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REPRINT

Traffic Signals: Capacity and Timing Analysis

R. AKÇELIK

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NOTE:

This report is related to the intersection analysis methodology used in the SIDRA INTERSECTION software. Since the publication of this report, many related aspects of the traffic model have been further developed in later versions of SIDRA INTERSECTION. Though some aspects of this report may be outdated, this reprint is provided as a record of important aspects of the SIDRA INTERSECTION software, and in order to promote software assessment and further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. This report was originally published by the Australian Road Research Board and a printed version may be available through the ARRB website.

Traffic Signals: Capacity and Timing Analysis

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Executive Summary

Traffic signal capacity and timing analysis techniques are presented as an aid for detailed geometric and operational design of signalised intersections. The background to these techniques is provided to permit the development of a better understanding of the subject.

The report introduces several changes to the existing methods, a basic change being from phase-related methods to movement-related methods. A critical movement identification method is presented as a procedure which automatically satisfies minimum green time constraints and allows the use of different degrees of saturation for different movements as a measure of priority and restraint. There are changes in the saturation flow estimation method, and there is an emphasis on the use of measured saturation flows in practice.

Detailed discussions of opposed turn, lane utilisation and short lane problems are presented in the section on saturation flows. New formulae are given for predicting delay, number of stops and queue length, which are applicable to both under-saturated and over-saturated conditions. Formulae for calculating a practical cycle time, an approximate optimum cycle time using a stop penalty parameter, and green times in both simple and complicated multiple overlap cases are also included.

The prediction of operating characteristics and calculation of signal timings for vehicle-actuated and co-ordinated signals are discussed separately in appendices. All other aspects of this report are applicable to both fixed-time and vehicle-actuated, isolated and co-ordinated signals. Signal design questions in general, and the question of design period in particular, are discussed and a recommended design procedure which brings together various methods described in the report is given (Section 8.3). Suggestions for simplified analysis which can be adopted for planning and preliminary design purposes are included. Numerical examples are presented to illustrate the methods described in the report.

Reference: Akçelik, R. (1981). TRAFFIC SIGNALS: CAPACITY AND TIMING ANALYSIS. ARRB Transport Research Ltd. Research Report No. 123. 108 pages. Sixth Reprint 1995.

Keywords: Traffic signal/junction/traffic lane/design (overall design)/vehicle actuated/ fixed time (signals)/linked signals/area traffic control/traffic flow/turn/capacity (road, footway)/congestion (traffic)/delay/queue/stops/deceleration/acceleration/bunching/ pedestrian/priority(traffic)/safety/fuel consumption/cost/program (computer)

An effectiveness audit of the intersection capacity research at the Australian Road Research Board has been carried out. Research Report No. 242 presents the findings of this evaluation, including comments on current and future research directions on capacity research.

Recent ARRB research in this area has concentrated on vehicle-actuated signals and paired intersections. Results of these research efforts are being published and will be incorporated into future versions of SIDRA.

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AKÇELIK, R. (1990). Calibrating SIDRA. Australian Road Research Board. Research Report ARR No. 180. (Second Edition, First Reprint 1993).

TAYLOR, M.A.P. BARTON, E.V., BLISS, J. and O'BRIEN, A.P. (1993). Effectiveness Audit of ARRB Intersection Capacity Research. Australian Road Research Board. Research Report ARR No. 242.

Notation and definitions

		Section
a	Start lag (sum of intergreen time, I, and start loss for a movement).	2
b	End lag (end gain for a movement).	2
c	Cycle time ($= \Sigma(G + I)$ for phases $= \Sigma(g + \ell)$ for critical movements).	2,4,7
c_m	Minimum cycle time — sum of minimum required movement times for critical movements ($= \Sigma\, t_m = \Sigma\,(G_m + I)$).	7
c_p	Practical cycle time — the cycle time which gives maximum acceptable degrees of saturation, x_p, for critical movements.	7
c_o	Optimum cycle time — the cycle time which gives the minimum value of a chosen performance measure, e.g. delay, delay and stops, queue length, etc. (eqn (7.1) gives an *approximate* optimum cycle time for critical movements at an isolated intersection).	7
c_{max}	Maximum acceptable cycle time (normally 120-150 s).	4,7,8
d	Average delay per vehicle (s)	6
D	Total delay per unit time ($= qd$).	6
e	Through car equivalent for a vehicle and turn type (through car units per vehicle).	5,F
e_o	Opposed turn equivalent.	5,F
f	A multiplying factor used in various meanings with different subscripts, e.g. f_c = traffic composition adjustment factor for saturation flow estimation.	5,6,F,G
F	Phase change time — the time during the signal cycle when there is a change of right of way, i.e. a movement is stopped and another started after an intergreen period (the displayed and effective green periods start at times $F + I$ and $F + a$, respectively).	2,4,7
g	Effective green time — total movement time less lost time ($= t - \ell$).	2,4,7
g_m	Minimum effective green time ($= G_m + I - \ell$).	4,7,B
g_s	Saturated portion of green period.	5,F
g_u	Unsaturated portion of green period ($= g - g_s$).	5,F
G	Displayed (controller) green time.	2,4,7
G_m	Minimum displayed green time.	4,7,B

h	Stop rate — average number of complete stops per vehicle.	6,G
H	Number of complete stops per unit time ($= qh$).	6,G
I	Intergreen time (yellow plus all red) — time from the end of the green period on one phase to the beginning of the green period on the next phase (intergreen time for a movement is the intergreen time of the starting phase).	2
j	Average queue space per vehicle (m).	5,6,F,G
ℓ	Movement lost time — the time which is effectively lost, i.e. start lag less end lag ($= a - b =$ Intergreen + start loss − end gain).	2,E
L	Intersection lost time — the sum of critical movement lost times.	2,4
N	Average stop-line queue at the start of the green period — the number of vehicles in the queue in an average cycle, measured as idealised arrivals at the stop line ($= qr + N_o$). The actual extent of the stationary queue at the start of green time is larger than this (= 1.10 N as a first approximation).	6
N_o	Average overflow queue — number of vehicles left in the queue when the signals change to red in an average cycle.	6
N_m	Maximum back of the queue — the maximum physical extent of the queue (in vehicles) in an *average* cycle, which is reached some time after the start of the green period ($= qr/(1-y) + N_o$).	6
N_c	Critical queue — the maximum queue length (in vehicles) which is exceeded only in a negligible number of cycles ($= 2N_m$, roughly, considering both undersaturated and oversaturated conditions).	6
O	Offset — the difference in starting times of the green periods at successive signals.	2,7,I
q	Flow rate (movement) — average number of arrivals per unit time (for oversaturated conditions, this is the demand rate).	3,4,6
qc	Average number of arrivals per cycle (vehicles).	3,6
Q	Capacity (movement) — maximum number of departures per unit time (sg/c).	3,6
r	Effective red time — cycle time less effective green time ($= c - g$).	6,F
ρ	Lane utilisation ratio — the ratio of the degree of saturation of a lane to the critical (largest) degree of saturation in any lane of the movement.	5
s	Saturation flow — maximum steady rate of departure from the queue during the green period (vehicles per unit time).	2,5,E,F
s_u	Opposed turn saturation flow (filter rate) which is a function of the opposing flow rate and the gap acceptance characteristics of the opposed turn. This is similar to the 'maximum entry flow' used in roundabout and unsignalised intersection capacity modelling.	5,F
sg	Capacity (maximum number of departures) per cycle (vehicles).	3,6

t	Movement time — combined effective green time and lost time ($= g + \ell = \Sigma\ (G + I)$ where the summation is for the phases during which the movement has right-of- way).	4
t_m	Minimum required movement time ($= G_m + I = g + \ell$)	4
T	Sum of various movement times (used for critical movement identification).	4
T_f	Flow period (in hours for use in eqn (6.1)) — the time interval during which an average arrival (demand) rate, q, persists (QT_f is the throughput, i.e. the maximum number of vehicles which can be discharged during T_f).	6
tcu	Through car unit — a through (straight-ahead) car.	5,F
u	Green time ratio (movement) — the proportion of the cycle which is effectively green ($= g/c$)	3,6
U	Intersection green time ratio — the ratio of available green time to cycle time; summation for the whole intersection of the critical movement green time ratios ($= (c - L)/c = \Sigma u$).	3,4,8
veh	Vehicle.	5,F
x	Degree of saturation (movement) — the ratio of flow to capacity ($= q/Q = qc/sg = y/u$).	3,4
X	Intersection degree of saturation — the largest (critical) movement degree of saturation.	3,4
x_p, X_p	Practical degree of saturation — maximum acceptable degree of saturation for an individual movement (x_p) or for the intersection (X_p).	3,4,8
y	Flow ratio (movement) — the ratio of flow to saturation flow ($= q/s$).	3,4
Y	Intersection flow ratio — summation for the whole intersection of the critical movement flow ratios (Σy).	3,4
Phase	A distinct part of the signal cycle in which one or more movements receive right of way is called a phase (U.K. 'stage'). A phase is identified by at least one movement which gains right of way at the start of it and at least one movement which loses right of way at the end of it.	2,A
Movement	Each separate queue leading to the intersection and characterised by its direction, lane allocation and right-of-way provision (phasing arrangement).	2,5
Overlap Movement	A movement which receives right-of-way during two or more consecutive phases.	2,4,7,A
Critical Movement	The movements which determine the capacity and timing requirements of the intersection.	2,4,7,A
Opposed Movement	A movement which has to give way to a higher priority (opposing) movement during the green period. This may be a right or left turn movement giving way to an opposing vehicle or pedestrian movement.	5,F

Note: All signal timing parameters (c, g, G, I, ℓ, L, etc.) are in units of seconds. When using various formulae given in this report, care must be taken to use consistent units for signal timing parameters and flow and saturation flow, e.g. flow rate, q, in eqn (6.3) for calculating delay must be in veh/s not in veh/h.

1. INTRODUCTION

This report presents techniques for the analysis of capacity and timing requirements of traffic at signalised intersections. Its purpose is to both provide methods that can be used by the traffic engineer for detailed geometric and operational design of signalised intersections and to also explain their background to help traffic engineering students develop a better understanding of the subject.

The methods given in this report follow the basic framework established in earlier publications which have influenced the Australian and U.K. signal design practices (Miller 1968*b* ; Webster and Cobbe 1966). Summaries and critical reviews of these and other traffic signal analysis methods developed to date can be found in reports by Akcelik (1979a), Allsop (1979), Pretty (1979) and Robertson (1979). The present report introduces several changes to the traditional techniques, a basic change being from 'phase-related' methods to 'movement-related' methods. An important aspect of this change is the use of 'movement lost time' concept instead of 'phase lost time' concept, which leads to a definition of the intersection lost time as 'the sum of critical movement lost times' rather than 'the sum of phase lost times'. This and other aspects of the new approach facilitates a clearer understanding of the relationships between the movement and signal phasing characteristics and an easier handling of complicated phasing systems associated with modern group or multi-phase signal controllers.

The basic concepts of movements and phases are introduced, and the definitions of the movement and intersection parameters such as saturation flow, effective green time, lost time, flow ratio and degree of saturation are given in Sections 2 and 3. A critical movement identification method is described in Section 4 in the form of a procedure which automatically satisfies minimum green time constraints and allows for the use of different degrees of saturation (levels of service) for different movements as a measure of priority and restraint.

Saturation flow is the most important single parameter in the capacity analysis of signalised intersections and is discussed in Section 5. New definitions of environment class, lane type and turn type are introduced for use in estimating saturation flows. It is recommended that saturation flows and arrival flow rates be expressed in vehicle units (rather than in through car units as in the past) for use in signal design calculations and that, wherever possible, measured saturation flows and lost times should be used rather than estimates derived from general averages based on historical data. Explicit and detailed treatments of opposed turn, lane utilisation and short lane problems are given since these have important implications in practice.

More detailed discussions and additional material for Sections 2 to 5 are given in Appendices A to F, including a discussion of fixed and variable phasing systems (Appendix A), a method for calculating minimum green times (Appendix B), numerical examples to illustrate the application of the critical movement identification method given in Section 4 (Appendix D), a method for measuring saturation flow and lost time in the field (Appendix E), and methods for dealing with complicated cases of two saturation flows per movement (due to opposed and unopposed turns) and signal timings when the saturation flow falls off during the green period (Appendix F). The concepts and methods given in these sections and appendices are essentially related to capacity, and as such they are applicable to both fixed-time and vehicle-actuated, isolated and co-ordinated signals.

In Section 6 on measures of performance, formulae are given for predicting delay, number of stops and queue length at traffic signals for vehicles as well as pedestrians. These formulae are applicable to both undersaturated (below capacity) and oversaturated (above capacity) conditions, unlike the formulae in earlier publications which are applicable to only undersaturated conditions (Webster 1966; Miller 1968*b*). Strictly speaking, the formulae given in Section 6 are applicable to isolated fixed-time signals. However, the overflow queue concept which is the basis of these formulae is applicable to both fixed-time and vehicle-actuated, isolated and co-ordinated signals. The vehicle-actuated and co-ordinated signal cases are discussed separately in Appendices H and I.

The methods to calculate cycle time and green times to achieve desirable operating conditions are described in Section 7, including a new approximate optimum cycle time formula which uses a stop penalty parameter (weighting of stops relative to delays). These methods must be used in conjunction with the critical movement identification technique described in Section 4 so as to ensure that correct parameters are used allowing for minimum green time constraints. Signal timing suggestions for vehicle-actuated and co-ordinated signals are given in Appendices H and I.

Signal design questions in general, and the question of design period in particular, are discussed, and a recommended design procedure which brings together various methods given in the report is described in Section 8. Suggestions for simplified analysis which can be adopted for preliminary design or planning purposes are included in Section 8. Numerical examples are given in Section 9 to illustrate the use of the methods described in the previous sections of the report.

It must be emphasised that the level of sophistication of the methods given in this report must not be interpreted as a suggestion that good engineering judgement can be replaced by a mechanical process. The approximate nature of various formulae, limitations in the models used to derive them, the average nature of some data (e.g. for saturation flow estimation) and limitations in field data collection methods must be kept in mind when interpreting the results obtained by applying the methods recommended in this report. Considering the multiplicity of often-conflicting traffic management/control objectives (safety, vehicle delays and stops, fuel consumption, pollutant emissions, pedestrian delays and stops, public transport priority, and so on), there is much room for engineering judgement when developing final design options and choosing the design to be implemented.

The methods described in this report are useful for the traffic engineer to make necessary design decisions on an analytical base. The question of the level of detail and accuracy of the method to be used in relation to the purpose of design is discussed in Section 8.

The efficiency of the usage of the methods given in this report can be improved by using a computer program. Among the computer programs used in Australia and overseas SIDRA (Akcelik 1979*b*), SIMSET (Sims 1979), SIGSET, SIGCAP (Allsop 1971 and 1976; Pretty 1980a) and CAPCAL (Hansson 1980) should be mentioned. The concepts and methods employed in these programs differ from those given in the present report. SIDRA program is currently being modified to implement the methods given in this report.

In order to make the equations, diagrams and tables easier to locate, they have been numbered according to the section they are in, e.g. eqn (4.1) is in Section 4, *Fig. F.3* is in Appendix F.

The reader may wish to refer to Section 8.3 first in order to gain an overall view about the steps involved in a complete traffic signal analysis exercise.

