20/0.92 = 0.217$ respectively. Therefore, eqn (19) is

$$0.506c + 0.217c + 14 = c - 15$$

Solving this equation, the cycle time is found: $c = 105$ seconds. The green times for phases A and B are then calculated as $g_A = 0.506 \times 105 = 53$ and $g_B = 0.217 \times 105 = 23$ seconds.

EXAMPLE 3: DESIGN ALTERNATIVES

49. This example is based on Examples 5 to 7 of ARRB (1968). The example is developed here in detail in the form of an alternative analysis. It is suggested that the interested reader study the various aspects of traffic signal design which are brought forward in this example. These have both practical and theoretical importance and hence further work is recommended. The original flow and saturation flow data have been used for the purpose of comparison with the treatment in ARRB (1968). A 'link' description of the intersection is used here for better description of the individual movements with respect to their physical (approach, lane, turning type) and signal phasing characteristics. The link description adopted is shown in Fig. 9b. Intersection data are given in Figs 9a, 9b and Table V. The signal design for this intersection is critically dependent on the treatment of the North approach. The lanes are combined together in ARRB (1968) whereas two separate links are used in the present paper in order to take into account the lane distribution of through vehicles on this approach. Because of very light right-turn flows on other approaches, flow characteristics are assumed to be constant for all alternatives and a single link is used to describe each approach road.

50. In analysing this problem, several alternatives have been developed. As shown in Fig. 9c, two-phase and three-phase systems have been considered. Saturation flow in veh/h has been calculated using tcu factors determined by traffic composition and turning flows employing the tcu equivalents given in ARRB (1968). The lane arrangements, flow, saturation flow and flow ratio values for each approach are given in Table V, in which the data for the North approach (links 3 and 5 combined) are for Alternative 1. However, links 3 and 5 have been modelled separately in order to predict the effects of changes in lane utilisation on the North approach. The data for links 3 and 5, which change for each alternative as a function of the number of available lanes, signal phasings and timings (which determine the right-turning vehicle equivalent, E_{rt}) and assumed or predicted lane distribution, are given in Table VI. In two-phase cases where right turners are subject to gap acceptance, E_{rt} values and the resulting saturation flows have been found as equilibrium values after several iterations of signal calculations. For all alternatives it has been assumed that the right-turn lane on link 5 is long enough to provide for sufficient storage capacity resulting in no loss of saturation flows on the North approach. (For convenience, the right-turning vehicle equivalent can be calculated from

$$E_{rt} = \frac{1.5}{f(1-x)/(1-y) + (4.5/g)}$$

where x and y are calculated for the opposing traffic using the values of q , s , g and c defined as in ARRB (1968).)

51. The alternatives are described below and the calculated values of flow ratio, cycle time, degree of saturation, delay, etc. for each alternative are given in Table VII. Delays have been calculated from the Miller formula.