2. MOVEMENTS AND PHASES

Signal phasing is the basic control mechanism by which the operational efficiency and safety of a signalised intersection is determined. The current developments in signalisation technology provide increasingly flexible, but inevitably more complex, signal phasing options. It is therefore important to understand clearly how traffic movements and signal phases relate to each other. The underlying movement and phase concepts are described first before various movement parameters are defined below. A detailed discussion on phasing systems is given in Appendix A.

2.1 MOVEMENTS AND PHASES

Each separate queue leading to the intersection and characterised by its direction, lane usage and right of way provision is called a *movement*. The allocation of rights of way to individual movements is determined by the signal phasing system. An example is given in *Fig. 2.1* which illustrates a simple *phasing diagram* for a T junction. Four movements numbered 1 to 4 are shown in *Fig. 2.1a*. The *intersection plan* which shows lane arrangements is given in *Fig. 2.1b*. The movements from each approach road are described according to their unique right of way (phasing arrangement) and lane allocation/usage characteristics. General rules for movement description in relation to lane allocation/usage are given in Section 5.1.

Signal *phase* is a state of the signals during which one or more movements receive right of way. Signal phases will be defined in such a way that when there is a change of right of way, that is when a movement is stopped and another started, there is a *phase change*. A phase is identified by at least one movement gaining right of way at the start of it and at least one movement losing right of way at the end of it. In the past, the term 'phase' has been used in different ways. The definition used here is the same as the term 'stage' used elsewhere (Robertson 1969; Allsop 1971, 1972 and 1976; Vincent, Mitchell and Robertson 1980; Allsop and Murchland 1978).

A movement which receives right of way during more than one phase is called an *overlap movement*. In *Fig. 2.1*, Movement 1 is an overlap movement, and

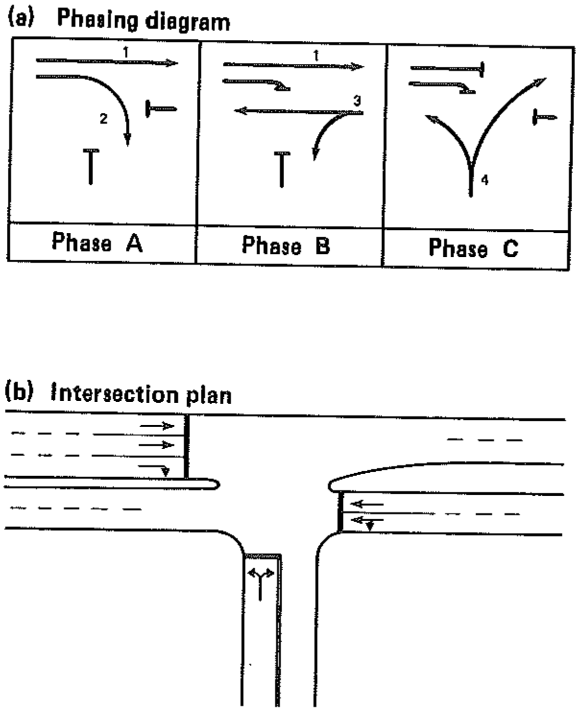


Fig. 2.1 — Signal phasing diagram and intersection plan (an example)

all others are non-overlap movements. More complex phasing systems will have more than one overlap movement, and the definitions given in this section will help solve such complicated phasing problems.

A phasing system can be described by a *phase-movement matrix*, in which the numbers of phases (labelled by letters) which start and stop each movement, i.e. *starting and terminating phases*, are specified as shown in *Table 2.1* for the phasing system illustrated in *Fig. 2.1*.

TABLE 2.1

PHASE-MOVEMENT MATRIX

Movement	*Starting Phase*	*Terminating Phase*
1	A	C
2	A	B
3	B	C
4	C	A

2.2 SIGNAL CYCLE

One complete *sequence* of signal phases is called a *signal cycle*. An example of a *cycle diagram* is given in *Fig. 2.2* which corresponds to the phasing system described in *Fig. 2.1*. The time from the end of the green period on one phase to the beginning of the green period on the next phase is called the *intergreen time* (I). It consists of yellow and all-red periods as shown in *Figs 2.2* to *2.4*. During the all-red period, both the terminating and the starting phases/movements are shown red signal simultaneously. From *Figs 2.2* to *2.4*, it is seen that the *phase change times* (F) are defined as phase termination times which occur at the end of the green period, and the intergreen period is the initial part of the phase. Therefore, the green period starts at time

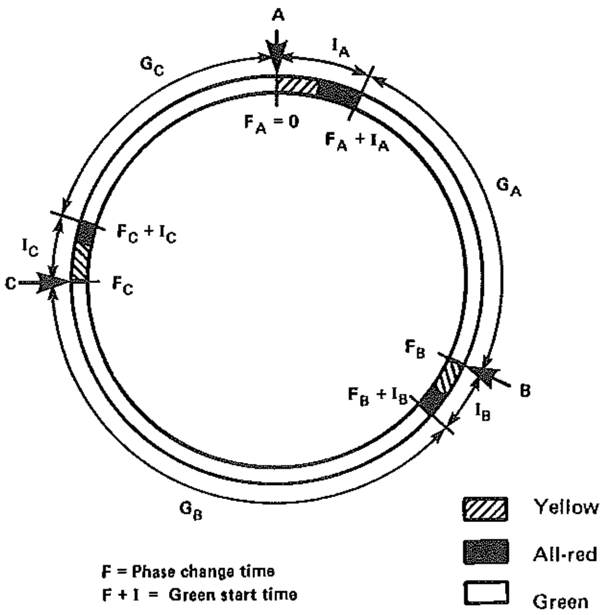


Fig. 2.2 — Signal cycle diagram (for the example in Fig. 2.1)

$(F + I)$. If the *displayed green time* for a phase is G, then the green period ends at time $(F + I + G)$. This is the phase change time for the next phase. The cycle diagram can be constructed by setting the first phase change time to zero and adding the $(I + G)$ value of the first phase to find the second phase change time, and so on. The sum of all phase intergreen and green times is the *cycle time*:

$$c = \Sigma(I + G) \qquad (2.1)$$

This is the time at which the last phase green period ends; hence it is the first phase change time in the next signal cycle. For example, in *Fig. 2.2*, $F_C + I_C + G_C = c = F_A$.

2.3 MOVEMENT CHARACTERISTICS

The basic movement characteristics are illustrated in *Fig. 2.3* in relation to a corresponding signal phasing arrangement. The basic model used is essentially a traditional one (Clayton 1940-41; Wardrop 1952; Webster 1958; Webster and Cobbe 1966; Miller 1968*a* and *b*; Allsop and Murchland 1978). However, some definitions which follow are new. An important new aspect of *Fig. 2.3* is that the two phase change times (F_i, F_k) to start and stop the movement under consideration do not need to correspond to consecutive phases because it may be an overlap movement. The basic movement characteristics, namely saturation flow, effective green time and lost time are described in the following sections.

2.3.1 Saturation Flow and Effective Green Time

The basic model assumes that when the signal changes to green, the flow across the stop line increases rapidly to a rate called the *saturation flow*, *s*, which remains constant until either the queue is exhausted or the green period ends. The departure rate is lower during the first few seconds while vehicles accelerate to normal running speed. Similarly, the departure rate is lower during the period after the end of green because some vehicles stop and others do not. However, for opposed turning movements, the departure rate during intergreen may actually be higher but the basic model is still valid. It should be noted that the saturation flow is the maximum departure rate which can be achieved when there is a

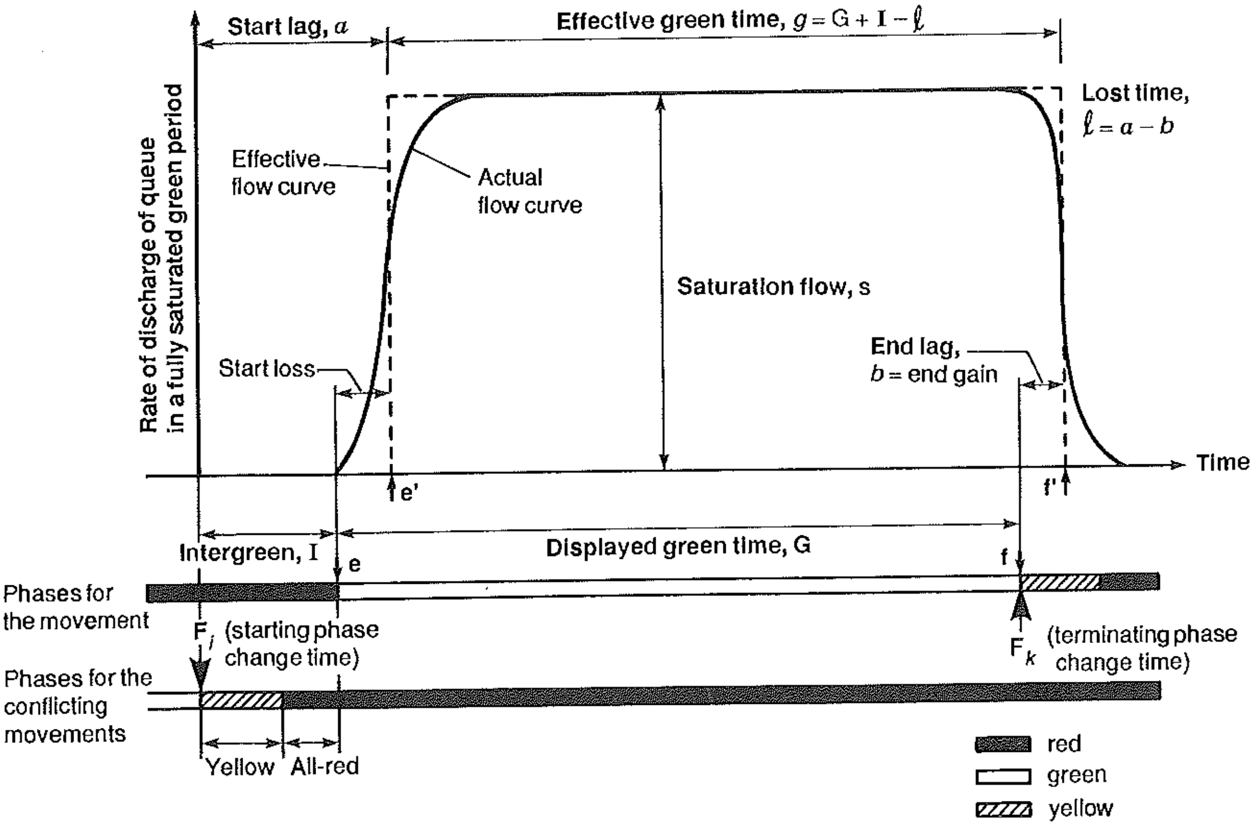


Fig. 2.3 — Basic model and definitions

queue. *Fig. 2.3* illustrates a *fully-saturated* green period, i.e. a case when the queue persists until the end of the green period.

As indicated by the dotted line in *Fig. 2.3*, the basic model replaces the actual departure flow curve by a rectangle of equal area, height of which is equal to the saturation flow (s) and whose width is the *effective green time* (g). In other words, the area under each curve is (sg) which is the maximum number of departures in an average cycle. The time between the start of displayed green and the start of effective green periods (ee') is considered to be a *start loss*. Similarly, the time between the end of displayed green and the end of effective green periods (ff') is considered to be an *end gain*. Therefore, the effective green time is equal to the displayed green time plus end gain less start loss ($g = G + ff' - ee'$).

2.3.2 Start and End Lag Times

The start and end times of the effective green period for a movement are best defined with reference to phase change times. This is of particular importance for the design of a co-ordinated signal system where *offsets* are defined in relation to phase change times of successive signals (see Appendix I). For this purpose, a *start lag* (a) is defined as the sum of the movement intergreen time and start loss, and an *end lag* (b) is defined simply as the end gain, i.e. $a = I + ee'$ and $b = ff'$ as shown in *Fig. 2.3*. It should be noted that the movement intergreen time is the intergreen time of the starting phase of the movement.

The start and end of an effective green period are given by ($F_i + a$) and ($F_k + b$), where F_i and F_k are the two phase change times which start and stop the movement, respectively. Thus, by defining the start lag to include the intergreen time, the same set of reference points in the cycle (i.e. phase change times) are used to find the start and end times of both displayed and effective green periods. This is seen in *Fig. 2.4*, which is the *timing diagram* for the phasing system shown in *Fig. 2.1* (numerical data are given in Appendix C). The top line in *Fig. 2.4* which shows the phase change times, intergreen times and phase green times (F, I, G) represents the operation of the signal controller. It is a linear representation of the cycle diagram given in *Fig. 2.2*. Each of the other four lines corresponds to one movement with the effective green, start lag and end lag times (g, a, b) shown. For example, for Movement 1 whose starting and terminating phases are A and C, respectively, the intergreen time is $I_1 = I_A = 6$ s, the start lag is $a_1 = 8$ s (i.e. the start loss is 2 s), the end lag (end gain) is $b_1 = 2$ s, and hence, the effective green period starts at time $F_A + a_1 = 0 + 8 = 8$ s and ends at time $F_C + b_1 = 75 + 2 = 77$ s.

2.3.3 Movement Lost Time

The *movement lost time* (ℓ) is defined as the difference between the start and end lag times:

$$\ell = a - b \tag{2.2}$$

Therefore, the movement lost time is equal to the intergreen time (I) plus start loss less end gain ($\ell = I + ee' - ff'$, see *Fig. 2.3*). If it is assumed that the start loss is equal to the end gain, the movement lost time becomes equal to the intergreen time. Because the intergreen time is included in the lost time definition, the intersection capacity and timing problem can now be presented in terms of movement characteristics alone. Typical values of the intergreen and lost times are mentioned in Section 2.6.

The movement lost time definition given in eqn (2.2) means that the relationship between the displayed green time, G, and the effective green time, g (see *Fig. 2.3*) is

$$g + \ell = G + I \tag{2.3}$$

Suppose the movement under consideration gains right of way at phase change time F_i and loses right of way at phase change time F_k. That is, the starting and terminating phase numbers are i and k respectively; hence the movement has right of way during phases i to $k - 1$ (for a non-overlap movement i to $i + 1$). Then the intergreen time I in eqn (2.3) is the starting phase intergreen time I_i which is included in the lost time ℓ.

The displayed green time, and hence effective green time in eqn (2.3), includes the intermediate phase intergreen times as well as the green times of

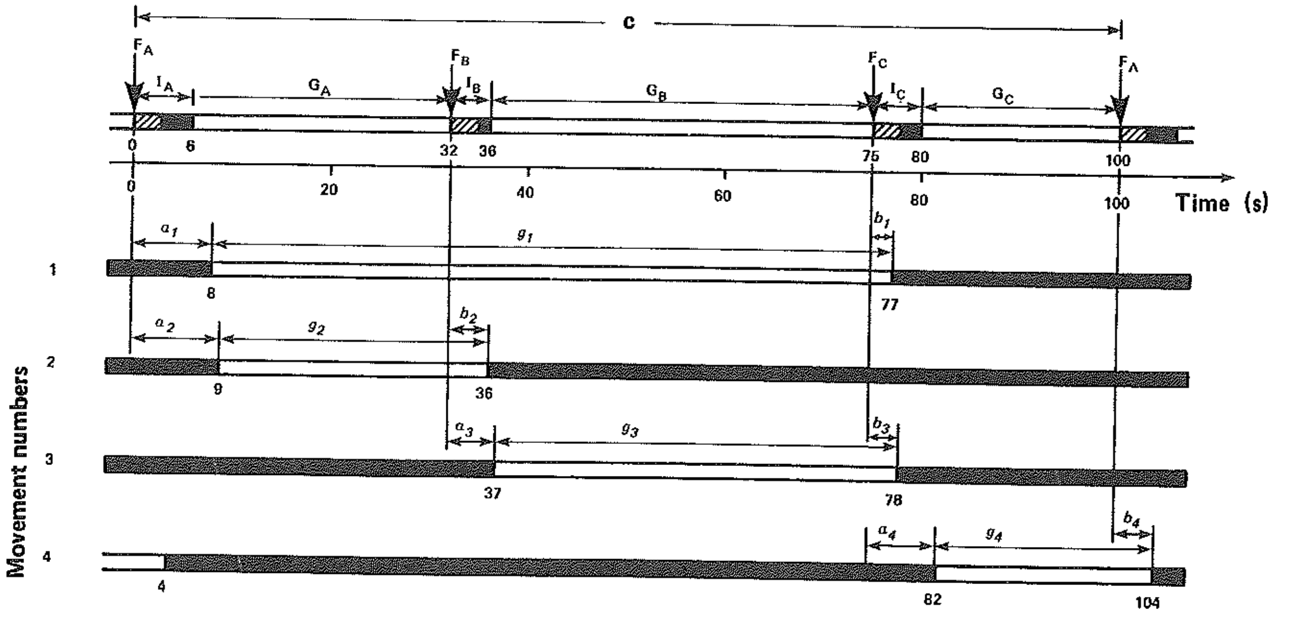


Fig. 2.4 — Signal timing diagram (for the example in Fig. 2.1)

phases during which the movement has right of way. For movement j:

$$G_j = \sum_{i}^{k-1} G + \sum_{i+1}^{k-1} I \qquad (2.4)$$

For example, the overlap movement 1 in *Fig. 2.1* will have $G_1 = G_A + G_B + I_B = 26 + 39 + 4 = 69$ s, whereas for the non-overlap movement 3, simply $G_3 = G_B = 39$ s (see *Fig. 2.4*).

Because the difference between the starting and terminating phase change times is equal to the sum of the displayed green and intergreen times, i.e. $F_k - F_i = G + I$ as seen in *Fig. 2.3*, it is useful to note that

$$g = F_k - F_i - \ell \qquad (2.5)$$

Using eqn (2.5), the effective green time of a movement can be calculated directly from known phase change times. For example, in *Fig. 2.4*, the lost time for Movement 1 is $\ell_1 = a_1 - b_1 = 8 - 2 = 6$ s, and the starting and terminating phase change times are $F_A = 0$, $F_C = 75$; therefore the effective green time is $g_1 = 75 - 0 - 6 = 69$ s, or from eqn (2.3), $g_1 = G_1 + I_1 - \ell_1 = 69 + 6 - 6 = 69$ s.

2.4 CRITICAL MOVEMENTS

Eqn (2.1) expresses the cycle time in terms of the phase green and intergreen times. A similar relationship holds for movement parameters, namely

$$c = \Sigma\,(g + \ell) \qquad (2.6)$$

where the summation is for *critical movements*. The movements which determine the capacity and timing requirements of the intersection are called critical movements. If sufficient time is allocated to each critical movement to meet its capacity requirement, then *all* movements will have sufficient capacity. If all movements were non-overlap movements, there would be one critical movement per phase. This would be the movement which requires the longest time $(g + \ell)$ in the phase. The corresponding phase green time G is determined by eqn (2.3). When there are overlap movements, the phase green times are not easily found as indicated by eqn (2.4). However, it is possible to determine the required cycle time without reference to phase times. As implied by eqn (2.6), the cycle time can be determined if the critical movements are identified.

A new method is described in Section 4 for critical movement identification. It is useful to mention at this stage that, the critical movements for the phasing system shown in *Fig. 2.1* as an example are either (1 and 4) or (2, 3 and 4). This can be seen from the *critical movement search diagram* given in *Fig. 2.5*. In this diagram, nodes correspond to phase changes and links to movements. The diagram in *Fig. 2.5* is constructed from the phase-movement matrix given in *Table 2.1*. The required movement times $(g + \ell)$ can be defined in relation to capacity, and the critical movement identification is a matter of finding the 'longest path' (from A to A in *Fig. 2.5*).

2.5 INTERSECTION LOST TIME

Eqn (2.6) can be written as $c = \Sigma g + \Sigma \ell$, and an *intersection lost time* can be defined as

$$L = \Sigma \ell \qquad (2.7)$$

where the summation is for *critical movements*. This definition should be understood clearly before the signalised intersection capacity and timing methods can be used. In contrast with traditional methods, the intersection lost time is defined as the sum of 'critical movement lost times' rather than the sum of 'phase lost times'. As such, L will exclude the intermediate phase intergreen times corresponding to the critical overlap movements. For example, in *Figs 2.1* and *2.5*, if movements 1 and 4 are critical, $L = \ell_1 + \ell_4$ (excludes I_B); otherwise $L = \ell_2 + \ell_3 + \ell_4$ (includes all intergreen times).

The intersection lost time definition given by eqn (2.7) allows for any combination of overlap movements. This and other intersection parameters described in Section 3 require the identification of critical movements before they can be calculated. A general method is described in Section 4, which also allows for the effect of minimum green times.

2.6 DISCUSSION

The movement lost time concept described in this section overcomes various difficulties associated with the 'phase lost time' concept used in earlier publications (Webster and Cobbe 1966; Miller 1968*a* and *b*; Allsop 1972). With the phase lost time concept, it is necessary to use 'extra effective green times' (Allsop 1971, 1972 and 1976) to handle: (a) lost time differences between movements due to different start loss and end gain values; and (b) the overlap movement cases where the intermediate phase intergreen times are green for the movement under consideration. The phase lost time definition may itself be confusing because it is based on the use of the end gain value of one movement and the start loss value of another movement. The movement lost time concept facilitates a clearer understanding of the relationships between the movement and signal phasing characteristics. In addition, the start and end lag definitions associated with this concept allow for the description of both displayed (controller) and effective green periods (start and end times, duration) in relation to phase change times.

In practice, the intergreen times can be specified for movements (in group controllers) or for phases (in phase controllers) and the definitions given here are applicable in both cases. However, for the sake of

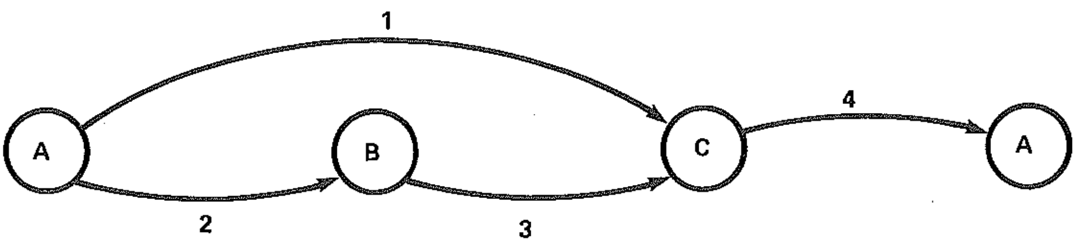


Fig. 2.5 — Critical movement search diagram (for the example in Fig. 2.1)

simplicity in formulating the capacity and timing methods, it will be assumed that all movements which start during a phase have the same intergreen. Any extra intergreen times can be counted as start losses, but by using the start lag concept this is automatically accounted for.

The intergreen time is determined to allow vehicles in the terminating phase to clear the intersection. It therefore depends on the width of the intersection and the vehicle speeds, and would normally be in the range 4 to 8 s. This subject is discussed in detail and a method to determine intergreen times is recommended by Hulscher (1980).

The difference between the movement intergreen time and lost time ($I - \ell$ = end gain less start loss) vary from site to site considerably. As a general-purpose average value, Webster and Cobbe (1966) recommend 1 s, and Miller (1968*a* and *b*) recommends 0.5 s. For simplistic analysis, $\ell = I$ may be used, i.e. end gain = start loss may be assumed. However, the saturation flows and lost times should be measured for each movement at the particular intersection, wherever possible. A simple method is described in Appendix E for this purpose.

It should be noted that the basic assumption that the saturation flow is *constant* throughout the green period may not be valid in certain circumstances where, in fact, saturation flow is dependent on signal timings. For example, saturation flow may fall off during the green period in the case of a short lane (turn slot, parking on the approach road, etc.). Opposed turning movement saturation flows and lost times are also dependent on signal timings. The capacity and timing calculations must be iterative or special solutions using fixed green times must be sought in these cases. The methods to be used when the saturation flow and lost time are dependent on signal timings are described in Section 5 and Appendix F.

3. CAPACITY AND DEGREE OF SATURATION

3.1 MOVEMENT CONSIDERATIONS

The *capacity* of a movement at traffic signals depends on the maximum sustainable rate at which vehicles can depart, i.e. the saturation flow, s, and the proportion of the cycle time which is effectively green for that movement, g/c, and is given by the formula:

$$Q = s\,(g/c) \qquad (3.1)$$

where the capacity, Q, is in the same units as the saturation flow. An alternative explanation is that sg is the capacity per cycle as expressed above in the discussion of the basic model (*Fig. 2.3*); therefore the capacity per unit time, Q is given by sg/c.

The proportion of effective green time to cycle time is called the *green time ratio* for the movement:

$$u = g/c \qquad (3.2)$$

Another useful movement parameter is the ratio of arrival flow, q, to saturation flow. This is called the *flow ratio*:

$$y = q/s \qquad (3.3)$$

The movement *degree of saturation* is the ratio of arrival flow to capacity. From eqns (3.1) to (3.3), it is given by

$$x = q/Q = qc/sg = y/u \qquad (3.4)$$

The flow ratio can be considered to be a constant parameter which represents the 'demand', and the green time ratio can be considered to be the control parameter which represents the 'supply'. The degree of saturation is the ratio which relates these two parameters.

To provide adequate movement capacity,

$$Q > q \quad \text{or} \quad x < 1 \qquad (3.5)$$

It follows that

$$sg > qc \quad \text{or} \quad u > y \qquad (3.6)$$

The movement capacity can be increased (or the degree of saturation decreased) by increasing the green time ratio allocated to that movement. However, this increase in capacity is at the expense of the movements which do not have right of way during the same phase. This leads to the intersection capacity and degree of saturation concepts.

3.2 INTERSECTION CONSIDERATIONS

The condition for the capacity to be adequate to meet the demand of all movements, i.e. that the inequality (3.5) or (3.6) is satisfied for all movements, is the *intersection capacity* condition. By summing the inequalities (3.6) for the *critical movements*, the following condition must hold for the intersection

$$\Sigma u > \Sigma y \qquad (3.7)$$

These parameters will be defined as the *intersection green time ratio*

$$U = \Sigma u \qquad (3.8)$$

and the *intersection flow ratio*

$$Y = \Sigma y \qquad (3.9)$$

It should be emphasised that, as in eqn (2.7) for the intersection lost time L, U and Y are the sums of the respective *critical movement* parameters. The intersection green time ratio is equal to the ratio of the total available green time to cycle time

$$U = (c - L)/c \qquad (3.10)$$

where $c - L = \Sigma g$ is the sum of critical movement green times.

The *intersection degree of saturation, X*, is defined as the largest (movement) degree of saturation. The minimum intersection degree of saturation for a given cycle time is obtained when the critical movement green times are proportional to the corresponding flow ratios (Ohno and Mine 1973). This is an equal-degree-of-saturation method (Webster 1958; Akcelik 1978*a* and *b*), that is

$$x_1 = x_2 = \ldots = x_c = X \qquad (3.11)$$

where x_1, x_2, ..., x_c are the critical movement degrees of saturation. The intersection degree of saturation as defined by eqn (3.11) can be calculated without need to calculate green times. It is given by:

$$X = Yc/(c - L) \qquad (3.12)$$

The critical movement identification method given in Section 4 determines the movements whose green times must be set equal to specified minimum green time values. If such a movement is identified as critical, its minimum green time value is added to the intersection lost time, L, and its y and u values are not included in the corresponding intersection parameters Y and U. The intersection degree of saturation obtained from eqn (3.12) using the adjusted L, Y, U values is therefore applicable to all critical movements except those whose green times are determined by specified minimum values.

Because X is defined as the largest (movement) degree of saturation, the condition for the intersection that

$$X < 1 \tag{3.13}$$

satisfies the condition $x < 1$ for all movements. This is valid for both the equal- and unequal-degree-of-saturation methods. The operating conditions which satisfy the inequality (3.13) are usually referred to as *undersaturated* conditions. Otherwise *oversaturated* conditions will prevail (see Section 6).

3.3 PRACTICAL DEGREES OF SATURATION

In practice, there is an acceptable maximum degree of saturation which must be less than 1.0 because traffic conditions become unstable as arrival flows approach capacity resulting in excessive delays, stops and queue lengths. This is called the *practical degree of saturation*, and is denoted by x_p for a movement and X_p for an intersection. A study of various operating characteristics such as delay, number of stops and queue length with respect to increasing degrees of saturation indicates that practical degrees of saturation in the range from 0.8 to 0.9 represent satisfactory operating conditions. A consideration of what combinations of intersection parameters (Y, L, c) result in various values of X indicates that this is a feasible range (Akcelik 1978*a* and *b*). A value of 0.9 is implied in Webster and Cobbe (1966). Although a value of 0.95 represents undesirable operating conditions (long delays, unstable queues, etc.), this can be used as an absolute practical limit to represent undersaturated operating conditions.

The methods described in the subsequent sections of this guide for critical movement identification and for practical signal timing and spare capacity calculations make use of the practical degree of saturation concept. The use of x_p (or X_p) = 0.9 is recommended as a general-purpose value (0.95 as an absolute limit to be used under heavy demand conditions). In choosing practical degrees of saturation, variation in arrival flow rates during the peak period should also be considered. For example, a value of 0.9 for the peak hour period will have some room for coping with higher degrees of saturation resulting from peak flows within that period, i.e. peak half hour, peak 15 minute, etc., without a system breakdown. Therefore, a value less than 0.9 may be chosen depending on the particular peaking characteristics during the design/control period.

Different practical degrees of saturation can be chosen for different movements (Allsop 1972 and 1976). This leads to an unequal-degree-of-saturation method for finding signal timings and can be used as a measure of priority and restraint (Akcelik 1979*a*).

4. CRITICAL MOVEMENT IDENTIFICATION

Various intersection parameters described in previous sections are defined in terms of critical movements. The intersection lost time, green time ratio and flow ratio (L, U, Y) are defined as sums of the corresponding critical movement parameters (ℓ, u, y) in eqns (2.7), (3.8) and (3.9). The intersection degree of saturation, X, is defined as the largest (movement) degree of saturation. For the equal-degree-of-saturation method (eqn (3.11)), X is equal to the critical movement degrees of saturation, x. In other words, the critical movements have the highest degrees of saturation. As such, they determine the capacity and timing requirements of the intersection. The critical movement identification is therefore the first step in carrying out capacity and timing calculations. This was discussed briefly in Section 2.4. A detailed discussion follows a description of the movement time concept which is fundamental to the application of the critical movement identification method.