Alternative 1

52. This is the original ARRB (1968) solution (Example 5, p. 29), the only difference being in the value of

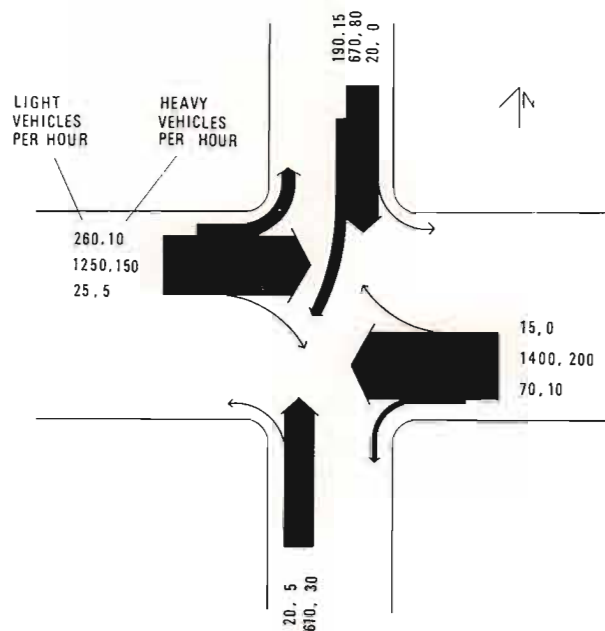


Fig. 9 (a) — Intersection flows

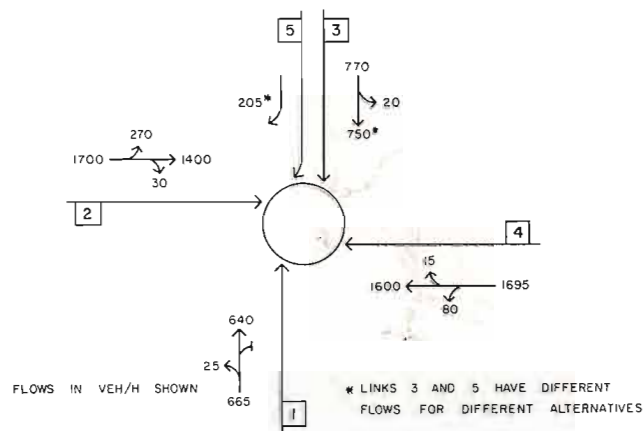


Fig. 9 (b) — Link diagram

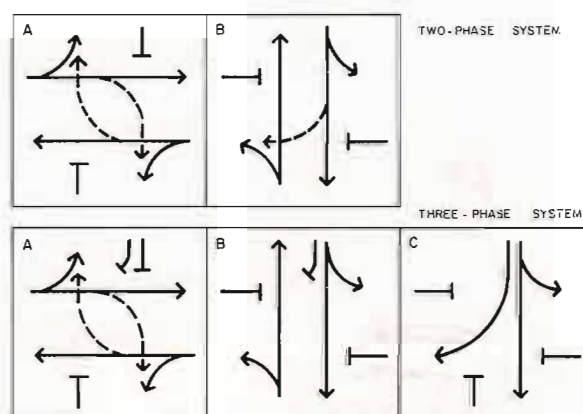


Fig. 9 (c) — Phasing diagrams

Fig. 9 — Intersection flows, signal phasing and link diagrams for Example 3

E_{rt} (original solution uses 2.9, whereas the present paper uses the calculated value of 3.0). This is a two-phase system with two lanes available on the North approach. The solution is based on 'equal flow ratios' on links 3 and 5 which is a result of combining the two lanes together to calculate the saturation flow and the flow

ratio for the approach as a whole, i.e. $y_3 = y_5 = y_N$. The lane distribution given in *Table VI* for Alternative 1 must exist to produce this result. In this distribution, 145 through vehicles per hour share the right lane with the right-turning vehicles.

Alternative 1B

53. The delays given in *Table VII* for Alternative 1 show that the delay to vehicles in the right lane (link 5) would be considerably higher than the delay to vehicles in the left lane (link 3). However, one would expect that a smaller number of through vehicles would use the right lane in such a situation in practice, i.e. through vehicles would choose the left lane instead of queueing behind the right-turning vehicles waiting for gaps in the opposing stream. In order to predict this effect, a 'lane assignment' principle can be stated as 'when subject to a lane choice, each driver will use the lane which gives the least delay'. This results in a lane distribution which gives equal delay per vehicle on each lane. After several iterations of signal calculations, the solution given in *Tables VI* and *VII* as Alternative 1B has been arrived at. In this solution, there are 80 through vehicles per hour using the right lane and the flow ratio of link 3 (left lane) is higher than the flow ratio of link 5 and hence becomes the representative flow ratio ($y_B = y_3 > y_5$). This flow ratio is higher than that in Alternative 1 and this results in a longer cycle time and a higher E_r value. The individual delays are equal on the two lanes of the North approach ($d_3 = d_5 = 27$ seconds per vehicle), but the total intersection delay is substantially higher than that in Alternative 1.

54. The difference between Alternative 1 and Alternative 1B is of both practical and theoretical importance. If an equal-delay solution represents the real life situation then the 'equal flow ratio' method is not valid. Therefore, the lane utilisation should be considered more carefully in cases of shared right-turn (opposed) and through-traffic lane at critical approaches in traffic signal design. Another point of importance is the difference in predicted delay when individual lanes are modelled separately and the approach is modelled as a whole. As mentioned in the note for *Table VII*, higher delays per vehicle are predicted when individual lanes are modelled separately. It has been found that this is a result of the random delay term of the delay formula (valid for both Miller and Webster formulae). More work is recommended on this subject.