4.1 MOVEMENT TIME CONCEPT

If sufficient time is allocated to each *critical* movement to meet its capacity requirement then *all* movements will have sufficient capacity. The time allocated to a movement is the sum of effective green time, g, and lost time, ℓ, and is given by:

$$t = g + \ell = I + G = \sum_{i}^{k-1} (I + G) \tag{4.1}$$

where the summation is for the phases during which the movement has right of way (starting phase i, terminating phase k), G is the displayed green time for the movement (see eqn (2.4)), and I is the movement intergreen time which is equal to the intergreen time of the starting phase, I_i. The parameter t will be called the *movement time* which is the parameter to be used for critical movement identification.

The *required* movement times can be calculated from

$$t = u\,c + \ell \tag{4.2}$$

where ℓ is the movement lost time (eqn (2.2)), c is the cycle time, and u is the required green time ratio. As a first estimate of the cycle time, c = 100 can be used in eqn (4.2), i.e. $t = 100\,u + \ell$. In some marginal cases, the value of cycle time may affect which movements are critical; the procedure described here allows for such cases.

The required green time ratio is calculated to achieve maximum acceptable (practical) degrees of saturation, x_p, and is given by

$$u = y / x_p \tag{4.3}$$

where y is the movement flow ratio (eqn (3.3)). As discussed in Section 3.3, different x_p values can be used for different movements. The use of x_p = 0.9 is recommended as a general-purpose value.

The movement time calculated from eqns (4.2) and (4.3) must satisfy the constraint that

$$t \geq t_m \tag{4.4}$$

where t_m is the minimum required movement time given by

$$t_m = G_m + I = g_m + \ell \qquad (4.5)$$

where G_m is minimum displayed (controller) green time, I is the intergreen time, g_m is the minimum effective green time and ℓ is the lost time for the movement under consideration. The value of G_m is determined by vehicle or pedestrian requirements. For pedestrian movements, G_m includes the walk and clearance periods, and may be rather large depending on the kerb-to-kerb crossing distance (see Appendix B).

If the minimum time constraint given by the inequality (4.4) is not satisfied, then the required movement time must be set equal to

$$t = t_m = G_m + I \qquad (4.6)$$

If such a movement is identified as critical, the value of t_m will be added to the intersection lost time (i.e. g_m will be treated as if it is a lost time for other critical movements), and the flow and green time ratios (y, u) for this movement will not be included in the calculation of the respective intersection parameters (Y, U).

4.2 CRITICAL MOVEMENT IDENTIFICATION

The critical movement identification method is based on the comparison of the required movement time, t, values. It is straightforward when all movements are non-overlap, i.e. receive right of way during one phase only, but more complicated when overlap movements exist. The two cases are discussed separately first, and then the general method is described fully.

4.2.1. Non-Overlap Movements

The critical movement in the phase is the one which has the largest required movement time, t. If the lost times are the same for all movements in the phase, also assuming an equal-degree-of-saturation solution, the critical movement can be identified as the one with the largest flow ratio, y. This is the method given by Webster and Cobbe (1966) and Miller (1968*b*). However, the movement lost times can be significantly different, in particular when the opposed turning movement lost times are treated explicitly. If the saturation flows as well as the lost times of all movements in the phase were the same, the critical movement could be identified as the one which has the highest arrival flow rate. The critical lane technique used in the U.S.A. is based on this approach (Institute of Transportation Engineers 1976; Messer and Fambro 1977). The lane with the highest flow rate is chosen to represent each movement. The flow rates are converted to through car units (i.e. 'normalised') using a common base saturation flow value for all movements (lanes). This method has the same restriction as the y-value method in that all movements in the phase must have the same lost time. Furthermore, additional calculations are necessary for calculating total movement (rather than lane) arrival and saturation flow rates expressed in vehicles per hour (rather than through cars per hour) in order to determine movement operating characteristics (delay, number of stops, etc.).

4.2.2 Overlap Movements

Eqn (4.1) shows that the movement time includes the green and intergreen times of all phases during which it has right of way. For an overlap movement, the t value must be compared with the sum of the t values of the non-overlap movements which represent the corresponding phases. The overlap movement is critical if its t value is larger. Otherwise, the non-overlap movements representing the relevant phases are the critical movements. This will be understood better after the general method is described below.

It is more difficult to identify the critical movements when there are more than one overlap movements with common phases (multiple overlap case). In such cases, some phases may not have a non-overlap movement representing the phase. A general method is described in Section 4.3 to cope with multiple overlap phase cases. It is recommended that the same method is used in all cases since various parameters calculated for critical movement identification will also be used in signal timing calculations. However, some steps of this method can be neglected according to the simplicity of the particular case.

4.2.3 Example

The method is described in Section 4.3 with the aid of a numerical example to facilitate easier understanding. The phasing system and the intersection plan are given in *Fig. 4.1*. This is a modification of the example given in Section 2 (*Fig. 2.1*) so as to represent more realistic conditions at a T junction. The left- and right-turning movements from the side road are described separately (movements 4 and 5) resulting in a multiple overlap case because the overlap movements 1 and 4 also overlap with each other. The pedestrian movements 6 and 7 are included.

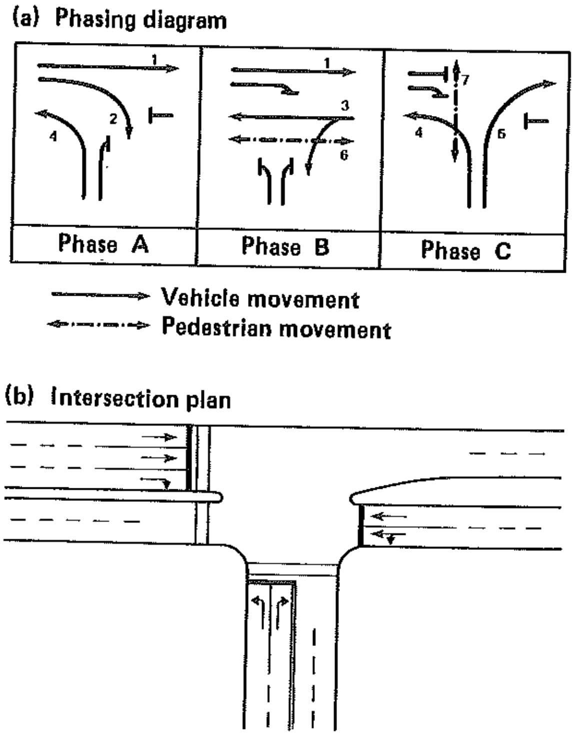


Fig. 4.1 — Signal phasing diagram and intersection plan (an example)

4.2.4 Data

The following data must be specified for each movement before the critical movement identification procedure, and hence the capacity and timing calculations, can be commenced:

(a) starting and terminating phase numbers (or letters for convenience);

TABLE 4.1

CRITICAL MOVEMENT SEARCH TABLE (FOR THE EXAMPLE IN FIG. 4.1)

(a) DATA

Movement	Starting Phase	Termin. Phase	Intergreen Time (I)	Min.Disp. Green (G_m)	Arrival Flow (q)	Satur. Flow (s)	Lost Time (ℓ)	Min.Eff. Green (g_m)	Prac. Deg.Sat. (x_p)
1	*A*	*C*	*6*	*8*	*650*	*3480*	*6*	*8*	*0.90*
2	*A*	*B*	*6*	*6*	*240*	*1510*	*5*	*7*	*0.92*
3	*B*	*C*	*5*	*8*	*920*	*3260*	*4*	*9*	*0.85*
4	*C*	*B*	*5*	*8*	*580*	*1240*	*8*	*5*	*0.90*
5	*C*	*A*	*5*	*6*	*170*	*1490*	*3*	*8*	*0.92*
6	*B*	*C*	*5*	*14*	*Pedestrians*		*19*	*15*	–
7	*C*	*A*	*5*	*17*	*Pedestrians*		*22*	*18*	–

(b) CALCULATIONS

Movement	y = q/s	u = y/x_p	100 u+ℓ	t_m = G_m + I	t	Check for c = *90*			
						uc + ℓ	t'	g	x = (c/g) y
1	*0.19*	*0.21*	*27*	*14*	*27*	*25*	*25*	*62*	*0.28*
2	*0.16*	*0.17*	*22*	*12*	*22*	*20*	*20*	*29*	*0.50*
3 *	*0.28*	*0.33*	*37*	*13*	*37*	*34*	*34*	*30*	*0.84*
4 *	*0.47*	*0.52*	*60*	*13*	*60*	*55*	*55*	*48*	*0.88*
5	*0.11*	*0.12*	*15*	*11*	*15*	*14*	*14*	*19*	*0.52*
6	–	–	–	*19*	*19*	–	*19*	*30*	–
7	–	–	–	*22*	*22*	–	*22*	*18*	–

* Critical movements

(b) intergreen time, I (seconds);

(c) minimum displayed green time, G_m (seconds);

(d) arrival flow rate, q (veh/h);

(e) saturation flow rate, s (veh/h);

(f) lost time, ℓ (seconds); and

(g) practical (maximum acceptable) degree of saturation, x_p.

The data for the phasing system shown in *Fig. 4.1* are given in *Table 4.1a*. The first three columns of this table is the phase-movement matrix for this example. Additional data are given in Appendix C. Minimum effective green time is calculated as $g_m = G_m + I - \ell$ from eqn (4.5).

4.2.5 Critical Movement Search Table and Diagram

It is recommended that the method is implemented with the aid of a *critical movement search table* as shown in *Table 4.1* with the data and calculations for the present example. A blank table is given in Appendix J.

A second aid is the *critical movement search diagram* which has already been mentioned in Section 2. The search diagram in *Fig. 4.2* is for the phasing system shown in *Fig. 4.1* and the phase-movement matrix data given in *Table 4.1a*. In this diagram, the nodes correspond to phase change events, and the links to movements.

The critical movement search diagram is in fact a cyclic critical path diagram (Zuzarte Tully and Murchland 1977). The critical movement search is therefore a matter of identifying all paths from phase A to A plus any additional paths, e.g. one path from B to B in *Fig. 4.2*, calculating the total movement time, $T = \Sigma t$, for each path, and finding the path which gives the largest T value. The movements which form this path are the critical movements. The process involves the elimination of some non-overlap movements at an early stage. The method described here is based on a movement oriented approach and is helpful for the understanding of how a particular phasing system operates. It is described in the next section as a step-by-step procedure which has been set up in such a way that various constraints involved in the process are satisfied automatically.

4.3 THE PROCEDURE

The basic steps of the procedure involve the critical movement identification (Steps 1 to 4) as well as the capacity and timing calculations (Steps 5 to 10). Steps 8 and 10 provide various checks using the calculated signal timings. An overall signal design procedure is described in Section 8.3.

1. Determine the required movement times, t (use the critical movement search table).
2. Prepare a critical movement search diagram.
3. Non-overlap movements: determine representative movements and reduce the critical movement search diagram.
4. Overlap movements: calculate the total required movement time, T, for each possible movement combination (path). The critical movements are those which give the largest T value.
5. Calculate the intersection lost time, flow ratio and green time ratio (L, Y, U) as the sum of the corresponding critical movement parameters (ℓ, y, u).
6. Calculate the practical and approximate optimum cycle times (c_p, c_o).
7. Choose a cycle time between c_p and c_o (also to satisfy $c \leq c_{max}$, where c_{max} is a specified maximum cycle time).
8. For the chosen cycle time check if the critical movements are valid.
9. Calculate green times.
10. Calculate movement degrees of saturation, x and check if $x \leq x_p$ for all movements.

Each step of the procedure is now described in detail with reference to the example described earlier.

Step 1: Critical Movement Search Table

Enter the movement and phase data in the critical movement search table (*Table 4.1a*). For each movement, calculate (*Table 4.1b*) $y = q/s$, $u = y/x_p$, $(100u + \ell)$ and $^{1}(t_m = G_m + I)$ values. It is sufficient to calculate y and u with two digit accuracy and $(100u + \ell)$ as a whole number. Determine the required movement time, t as $(100u + \ell)$ or $^{1}t_m$, whichever is the greater.

If t is chosen as t_m:

(a) reset the lost time to t_m as shown for movements 6 and 7 in *Table 4.1a*; and

(b) cross out the flow ratio, y, and the green time ratio, u, so that they are not included in the respective intersection parameters Y and U, if this happens to be a critical movement.

For pedestrian movements, ignore the y and u values (i.e. use zero values in effect), and always set $t = t_m$ as shown for movements 6 and 7 in *Table 4.1b*.

Step 2: Critical Movement Search Diagram

Draw a critical movement search diagram from the phasing diagram (*Fig. 4.1a*) or the movement-phase data given in *Table 4.1a*, as shown in *Fig. 4.2a*. The diagram can be prepared as follows.

(1) Draw nodes (circles) from phase A to phase A in order, e.g. A, B, C, A in *Fig. 4.2a*.

(2) Connect the nodes by links (movements) according to the start and end phase data (phase-movement matrix given in *Table 4.1a*, e.g. movement 1 connects A to C (overlap), movement 2 connects A to B (non-overlap) and so on.

(3) If all movements are not exhausted using nodes A to A, extend the diagram by adding node B, C etc. as required, e.g. movement 4 requires the addition of node B in *Fig. 4.2a* (do not repeat the links which have already been entered in the diagram).

Note that, in most cases, it is possible to draw a search diagram with no movements intersecting each other.

Step 3: Non-overlap Movements

Compare t values of the non-overlap movements in each phase, choose the movement with the largest t value (representative movement) and eliminate others. For example, $t_3 > t_6$ in phase B and $t_7 > t_5$ in phase C; hence eliminate movements 6 and 5. Either prepare a reduced search diagram as shown in *Fig. 4.2b*, or simply cross out the links to be eliminated from the full diagram.

For a phasing system with no overlap movements, the critical movements are simply those left in the reduced diagram (one movement per phase). Go to Step 5 in this case.

Step 4: Overlap Movements

Similarly, compare t values of the overlap movements which receive right of way during the same phases, i.e. have the same starting and terminating phase numbers, choose the movement with the largest t value and eliminate others.

Identify possible movement combinations (paths) which complete one cycle in the reduced search diagram. For example, (1 and 7) or (2, 3 and 7) from node A to node A, and (3 and 4) from node B to node B. The critical movements are (1, 7) or (2, 3, 7) or (3, 4), whichever corresponds to the longest (critical) path. This is determined by summing the required movement time values, i.e.

$$T_{1,7} = t_1 + t_7 = 27 + 22 = 49$$

$$T_{2,3,7} = t_2 + t_3 + t_7 = 22 + 37 + 22 = 81$$

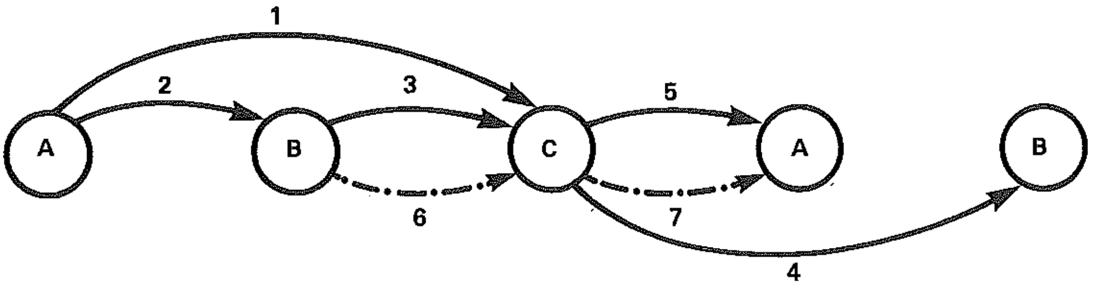
$$T_{3,4} = t_3 + t_4 = 37 + 60 = 97$$

Because $T_{3,4}$ is the largest required total time, the critical movements are (3 and 4).

Step 5: Intersection Parameters

Calculate the intersection parameters L (lost time), Y (flow ratio) and U (green time ratio) as the sum of the corresponding critical movement parameters. In the example, $L = \ell_3 + \ell_4 = 4 + 8 = 12$, $Y = y_3 + y_4 = 0.28 + 0.47 = 0.75$, and $U = u_3 + u_4 = 0.33 + 0.52 = 0.85$.

(a) Full diagram



(b) Reduced diagram

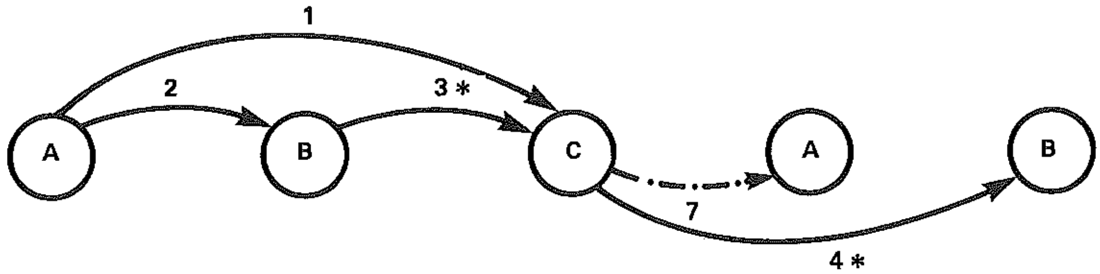


Fig. 4.2 — Critical movement search diagram (for the example in Fig. 4.1)

Note that L must include the minimum green time value of any critical movement whose required value is $t = t_m$, and (Y, U) values must not include the corresponding values (y, u) for such movements. This will be automatically achieved if Step 1 of the procedure is carried out correctly.

Step 6: Cycle Time Calculations

Calculate the practical and approximate optimum cycle times (see eqns (7.1) and (7.2) in Section 7) using the values of L, U, Y calculated above. Using a stop penalty parameter of $k = 0.2$, the approximate optimum cycle time for the present example is

$$c_o = \frac{1.6\,L + 6}{1 - Y} = \frac{1.6 \times 12 + 6}{1 - 0.75} = 101 \text{ s}$$

The practical cycle time is

$$c_p = \frac{L}{1 - U} = \frac{12}{1 - 0.85} = 80 \text{ s}$$

This is the minimum cycle time required to achieve the practical degree of saturation constraints specified by data ($x = x_p$ for critical movements, and $x < x_p$ for non-critical movements).

Step 7: Cycle Time Choice

Choose a cycle time, c between c_p and c_o, also to satisfy the condition that a maximum acceptable cycle time value c_{max} (normally 120–150 s) is not exceeded. For co-ordinated signals, the cycle time choice is based on area considerations (see Appendix I). For all practical purposes, it is sufficient to choose a cycle time rounded up to the nearest multiple of 5 s or 10 s. For the present example, choose c = 90.

Step 8: Checking Critical Movements

For the chosen cycle time, *check* if the critical movements are valid (there may be some cases when the critical movements are dependent on the cycle time, in particular when some total required movement times, T, are similar). For this purpose, calculate $(uc + \ell)$ values, determine the new required movement time t', as $(uc + \ell)$ or t_m, whichever is larger, and calculate new total required times, as in Step 4. In the example (see *Table 4.1b* and *Fig. 4.2b*):

$$T_{1,7} = 25 + 22 = 47$$
$$T_{2,3,7} = 20 + 34 + 22 = 76$$
$$T_{3,4} = 34 + 55 = 89$$

Movements 3 and 4 are the critical ones, and hence there is no change. If the critical movements change at this stage, repeat the procedure from Step 5.

Step 9: Green Times

Calculate the movement and phase green times, as well as the phase change times, according to the method described in Section 7. For practical reasons, round the green times to the nearest second. For the present example, firstly the critical movement green times are found as $g_3 = 30$ and $g_4 = 48$ (note that the sum is $g_3 + g_4 = 78 = c - L$). Then the non-critical movement green times are found as $g_7 = 18$ $(= g_m)$, $g_2 = 29$, and $g_1 = 62$ (see *Table 4.1b*). The corresponding phase green times are $G_A = 28$, $G_B = 29$, $G_C = 17$, and the phase change times are $F_A = 0$, $F_B = 34$, $F_C = 68$ (see Appendix C for detailed calculations).

Step 10: Movement Degrees of Saturation

As a final check, calculate movement degrees of saturation, x, using the allocated green times to see if $x \leq x_p$ for all movements (for the present example, see the last column of *Table 4.1b*). This condition will be satisfied unless the practical cycle time is found to be larger than the maximum cycle time ($c_p > c_{max}$) and therefore the chosen cycle time is the maximum cycle time ($c = c_{max}$). Finding $x > 1$ with this cycle time indicates that oversaturation is inevitable unless steps are taken to improve intersection conditions (see Section 8.4).

4.4 DISCUSSION

The critical movement identification technique allows the application of traditional and new capacity and timing methods (Webster and Cobbe 1966; Miller 1968*b*; Messer and Fambro 1977; Akcelik 1979*a*) to any phasing system from a simple one with no overlaps to a complicated one with multiple overlaps. The method will allow the traffic engineer to consider more than one phasing alternative including some phasing systems which could not be analysed manually before.

The method has been designed in such a way that minimum green time constraints are automatically satisfied. Differences in movement lost times are also automatically allowed for. Pedestrian movements can be described explicitly to represent both the concurrent vehicle-pedestrian and the pedestrian-only phase arrangements. The associated lost times are correctly allowed for in the capacity and timing analysis. Any special clearance, and early or late release periods of fixed value can be described as start or end lag times for appropriate movements. By including these periods in the relevant movement lost times, such special facilities are easily handled using this method.

Another feature of the method described in this section is that it allows for the calculation of signal timings which give different degrees of saturation (levels of service) for different critical movements. The unequal-degree-of-saturation method can be used to achieve a degree of priority (and restraint). For example, it may be desirable to give a higher level of service to a major arterial carrying a heavy volume of traffic (priority) and a lower level of service to an intersecting minor road to limit the amount of traffic entering the major arterial (gating control). This would be of particular value in a co-ordinated signal system with queueing problems on the major road.

The critical movement identification method is applicable to both isolated and co-ordinated signals. The only difference is in the calculation and choice of cycle time (Steps 6 and 7). For an intersection in a co-ordinated signal system the calculation of a practical cycle time, c_p, may be sufficient. The approximate optimum cycle time formula given in Section 7 is for isolated intersections. The cycle time to be chosen for use at an intersection in a co-ordinated system depends on area considerations.

In some cases, a movement may have two green periods per cycle, i.e. it may receive right of way twice during non-consecutive green periods within one cycle (see Appendix A). In these cases, the same saturation flow applies during both green periods but there are two lost times and the minimum green time applies twice. The critical movement identification method can be used by entering the movement in the search table and the search diagram twice with the required movement times calculated as ($100\ (u/2) + \ell$) or ($t_m = G_m + I$) for each green period separately. Then, the normal method can be applied by treating the movement as two separate links each with its own required time. A more complicated case is where the saturation flows for the two green periods are different (due to a phasing system which allows for opposed turns during one period and unopposed turns during another period). This case is discussed in detail in Appendix F.2.

The example given in this section demonstrates a multiple overlap case where only two movements (3 and 4) are identified as critical movements. It should be noted that as arrival flow rates change throughout the day the critical movements will also change (see Example 1 in Appendix D for a variation to the present example to demonstrate this). Further examples are given in Appendix D to show the application of the method given in this section to various examples from the literature (Allsop and Murchland 1978; Messer and Fambro 1977; Webster and Cobbe 1966; Yagar 1975; Zuzarte Tully and Murchland 1977).

5. SATURATION FLOWS

The saturation flow concept introduced in Section 2 as part of the basic model represents the most important single parameter in the capacity and timing analysis of signalised intersections. The basic model states that the saturation flow is the maximum constant departure rate from the queue during the green period. An important point to note is that the lost time concept is integral to the saturation flow concept and the success of the signal design exercise depends on correct use of these two parameters.

The factors which influence saturation flows are the environment class (defined according to the degree of interference to the free movement of vehicles), lane type (defined according to the characteristics of turning manoeuvres), lane width, gradient and traffic composition. The cases of lane underutilisation (unused lane capacity) and short lane (limited lane length, parking on the approach road, etc.) are also important in this respect.

These factors can be taken into account by proper description of *movements* from an approach road as described in this section. This report recommends calculating individual movement saturation flows by summing saturation flows of individual lanes allocated to each movement. This contrasts with the method of expressing saturation flows as a function of the total approach width (Webster and Cobbe 1966; Highway Research Board 1966). By expressing saturation flows on a lane-by-lane basis, traffic from individual lanes can be considered separately, or combined together so as to describe movements to be used as the basis of intersection capacity and timing calculations. The recommended movement description method is given below, followed by descriptions of saturation flow measurement and estimation methods.

5.1 MOVEMENT DESCRIPTION

The movements from an approach road must be described in such a way that each movement has a unique set of characteristics which determine its flow, saturation flow and lost time values, and hence the time to be allocated to meet its capacity requirements. In the capacity and timing analysis, separate calculations are made for each movement and the requirements of all movements are compared so as to identify those which determine the intersection capacity and timing requirements (Section 4).

The movements are described primarily according to the right of way provisions as determined by the signal phasing system. Secondly, various movements from an approach road can be described according to lane allocation and lane utilisation using the following rules:

(a) describe traffic in an exclusive lane as a separate movement;

(b) describe traffic in an under-utilised lane as a separate movement; and

(c) combine traffic in lanes (including the shared lanes) with equal utilisation and describe the combination as a separate movement.

These rules indicate that movement saturation flows depend on the distribution of traffic among lanes. The traffic engineer's design decision regarding the allocation of various lanes to various movements (as exclusive or shared lanes) will therefore affect saturation flows. At the same time, drivers' lane choices will determine the saturation flows under a given lane arrangement. A detailed discussion of the subject of lane utilisation is given in Section 5.7. In summary, the degrees of saturation in various lanes will be equal under equal lane utilisation conditions. The under-utilised lanes will have lower degrees of saturation (i.e. capacity is available but not used). In order to identify lane under-utilisation cases, turning flow proportions (related to destinations at not only the intersection under study but also the downstream intersection) as well as driver behaviour related to perceived delays in various lanes (e.g. avoiding the lanes with opposed movements or the lanes carrying large volumes of heavy vehicles, buses, trams, etc.) must be considered. It is useful to note at this stage that the critical movement identification technique described in Section 4 will automatically eliminate from further consideration a movement which represents an under-utilised lane because it will have a lower 'required movement time'.

5.2 MEASUREMENT OF SATURATION FLOWS

Ideally, saturation flows and lost times should be *measured* in the field. A method which can be used for this purpose is described in Appendix E. However, there is a need for a method to *estimate* saturation flows for new signal installations. Also, when designing for modifications to an existing signalised intersection, the changes to saturation flows must be estimated in relation to changing intersection geometry (number and width of lanes, turning radii, lane lengths, etc.), lane allocation and phasing arrangements. Saturation flow depends on traffic composition, i.e. the proportion of heavy vehicles and the proportion of turning vehicles, and therefore there is a need to estimate changes in saturation flow as the traffic composition changes. Even the changes to signal timing will affect saturation flows of opposed movements and movements using short lanes (e.g. a kerb lane with parking), therefore these changes must be predicted for detailed design purposes. A saturation flow estimation method is described below to allow for the above factors.

The main disadvantage of using estimated rather than measured saturation flows is that the estimates will be based on general *averages* which may not be valid for a particular case. It is therefore recommended that saturation flows (and lost times) are measured in the field wherever possible using the method given in Appendix E. This can be used to establish the basic saturation flow values for a particular site (to replace general average values given in *Table 5.1*). These values can then be used as a basis for estimating saturation flows under modified design conditions. The field measurements are of particular use for identifying lane under-utilisation and other causes of reduced saturation flow (e.g. downstream congestion) at a particular site.

5.3 ESTIMATION OF SATURATION FLOWS

5.3.1 The Method

The following method is recommended for estimating movement saturation flows.

(a) For each lane allocated to the movement, *choose* a base saturation flow value from *Table 5.1* which gives *general average* saturation flows in *through car units per hour (tcu/h)* classified by the environment and lane type as defined below.

(b) *Adjust* the base saturation flow value to allow for various factors affecting saturation flow in order to obtain an *estimate* of saturation flow in *vehicles* /h for the particular movement.

(c) *Add* lane saturation flows to determine the movement saturation flow.

The adjustments required are explained below in detail. Some adjustments can be made to the movement saturation flow after adding lane saturation flows adjusted for other factors individually. In other words, steps (b) and (c) above can in fact be combined, or interchanged, as required. *This report recommends the use of saturation flow, measured or estimated, in vehicle units in the capacity and timing calculations in a consistent manner.* This approach prevents various errors which may be introduced by using arrival flows and saturation flows in through car units (see Appendix F).

5.3.2 The Basic Table

The base saturation flow values (in tcu/h) given in *Table 5.1* by environment class and lane type are based on extensive surveys carried out in Australia (Miller 1968*a* and *b*). However, the environment class and lane type definitions given below differ from those used in Miller (1968*b*).

Environment Classes

Class A: Ideal or nearly ideal conditions for free movement of vehicles (both approach and exit sides): good visibility; very few pedestrians; almost no interference due to loading and unloading of goods vehicles or parking turnover (typically but not necessarily in a suburban residential or parkland area).

Class B: Average conditions: adequate intersection geometry; small to moderate numbers of pedestrians; some interference by loading and unloading of goods vehicles, parking turnover, and vehicles entering and leaving premises (typically but not necessarily in an industrial or shopping area).

Class C: Poor conditions: large numbers of pedestrians; poor visibility; interference from standing vehicles, loading and unloading of commercial vehicles, taxis and buses, and high parking turnover (typically but not necessarily in a city centre area).

TABLE 5.1

AVERAGE SATURATION FLOWS IN THROUGH CAR UNITS PER HOUR FOR ESTIMATION BY ENVIRONMENT CLASS AND LANE TYPE

Environment Class	*Lane Type*		
	1	*2*	*3*
A	1850	1810	1700
B	1700	1670	1570
C	1580	1550	1270

Lane Types

Type 1: Through lane: a lane which contains through vehicles only.

Type 2: Turning lane: a lane which contains any type of turning traffic (exclusive left turn or right turn lane, or a shared lane from which vehicles may turn left or right or continue straight through). Adequate turning radius and negligible pedestrian interference to turning vehicles.

Type 3: Restricted turning lane: as for type 2, but turning vehicles are subject to small radius and/or some pedestrian interference.

It is seen from these definitions that various factors are implicit to the values given in *Table 5.1*. This simplifies the estimation procedure. However, some of these factors should be considered by the traffic engineer for modification in order to improve the intersection capacity, in particular for Environment Class C and Lane Type 3 conditions.