Alternative 2

55. This alternative follows Example 6 given in ARRB (1968). A three-phase system (early cut-off of the South approach traffic) is considered which allows unopposed right turns from the North approach as shown in *Fig. 9c*. As in the original example, it is assumed that the right lane is for right turns only. It is further assumed that a red arrow is shown to right turners during phase B (otherwise adjustment must be made for departures of right turners by gap acceptance during this phase which would result in a shorter green time for phase C). The resulting intersection flow ratio of $Y = 0.82$ is rejected in ARRB (1968) as a too high value for satisfactory operating conditions. In *Table VII*, it is seen that this solution does not really correspond to congested conditions. With a cycle time of 110 seconds, the degree of saturation, X is 90 per cent (mainly due to the small value of lost time) and the

delays are not very high. However, the system has no spare capacity and measures to improve operating conditions should be sought if the three-phase system is to be used (e.g. for safety reasons).

Alternative 3

56. This is the same as Alternative 2 except for the addition of an extra lane on the North leg (link 3) to improve operating conditions. Y value is dropped to 0.64 and, although the lost time is increased to 15 seconds (Alternative 2 is 'effectively' a two-phase system as shown in *Table VII*), the cycle time is decreased and spare intersection capacity is increased substantially. It is interesting to note that the total intersection delay is almost the same as that in Alternative 1 (and less than that in Alternative 1B), but delay to right-turning vehicles is increased (also compared with Alternative 2). This suggests that a two-phase system would be desirable in terms of operating conditions and cost, but safety reasons and the provision of larger spare capacity (hence longer design life) would affect the choice in this case.

Alternative 4

57. This is a two-phase system as in Alternative 1 but link 3 has two lanes as in Alternative 3. The right lane (link 5) is an exclusive right-turn lane as in Alternative 3, but right-turns are subject to gap acceptance as in Alternative 1 (or 1B). It can be seen that the resulting intersection performance is better than that with the three-phase system of Alternative 3 (given the same layout) both in terms of individual and total intersection delays.

Alternative 4A

58. This is a variation on Alternative 4 to show the effect of using a smaller cycle time. Instead of using the optimum cycle time, the minimum cycle time is used in order to improve right-turn saturation flow. After several iterations of signal calculations an equilibrium solution is found in which E_r is decreased to 2.6 (from 3.0 in Alternative 4). A minimum cycle time of 40 seconds is used which results in an intersection performance significantly better than that in Alternative 3.

59. It is seen that a two-phase system with an additional lane on the North approach is the best solution in this case. Additional analyses of right-turn gap acceptance have been carried out for all two-phase alternatives, and it has been found that sufficient time is available for, or can be given to phase B which will create gaps in the opposing flow (link 1) for right turners from link 5. (When link 5 has the critical flow ratio for phase B, presence detection would be necessary to extend green until the right turners clear the approach in the case of vehicle-actuated signals.) For example, in Alternative 4, the saturated portion of phase B green time, $g_s = 9$ seconds. The queue would build up and clear on the South leg (link 1) during $r + g_s = 43$ seconds of the cycle time. Assuming uniform arrivals, gaps should occur during $g - g_s = 17$ seconds of the unsaturated part of phase B green for link 1 (which would have 3.1 vehicles per cycle during this period). The average number of right turners per cycle would be 3.4 and there is sufficient capacity for these vehicles to depart during 17 plus 3 seconds of amber time.

TABLE V

DATA FOR EXAMPLE 3

Link	Approach	Lane Arrangement	Sat. Flow tcu/h	Flow veh/h	Sat. Flow veh/h	y
1	South		3550	665	3333	0.200
2	West		6600	1700	5631	0.302
3*	North		1700	975	2304 ($E_{rt} = 3.0$)	0.423
5*	North		1810			
4	East		6190	1695	5359	0.316

* Changes for each alternative shown in Table VI

TABLE VI

DATA FOR THE NORTH APPROACH IN EXAMPLE 3 FOR VARIOUS ALTERNATIVES

Alternative	Lane arrangement		E_{rt} for Link 5	Flows veh/h		Sat. Flows veh/h		Flow ratios	
	Link 5	Link 3		q_5	q_3	s_5	s_3	y_5	y_3
1			3.0	340†	635	804	1500	0.423	0.423
1B			3.2	285†	690	686	1514	0.415	0.456
2			1.0	205	770	1687	1531	0.122	0.503
3			1.0	205	770	1687	3197	0.122	0.241
4			3.0	205	770	589	3197	0.348	0.241
4A			2.6	205	770	679	3197	0.302	0.241