An important point to note in relation to the basic table is the site-to-site variations in saturation flow. Miller (1968*b*) stated that the standard deviation between sites was about 200 tcu/h for each class, and that only about 10 per cent of saturation flow differed by more than 300 tcu/h from those predicted (in other words, lane saturation flows in access of 2000 tcu/h were observed at various sites). Because this degree of variation is significant in practice, it is recommended that the saturation flows should be measured wherever possible, particularly at problem sites.

5.3.3 Adjustment Factors

The base saturation flow values taken from *Table 5.1* must be adjusted to allow for various factors which are not implicit to the values given in the table. These adjustments can be made to each lane separately, or to the movement as a whole after adding lane saturation flows if the factor is applicable to all lanes equally. The method can be summarised by the formula:

$$s = (f_w f_g / f_c) s_b \qquad (5.1)$$

where

s	=	saturation flow estimate in *vehicles* /h,
s_b	=	base saturation flow in *through car units* per hour (tcu/h) (from *Table 5.1*),
f_w	=	lane width factor,
f_g	=	gradient factor, and
f_c	=	traffic composition factor (tcu's per vehicle for a particular vehicle type and turning traffic mix).

The calculation of the adjustment factors are described in the following sections. It should be noted that the factors f_w, f_g are expressed below as constant adjustment parameters, whereas the traffic composition factor, f_c, may be a function of signal timings if there are opposed turns, in which case the calculations must be repeated until it is considered that the changes in the predicted saturation flows and signal timings are not significant.

5.4 LANE WIDTH AND GRADIENT

There is no need for adjustment for a lane width in range 3.0 to 3.7 m. For a lane width outside this range, an adjustment factor, f_w, must be calculated from the formula given below in order to scale up the base saturation flow value for a wide lane, or scale it down for a narrow lane:

$$f_w = \begin{cases} 1.0 & \text{for } w = 3.0 \text{ to } 3.7 \\ 0.55 + 0.14\,w & \text{for } 2.4 \leqslant w < 3.0 \\ 0.83 + 0.05\,w & \text{for } 4.6 \geqslant w > 3.7 \end{cases} \qquad (5.2)$$

where w is the lane width (m). The values of $f_w = 0.89$ for $w = 2.4$ m and $f_w = 1.06$ for $w = 4.6$ m can be considered to be lower and upper limits for the application of this formula.

For a varying lane width, the width at the narrowest point within 30 m of the stopline must be used in eqn (5.2). The exit lane must be at least as wide as the approach lane. Otherwise, the width of the exit lane must be used as the lane width. If an approach is flared to allow extra lanes, it should be assumed that no through vehicle will use a lane which does not continue on the exit side of the intersection. If a lane is of limited length, this should be treated as a *short lane* case (see Section 5.8).

If the effect of gradient is considered to be significant, the saturation flow should be multiplied by the factor:

$$f_g = 1 \pm 0.5\,(G_r/100) \qquad (5.3)$$

where G_r is the per cent gradient. Use $+G_r$ for downhill gradient to increase the saturation flow, and $-G_r$ for uphill gradient to decrease it. For example, $f_g = 0.95$ for an uphill gradient of $G_r = 10$ per cent.

It should be noted that the saturation flow value is still in through car units after the adjustments for lane width and gradient ($s_{tcu} = f_w\, f_g\, s_b$). A blank form is given in Appendix J which can be used for the calculation of this saturation flow.

5.5 TRAFFIC COMPOSITION

A traffic composition adjustment factor, f_c, must be calculated to modify the saturation flow value after adjustments for lane width and gradient so as to convert it from the standard through car units to vehicles ($s_{veh} = s_{tcu}/f_c$). This is determined by the proportions of various vehicle types and turning traffic which make up the movement under consideration. In this guide, the effects of different types of vehicle and turn are expressed in terms of *through car equivalents*, i.e. through car units per vehicle for each vehicle-turn combination (a through car unit is simply a straight-ahead car).

The traffic composition factor is calculated from

$$f_c = \frac{\Sigma e_i q_i}{q} \tag{5.4}$$

where

q_i = flow in vehicles for vehicle-turn type i,

q = total movement flow ($= \Sigma q_i$), and

e_i = through car equivalent of vehicle-turn type i (tcu/veh).

The value of f_c can be considered to be a flow weighted average through car equivalent (in tcu/veh as for e_i). It is recommended that the traffic composition adjustment factor given by eqn (5.4) is calculated using a table as shown in Examples 1 and 2 in Section 9. A blank form is given in Appendix J.

TABLE 5.2

THROUGH CAR EQUIVALENTS (tcu/veh) FOR DIFFERENT TYPES OF VEHICLE AND TURN

	Through	*Unopposed Turn*		*Opposed Turn*
		Normal	*Restricted*	
CAR	1	1	1.25	e_0
HV	2	2	2.5	$e_0 + 1$

The through car equivalents for use in eqn (5.4) can be taken from *Table 5.2*. These are *average* values based on surveys in Australia (Miller 1968*a* and *b*). A detailed discussion of the meaning of through car equivalents is given in Appendix F. For example, a through heavy vehicle (HV) is equivalent to 2 through cars (CAR's) according to *Table 5.2*. This means that, if the base saturation flow (*Table 5.1*) is 1700 tcu/h, the saturation flow of a movement which consists of through HV's only is 1700/2 = 850 veh/h. The definitions of the vehicle and turn types used in *Table 5.2* are given below.

5.5.1 Types of Vehicle

For the purpose of this report, only two vehicle types are considered. Any vehicle with more than two axles or with dual tyres on the rear axle is defined as a heavy vehicle (HV). Thus buses, trucks, semi-trailers (articulated vehicles), cars towing trailers or caravans, tractors and other slow-moving vehicles are classified as HV's. All other (light) vehicles are defined as CAR's.

5.5.2 Types of Turn

The effect of turning traffic (left or right) depends on whether it is an *opposed* movement (i.e. if it has to give way to, and look for gaps in, a higher priority *opposing* movement), or it is an *unopposed* movement. The opposing movement may be a vehicle or a pedestrian movement.

(a) *Unopposed turns:*

(i) *Normal:* This applies to both left-turning and right-turning vehicles and represents the conditions where the radius of curvature of the turn is reasonably large and there is no (or very little) pedestrian interference. For example, an equivalent of 1 for cars for unopposed normal turns in *Table 5.2* means that the saturation flow of turning cars under these conditions is the same as the base value given in *Table 5.1* (assuming standard lane width and level gradient). Based on the results of experiments and observations reported by Webster (1964) and Ellson (1969), the effect of curvature is very small if the radius of curvature of the turn is at least 15 m. Ellson's (1969) formula $s = 1850/(1 + 100\,r^{-3})$, where r is the radius of the path of turning cars (m), is useful for predicting the effect of this factor.

(ii) *Restricted:* This applies to both left-turning and right-turning vehicles which are subject to a smaller turning radius and some interference by pedestrians. Turning vehicles subject to interference by heavy pedestrian flows can be treated as opposed turns.

(b) *Opposed turns:* The value of the opposed turn equivalent e_0 for CAR's ($e_0 + 1$ for HV's) depends on signal timings and opposing movement characteristics. A method is given below for the calculation of e_0.

5.6 OPPOSED TURNS

The methods described below for the treatment of opposed turns can be used for any of the following opposed movement cases by choosing appropriate parameter values:

(a) filter right turns giving way to vehicles in an opposing stream;

(b) left turns, or right turns from one-way streets, giving way to pedestrians;

(c) filter left turns under 'left turn at any time with care' regulation, giving way to pedestrians or to vehicles in the opposing through and right-turn streams; and

(d) filter left turns according to 'left turn on red' rules.

The basis of the methods given below is described in detail in Appendix F. Different methods can be used for the shared lane and exclusive lane cases as described below. Examples 1 and 7 in Section 9 shows the use of these methods. The parameters used in the following equations are illustrated in *Fig. F.1* in Appendix F.

5.6.1 A Shared Lane

When the opposed turning vehicles share the same lane with other vehicles, the saturation flow must be

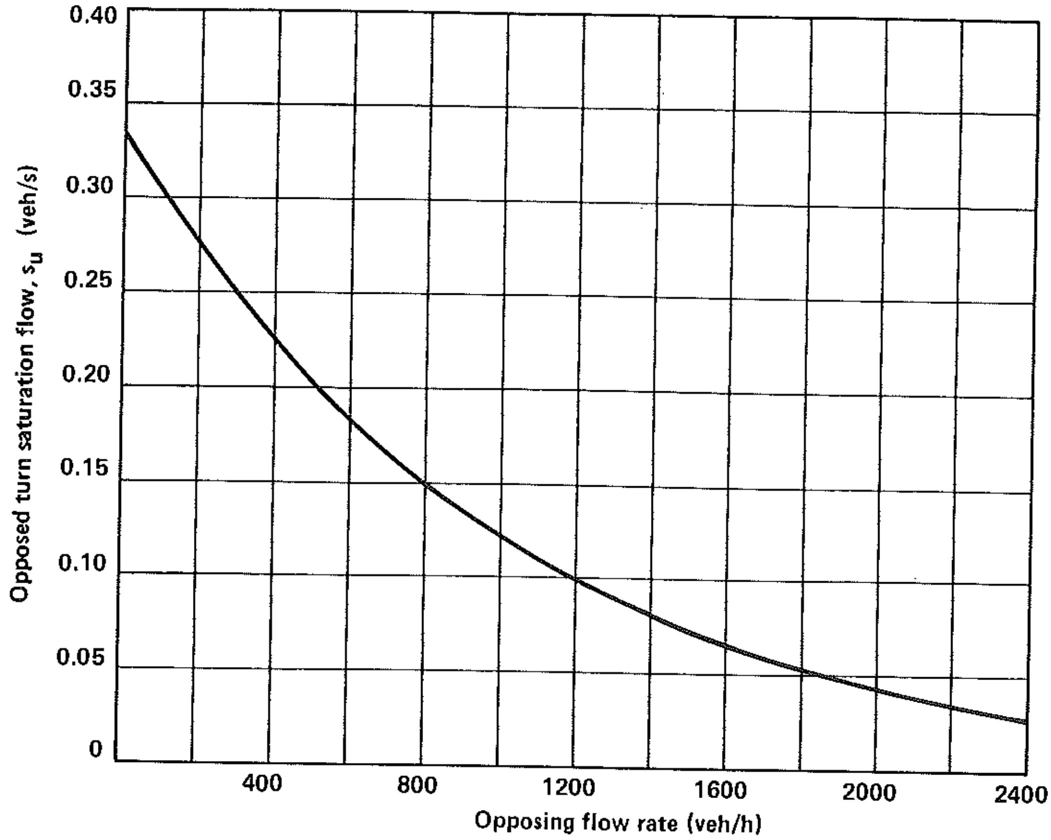


Fig. 5.1 — Opposed turn saturation flow during the unsaturated part of the opposing movement green period

calculated from eqns (5.1) to (5.4) using an opposed turn equivalent given by:

$$e_o = \frac{0.5g}{s_u g_u + n_f} \tag{5.5}$$

where

g	=	green time (s) for the movement with opposed turns,
s_u	=	opposed turn saturation flow (veh/s) given in *Fig. 5.1* as a function of the opposing movement flow rate,
g_u	=	unsaturated part of the opposing movement green (s) given by eqn (5.10),
$s_u g_u$	=	number of turning vehicles (per cycle) which can depart during the period g_u, and
n_f	=	number of turning vehicles (per cycle) which can depart after the green period from the shared lane in question.

The constant 0.5 is an approximation for the base saturation flow in tcu/s (= 1800 tcu/h) used for obtaining the opposed turn equivalent.

5.6.2 An Exclusive Lane

The above method can be used in this case also. The through car equivalent, e_o, given by eqn (5.5) can be used to calculate an effective opposed turn saturation flow from:

$$s_o = 1800/e_o \tag{5.6}$$

where 1800 is the base saturation flow (in tcu/h) used in eqn (5.5) and s_o is in veh/h.

Alternatively, the opposed movement saturation flow can be taken directly as s_u and an effective opposed movement green time can be calculated from:

$$g_o = g_u + (n_f/s_u) \tag{5.7}$$

where g_u, s_u and n_f are as in eqn (5.5). The corresponding lost time for use in critical movement identification and signal timing calculations is:

$$\ell_o = (G + I) - g_o \tag{5.8}$$

where $(G + I)$ is the sum of displayed green and intergreen times for the opposed turn phase.

5.6.3 Common Parameters

The capacity, Q, for the opposed movement in an exclusive lane is the same with both methods, and is given by:

$$Q = s_o\, g/c = s_u\, g_o/c \tag{5.9}$$

where Q, s_o and s_u are in veh/h. Although the same capacity, and hence the same degree of saturation is predicted by both methods, the use of eqns (5.7) and (5.8) improves the prediction of operating characteristics (delay, number of stops and queue length).

In the case of two exclusive turning lanes, the opposed movement saturation flow is $2s_o$, or $2s_u$, depending on the method used. Note that s_u and n_f used for calculating e_o and g_o (eqns (5.5) and (5.7)) are parameters per lane.

For eqns (5.5) and (5.7), the unsaturated part of the opposing movement green period can be calculated from:

$$g_u = \frac{g - yc}{1 - y} \qquad (5.10)$$

where y and g are the flow ratio and effective green time for the *opposing movement*, and c is the cycle time.

Eqn (5.10) is valid for $sg > qc$, i.e. when the opposing movement is undersaturated ($x = qc/sg < 1$). For the conditions when $x > 1$, simply use $g_u = 0$, in which case the capacity for the opposed movement is merely due to the departures after green (n_f). Furthermore, eqn (5.10) does not allow for any overflow queues which may be significant even for undersaturated conditions (see Section 6). For more elaborate calculations, the value of g_u from eqn (5.10) can be reduced by $N_o/(s - q)$, where N_o is the average overflow queue for the opposing traffic (eqn (6.1)).

It must be noted that the green time (g) in eqn (5.10) is not necessarily the same as the green time in eqn (5.5). Even if the movement with opposed turns and the opposing movement receive the same displayed green, different lost times may result in different effective green times. Furthermore, the displayed green times may be different, i.e. the opposing movement may be started earlier, or terminated later. It is therefore important that g_u from eqn (5.10) is calculated with due consideration to the opposing movement phasing. It must be reduced by an amount equal to the extra time for the opposing movement which is terminated later than the opposed movement (see *Fig. F.2* in Appendix F).

The value of n_f (number of departures after green) is important in determining the opposed movement capacity, particularly when the opposing movement green approaches saturation (hence $s_u\, g_u \approx 0$). It is therefore recommended that this parameter is chosen for the particular movement under consideration. It can be easily measured in the field (see Appendix E). A general purpose value of $n_f = 1.5$ can be used if the field data are not available. When the opposed green period is followed by an unopposed green period without an intergreen, then $n_f = 0$ must be used because any departures after the end of the opposed green period are part of the unopposed green period. This is a case of two saturation flows per cycle for the turning movement (see Appendix F).

It is important to note that, in cases where there are more than one opposing movement, the more critical one (smaller g_u) must be used as the opposing movement. For example, if the traffic in an under-utilised lane of the opposing approach road is described as a separate movement, the movement consisting of vehicles in other lanes must be used to determine g_u. However, s_u must be determined as a function of the sum of all opposing movement flows (for *Fig. 5.1*, or eqn (F.9) in Appendix F).

5.6.4 Iterative Calculations

Eqns (5.5) to (5.10) indicate that the opposed turn saturation flows, or effective green and lost times depending on the method used, are functions of signal timings. This requires that the signal capacity and timing analysis must be an iterative process starting with some initial estimates of timing or saturation flow values. It is recommended that the timings for the intersection are calculated assuming unopposed departures initially. Using these timings, the opposed turn characteristics can be calculated, and in turn these can be used to re-calculate signal timings. The calculations must be repeated until no significant changes are obtained in signal timings. The number of iterations can be reduced by keeping the cycle time constant after one or two iterations (see Example 7 in Section 9). For general purpose capacity analysis, an average value of $e_o = 3$ ($s_o = 600$ veh/h for an exclusive lane) can be used. It is also recommended that a fixed value of $e_o = 3$ is used for opposed turns in shared lanes (so as to avoid iterative calculations) when the opposed turns are only a small proportion of total movement flow. The value of e_o has little effect on the results in this case (e.g. for Movements 2 and 4 in Example 7 in Section 9).

5.7 LANE UTILISATION

On multi-lane approach roads, it is possible that the capacities of all lanes are not used fully, i.e. some lanes are *under-utilised.* This has important implications for signal design, and hence lane under-utilisation effects must be accounted for correctly in the capacity and timing calculations. Measures to prevent lane under-utilisation must be taken wherever possible.

The traffic engineer's design decision regarding lane arrangements (number and type), coupled with the choice of a phasing system, is very important in influencing lane utilisation. For a given lane arrangement/phasing system, drivers' lane choices will determine lane utilisation and saturation flows. Drivers can be considered to select lanes in accordance with two basic principles:

(a) according to their required destination, reflected in the turning volumes, not only at the intersection under study, but also at downstream intersections; and

(b) in accordance with *perceived* delays, due both to the length and the composition of the queue (due to the existence of heavy vehicles and opposed turners in particular), and also to interference from parked vehicles, bus stops and mid-block turners.

Various cases of lane under-utilisation which may result from drivers' lane choices are illustrated in *Fig. 5.2.*

Because the lane utilisation at a particular intersection is affected by destinations beyond that intersection as well as behavioural factors based on perceived rather than actual delays (or some other 'performance criteria'), it is difficult to describe a simple predictive method for this purpose. Field observations at the particular intersection under study are of particular importance for this reason.

The following discussion is given in order to facilitate a better understanding of the lane utilisation problem and to provide a basis for the treatment of kerb lane under-utilisation on multi-lane approach roads.

A *lane utilisation ratio*, ρ_i, for the ith lane can be defined as:

$$\rho_i = x_i/x \qquad (5.11)$$

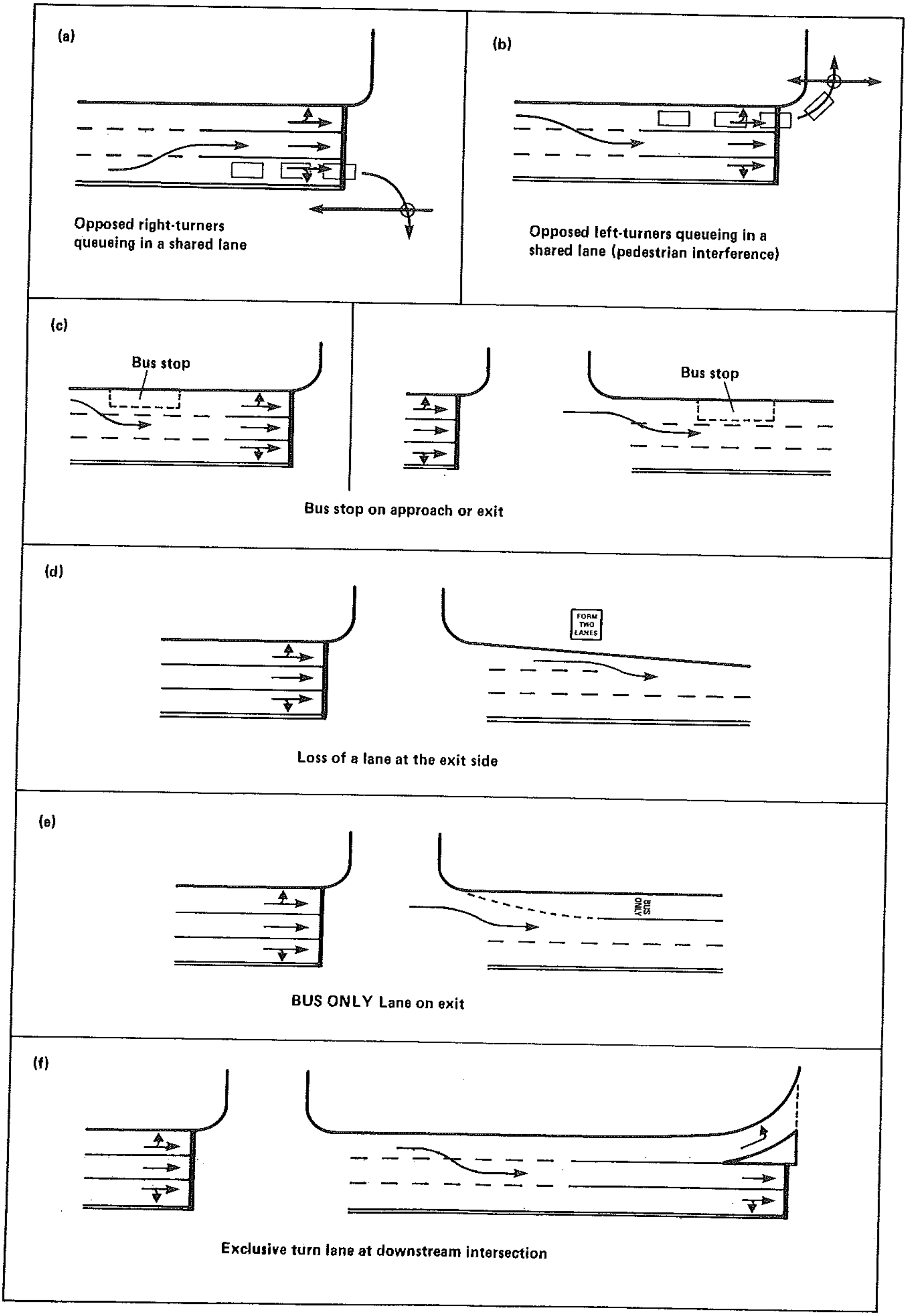


Fig. 5.2 — Various cases of potential lane under-utilisation

where

x_i = degree of saturation of the ith lane, and

x = the critical (largest) degree of saturation in any lane of the approach road.

According to eqn (5.11) *equal* lane utilisation means equal lane degrees of saturation, i.e. $x_1 = x_2 = \ldots = x_n = x$, and $\rho_1 = \rho_2 = \ldots = \rho_n = 1.0$, where n is the number of lanes. The degree of saturation is the ratio of flow to capacity, and hence equal lane utilisation means:

$$\frac{q_1}{Q_1} = \frac{q_2}{Q_2} = \cdots = \frac{q_n}{Q_n} = \frac{q_1 + q_2 + \ldots + q_n}{Q_1 + Q_2 + \ldots + Q_n}$$

$$= \frac{q}{Q} = x \qquad (5.12)$$

where q and Q are the movement flow and capacity values obtained as the sum of the lane flow and capacity values, q_i and Q_i, respectively ($i = 1,2,\ldots,n$).

Eqn (5.12) allows the calculation of movement degrees of saturation without the need to calculate individual lane flows for the lanes whose capacities are used equally. In this case, traffic in all lanes can be combined together and described as a single *movement* with a degree of saturation equal to x. This is the basis of the movement description method given in Section 5.1.

Eqns (5.11) and (5.12) allow for different lost times, hence different effective green times for different lanes. A simplification can be introduced by assuming that the effective green times are the same for all lanes. In this case, the lane utilisation ratio is:

$$\rho_i = y_i / y \qquad (5.13)$$

where

y_i = flow ratio (flow/saturation flow) of the ith lane, and

y = flow ratio of the critical lane.

For lanes with equal utilisation, $y_1 = y_2 = \ldots = y_n = y$, and eqn (5.13) becomes

$$\frac{q_1}{s_1} = \frac{q_2}{s_2} = \cdots = \frac{q_n}{s_n} = \frac{q_1 + q_2 + \ldots + q_n}{s_1 + s_2 + \ldots + s_n}$$

$$= \frac{q}{s} = y \qquad (5.14)$$

Therefore, for the lanes whose capacities are used equally, and whose effective green times are the same, the movement saturation flow can be calculated simply by adding lane saturation flows.

The lane flows can be calculated from the following formula based on eqn (5.12):

$$q_i = x\,Q_i \qquad (5.15)$$

or from the following formula based on eqn (5.14) when effective green times are the same:

$$q_i = y\,s_i \qquad (5.16)$$

where

q_i, s_i, Q_i = flow, saturation flow and capacity for the ith lane, and

x, y = the movement degree of saturation and flow ratio, respectively.

The above formulation of the lane utilisation problem indicates that lane flows may be significantly different even if the lane capacities are equally utilised. For example, a lane with lower capacity due to opposed turners will have a lower flow rate than other lanes (see Example 3 in Section 9). Lane under-utilisation corresponds to a different case where part of the available capacity is unused, i.e. the lane utilisation ratio, $\rho < 1$. Miller (1968*a* and *b*) reported the observed values of ρ in the range between 0.15 and 0.85.

The following are the two methods which can be used for the treatment of lane under-utilisation in signal timing and capacity calculations.

(a) Describe the traffic in the under-utilised lane and in the other lanes as two separate movements each with its own flow and saturation flow values, and hence, flow ratios, e.g. $y_1 = q_1/s_1$ for the movement representing traffic using the under-utilised lane and $y_2 = q_2/s_2$ for the movement representing traffic using the other lanes. From eqn (5.13), the utilisation ratio for the under-utilised lane is $\rho = y_1/y_2 < 1$.

(b) Alternatively, describe the traffic in all lanes as a single movement which has a flow ratio equal to the flow ratio of the traffic in the more critical lanes, $y = y_2$. This corresponds to a combined movement saturation flow of $s = \rho\, s_1 + s_2$, i.e. the under-utilised lane saturation flow is reduced by a factor of ρ before it is added to the other lane saturation flows (if ρ is known in advance, s_1 and s_2 in tcu/h can be used to calculate s in tcu/h which can then be converted to veh/h using the traffic composition factor, f_c, for traffic in all lanes).

Method (a) gives better estimates of delays, queue lengths, etc. because it uses more realistic departure patterns. Method (b) has the advantage of dealing with one movement only, but it must be based on the use of a lane utilisation ratio, ρ, determined with due consideration to the turning and through traffic proportions in the under-utilised lane.

The value of ρ can be calculated if the amount of through traffic using the under-utilised lane, q_{T1}, is known. This should normally be measured at the site under study. If q_{T1} is not available (e.g. when designing a new intersection), and it is considered that there are reasons for lane under-utilisation (see *Fig. 5.2*), $q_{T1} = q_T/2n$ can be used, where q_T = total through traffic flow and n = number of lanes available for through traffic. Then, to calculate ρ:

(i) find the under-utilised lane flow as the sum of q_{T1} and the appropriate turning flow, $q_1 = q_{T1} + q_L$; calculate the saturation flow, s_1, using a traffic composition factor based on this flow; and calculate $y_1 = q_1/s_1$;

(ii) reduce the amount of through traffic using the other lanes by q_{T1}, and calculate flow and saturation flow (q_2, s_2) accordingly (equal utilisation is assumed for these lanes; hence there is no need to calculate individual lane flows); calculate $y_2 = q_2/s_2$;

(iii) calculate $\rho = y_1 / y_2$; check if $\rho < 1$ (if the turning traffic flow is relatively high, the assumption about q_{T1} may lead to $\rho > 1$, in which case use $\rho = 1$, i.e. assume equal lane utilisation for all lanes);

(iv) if Method (b) is to be used, calculate the combined movement saturation flow as $s = \rho s_1 + s_2$.

This calculation process shows that either q_{T1} or ρ is variable when the saturation flow s_2, hence the flow ratio, y_2, is variable (e.g. because of an opposed turn equivalent which varies due to changes in signal timings). It is more convenient to treat q_{T1} as a fixed value with Method (a), whereas it is more convenient to treat ρ as a fixed value with Method (b). Examples 2 and 7 in Section 9 show the use of the methods described in this section.

5.8 SHORT LANE SATURATION FLOWS

A considerable number of situations exist whereby saturation flows on multi-lane approach roads are reduced due to what can in general terms be defined as short lane effects. Various short lane cases are illustrated in *Fig. 5.3.* The short lane effects will occur when the space for queueing is limited. This may be due to the limited length of a lane (e.g. a turn slot), parking on the approach road, a pre-intersection bus lane with a setback distance, or the blockage of a lane by the queue in the adjacent lane.

Consider the saturation flow of the movement using the approach road shown in *Fig. 5.4.* The number of vehicles which can queue in the short lane is (D/j), where D is the length of the short lane and j is the average space per vehicle in the queue. As illustrated in *Fig. 5.5*, the full saturation flow (s_1) of the short lane will last for a period $g_1 = (D/j)/s_1$ (note that $1/s_1$ is the average departure headway). Therefore, the movement saturation flow is $(s_1 + s_2)$ during the period g_1, but is reduced to s_2 (the saturation flow of the other lanes available) during the period $g_2 = g - g_1$. The capacity per cycle is $sg = (s_1 + s_2)\, g_1 + s_2\,(g - g_1)$, and hence the average movement saturation flow is

$$s = s_1 \frac{g_1}{g} + s_2 = s_1' + s_2 \qquad (5.17)$$

The first term of eqn (5.17) is the effective (reduced) saturation flow of the short lane, s_1', and since $s_1\, g_1 = D/j$,

$$s_1' = 3600\, D/jg \qquad (5.18)$$

where s_1' is in veh/h, D is in m, j is in m/veh and g is in seconds.