* Unopposed right turn

† Shared right-turn and through lane

TABLE VII

SOLUTION AND RESULTS FOR EXAMPLE 3

Alternative	Flow Ratio, y and Effective Green Time, g (seconds) for Phase			Intersection Flow Ratio Y	Intersection Lost Time L	Percentage Spare Capacity	Minimum Sat. Flow, s (tcu/h)	Optimum Cycle Time, c_o (seconds)	Cycle Time Chosen, c (seconds)	Intersection Degree of Saturation, X	Average Delay per Vehicle (seconds)					Total Intersection Delay, $\sum d_i$ (veh·h/h)	Remarks
	A	B	C								d_1	d_2	d_3	d_4	d_5		
1	0.316	0.423															
	24	31		0.739	10	12%	3510	65.3	65	0.87	11.1	19.4	27.9*	20.7	42.5*	29.89	* $y_3 = y_5 = 0.423$ gives $d_5 > d_1$
1B	0.316	0.456		0.772	10	7%	1700	88.3	90	0.87	12.8	26.6	26.5†	28.0	26.7†	35.29	† $d_5 = d_3 = 27s$ gives $y_B = y_3 > y_5$
	33	47															
2	0.316	0.200	0.122	0.819	10*	1%	1700	111.2	110	0.90	32.0	33.9	33.2	36.2	43.0	48.5‡	‡ $y_{B+C} = y_3 = 0.503 > y_B + y_C$
	39	35	21														
3	0.316	0.200	0.122	0.638	15†	23%	1810	74.6	75	0.80	28.8	19.5	14.1	20.2	57.7	30.32	† $y_{B+C} = y_3 = 0.241 < y_B + y_C$
	30	19	11														
4	0.316	0.348		0.664	10	24%	1810	59.0	60	0.80	12.0	15.7	12.7	16.3	34.6	22.03	
	24	26															
4A	0.316	0.302		0.618	10	33%	1810	51.9	40**	0.82	9.8	12.2	10.7	13.4	34.7	18.12	** Minimum cycle time used
	15	15															

Note: If links 3 and 5 are modelled together in Alternative 1 for delay calculations, i.e. using $q = 975$, $s = 2304$, average delay (both links) is found 22.4 seconds and total intersection delay is found 27.03 vehicle hours per hour.

REFERENCES

- ALLSOP, R.E. (1976). SIGCAP: a computer program for assessing the traffic capacity of signal-controlled road junctions. *Traff. Eng. Control*, 17, pp. 338-41.
- AUSTRALIAN ROAD RESEARCH BOARD (1968). Australian road capacity guide — Provisional introduction and signalised intersections. Bull. No. 4. Melbourne. (Now revised and reprinted as ARRB Research Report, ARR No. 79.)
- COUNTRY ROADS BOARD (1974). Road design manual — For rural and urban roads other than freeways. Melbourne.
- DEPARTMENT OF ROADS AND TRAFFIC (1974). Improved operation of urban transportation systems, Vol. 1: Traffic signal control strategies — A state-of-the-Art. Toronto, Ontario.
- HIGHWAY RESEARCH BOARD (1965). Highway capacity manual. Special Report 87. Washington, D.C.
- LOGIE, D.M.W. (1977). Computer aided design and evaluation of traffic management schemes. *Traffic Eng. Control*, 18, pp. 347-53.
- MAY, A.D. and PRATT, D. (1968). A simulation study of load factor at signalised intersections. *Traff. Eng.*, 38, pp. 44-9.
- MILLER, A.J. (1963). Settings for fixed-cycle traffic signals. *Oper. Res. Q.* 14, pp. 373-86.
- (1968). The capacity of signalised intersections in Australia. ARRB Bull. No. 3. Melbourne.
- (1969). On the Australian Road Capacity Guide. *Highw. Res. Rec.* 289, pp. 1-13.
- ROBINSON, C.C. (1976). Highway capacity. In 'Transportation and Traffic Engineering Handbook' (Ed.) J.E. Baerwald. Institute of Traffic Engineers. (Prentice-Hall: Englewood Cliffs, New Jersey.), pp. 309-76.
- SUTARIA, T.C. and HAYNES, J.J. (1977). A study of level of service at signalised intersections. Paper presented to the Transp. Res. Board, 56th Annu. Meet. (Reported by M.G. Richards in *Traff. Eng. Control*, 18, pp. 208-10.
- WEBSTER, F.V. (1958). Traffic signal settings. Road Res. Tech. Paper No. 39 (HMSO: London.)
- and COBBE, B.M. (1966). Traffic signals. Road Res. Tech. Paper No. 56. (HMSO: London.)