The average queue space per vehicle can be calculated from $j = 6\, p_1 + 12\, p_2$, where p_1, p_2 are the proportions of cars (light vehicles) and heavy vehicles in the short lane, respectively (6 m per CAR and 12 m per HV assumed). For example, for 80 per

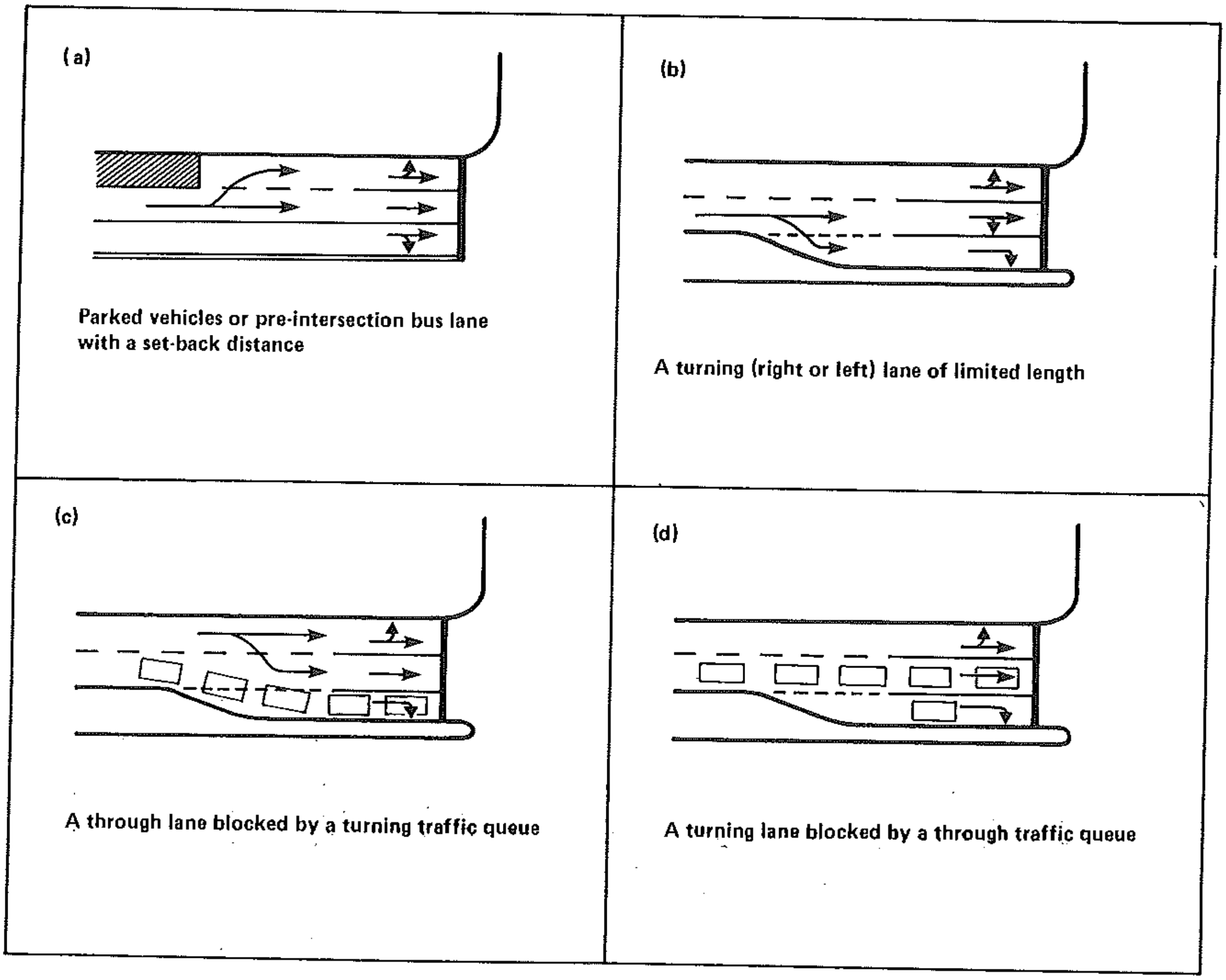


Fig. 5.3 — Short lane cases

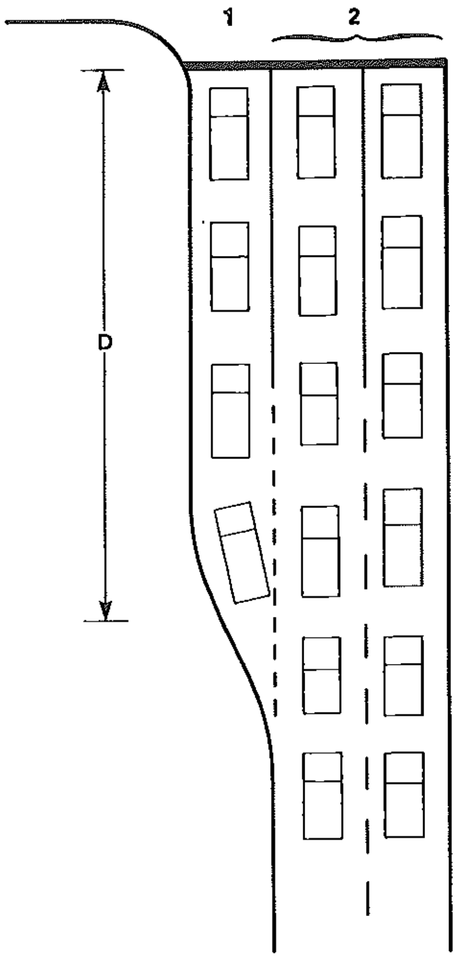


Fig. 5.4 — Queueing in an approach road with a short lane

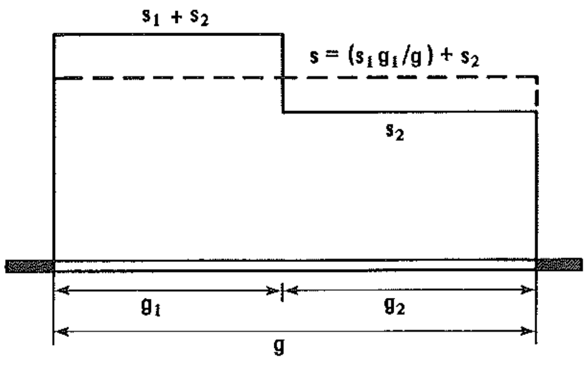


Fig. 5.5 — Short lane saturation flow

cent cars and 20 per cent HV's, $j = 7.2$ m/veh is obtained. Using this figure, the effective saturation flow of the short lane in vehicles per hour is

$$s_1' = 500\, D/g \qquad (5.19)$$

It is seen from eqn (5.19) that the short lane saturation flow can be increased to its maximum value of s_1 by either increasing the lane length or decreasing the green time. In other words, it is possible to sustain the full saturation flow rate, s_1, throughout the green period, g, if D is sufficiently long or g is sufficiently short (see Appendix F.3).

It is possible that the maximum back of the queue (see Section 6.4) is contained within the storage space provided by the short lane and the other lanes allocated to the movement in question. The full saturation flow rate is sustained in this case. It is therefore necessary to establish if the short lane effect occurs, i.e. if there is a loss in saturation flow during the green period, before using eqns (5.17) to (5.19). The following method is recommended for this purpose (see Examples 4 and 5 in Section 9).

(a) Calculate full movement saturation flow using the normal estimation method described in Sections 5.3 to 5.6. If signal timings are not known, calculate timings for the intersection using the initial estimates of saturation flows calculated assuming that short lane effects do not occur.

(b) Calculate the critical queueing distance, D_c, from

$$D_c = \frac{jqr}{n(1-y)} \qquad (5.20)$$

where

j	=	average queue space (m/veh),
q	=	arrival flow rate (veh/s),
r	=	effective red time (= c − g),
n	=	number of lanes available including the short lane, and
y	=	flow ratio (= q/s, where s is the movement saturation flow calculated in (a) assuming no short lane effects).

This formula is based on the assumption of uniform arrivals for calculating the maximum back of the queue in an average signal cycle. For more elaborate analysis, (qr) may be replaced by ($N_o + qr$) where N_o is the average overflow queue given by eqn (6.1) in Section 6.

(c) Compare the available short lane queueing distance, D, with the critical distance, D_c. If D is greater, the short lane effect does not occur and the saturation flow is as calculated in (a). Otherwise, calculate the short lane saturation flow, s_1', from eqns (5.18) or (5.19).

(d) Check if the calculated short lane saturation flow, s_1', is less than the full lane saturation flow, s_1. For this purpose, convert s_1' to tcu's as follows (note that s_1' from eqns (5.18) or (5.19) is in vehicle units because it is derived in relation to the average queue space per vehicle).

(i) Calculate $Q_1 = s_1'(g/c)$ in veh/h, where (g/c) is the green time ratio.

(ii) Calculate Q_1^* in tcu/h from

$$Q_1^* = \begin{cases} q_L^* + \dfrac{q_T^*}{q_T}(Q_1 - q_L) & \text{if } q_L < Q_1 \\[2ex] \dfrac{q_L^*}{q_L}\, Q_1 & \text{otherwise} \end{cases} \qquad (5.21)$$

where

q_L, q_L^*	=	turning flow demand to use the lane in question (left-turn in Fig. 5.4) in veh/h and tcu/h, respectively, and

q_T, q_T^* = through flow in veh/h and tcu/h, respectively.

(iii) Calculate s_1' in tcu/h as $(Q_1^*/Q_1)s_1'$, and check if this is less than s_1 (tcu/h). If so, calculate the movement saturation flow in tcu/h as $(s_1' + s_2)$, where s_2 is the sum of basic saturation flows of the other lanes allocted to the movement. The movement saturation flow in vehicle units is $s = (s_1' + s_2)/f_c$, where f_c is the traffic composition adjustment factor calculated in (a) above. If $s_1' \geqslant s_1$ is found, use the full movement saturation flow calculated in (a).

Examples 4 and 5 in Section 9 are given to illustrate the above method.

5.9 LANE BLOCKAGE

The queue forming in the adjacent lane may block a lane from being fully utilised for its full length. The two possible cases are illustrated in *Fig. 5.3.*

In the first case, a through traffic lane is blocked by a turning traffic queue which exceeds the storage capacity of the turn slot but the through lane is accessible from the other lane as shown in *Fig. 5.3(c).* This can be treated as a short lane case using the method described in Section 5.8. The available queueing distance D is to be taken as the length of the turn slot in this case (see Example 5 in Section 9). However, it is possible that the blockage is removed in time, e.g. by using a leading turn phase, and the through lane is fully utilised. This must be considered in the analysis of such cases.

In the second case, entry to an exclusive turning traffic lane is blocked by a queue in the adjacent lane as shown in *Fig. 5.3(d).* This can also be treated as a short lane problem, but with the following differences:

(a) exclusive turn lane flow is described as a separate movement,

(b) the reduced saturation flow is calculated from eqn (5.18) or (5.19) using the available queueing distance given by:

$$D = q_1/q_2\, D' \qquad (5.22)$$

where

D' = the full length of the turn slot,

q_1 = turning flow arrival rate, and

q_2 = the adjacent lane arrival flow rate.

(Note that the time taken for the lane to be blocked can be calculated as D'/jq_2 where j is the average queue space per vehicle in the adjacent lane.)

This formula assumes that the red period starts at the same time for both movements. However, if the red period for turning traffic starts earlier, additional vehicles can enter the turn slot before the through traffic queue starts forming in the adjacent lane. Similarly, if the green period for through traffic starts earlier, the blockage may be removed and additional vehicles may join the queue in the turn slot. The short lane capacity may be fully utilised as a result of this.

5.10 OTHER PROBLEMS

The basic model of saturation flow and lost time described in Section 2 assumes that the saturation flow remains constant throughout the effective green period. On the other hand, the short lane effects cause the saturation to fall off abruptly during the green period as shown in *Fig. 5.5.* Similarly, experience suggests that the saturation flow may decrease during a *long* green period because of increased departure headways of vehicles coming from the end of a long queue. It is important to note that, when such a case is observed in practice, shorter green and cycle times should be used to achieve higher capacities as in the case of short lanes (see Appendix F.3).

A further factor to be considered when estimating saturation flows is the downstream congestion effects. The blockage of downstream roads because of excessive queueing or lack of co-ordination can seriously reduce the saturation flow of the approach road under study. Although signal co-ordination by itself will not increase the capacity of any of the intersections, it may improve capacity at the upstream intersection by reducing or eliminating queueing at the downstream intersection (see Appendix I).

The basic saturation flows given in *Table 5.1* make some allowance for the effects of pedestrians, bus stops, trams, weather and time of day since they are based on measurements at intersections where these factors existed (Miller 1968*a* and *b*). However, the following should be noted about these factors.

(a) Pedestrians: If the pedestrian flow which conflicts with a turning flow is very heavy, the opposed turn saturation flow estimation method can be used to allow for the pedestrian flow interference explicitly. The calculation of the length of the unsaturated part of the green period (g_u) depends on the crossing distance and pedestrian flow levels. It is recommended that this is measured at the particular site under study together with the value of n_f for use in eqns (5.5) to (5.9).

(b) Bus stops: The effect of bus stops on the approach and exit sides can be considered in terms of lane under-utilisation (see *Fig. 5.2*).

(c) Trams: Miller (1968*a* and *b*) found that at tram stops with safety zones the trams had little effect on saturation flows whereas at intersections without safety zones, trams reduced saturation flows in the lane shared with the trams by 16 per cent and in other lanes by 5 per cent.

(d) Weather and time of day: A recent study in the U.K. (Branston 1979) show that the saturation flows in darkness and wet weather were about 100 tcu/h smaller than normal saturation flows and that the saturation flows during off-peak periods were about 150 tcu/h less than those in normal peak periods.

5.11 DISCUSSION

It can be argued that the base saturation flows and through car equivalents given in *Tables 5.1* and *5.2* (adopted from Miller 1968*a* and *b*) are based on surveys conducted a long time ago and that higher saturation flows exist today due to improvements in vehicle performance characteristics and due to changes in driving habits under higher congestion levels than found about 15 years ago. Validation of this argument would require extensive surveys to produce 'average' saturation flows in the same fashion as before. However, the usefulness of estimated saturation flows based on general averages is limited because site-to-site variations are significant. It is, therefore, recommended that *measured* saturation flows are used for detailed design purposes, wherever possible. Appendix E describes an easy method for measuring saturation flows and lost times. The field surveys are of particular value for identifying lane under-utilisation, short lane effects, downstream congestion and other causes of reduced saturation flow at a particular site.

It should also be noted that the definitions of environment classes, lane types and turn types for *Tables 5.1* and 5.2 are different from those given in Miller (1968*b*). It is considered that the present definitions are more general and more explicit in terms of the underlying reasons. For example, definitions based on geographical areas have been avoided, and turning lanes have been classified according to the ease of turn rather than as Type L (with right-turns) and Type R (with left-turns). Furthermore, one environment class has been eliminated for simplicity because a difference of 2 per cent from the next class has not been considered to be significant in view of the site-to-site variations of the order of 15 per cent.

Another important difference from the previous guide is the emphasis on the use of flows and saturation flows in vehicle units as measured in the field. The through car equivalents are needed for saturation flow estimation only. The use of arrival flows in tcu's in various calculations can cause many errors as discussed in Appendix F.1.

The need for iterative calculations in the case of opposed turns whose saturation flows (or lost times) depend on signal timings has been pointed out in Section 5.6. A complicated case occurs when a turning movement has two saturation flows during one signal cycle (a normal saturation flow as an unopposed movement during one phase and a reduced saturation flow as an opposed movement during another phase). The capacity and timing analysis method to be used in this case is described in Appendix F.2.

The saturation flow has been formulated as a function of signal timings when there are short lane effects. Predictions using known signal timings are relatively simple, but signal design calculations to achieve satisfactory operating conditions may be complicated. Fixed green time solutions which can be used in the case of a short lane and other cases of saturation flow falling off during the green period are explained in Appendix F.3.

In practice, the design objective should be to provide sufficient lane lengths for queueing, e.g. by constructing longer lanes, or by banning parking wherever possible. When inevitable, the use of short green times and cycle times to reduce or eliminate short lane effects should be adopted as a policy. A possibility is to use a repeat phase system (two separate green periods of length $g/2$, rather than a single green period of length g during the signal cycle). Similarly, the causes of lane under-utilisation should be prevented through careful design as far as possible.

6. MEASURES OF PERFORMANCE

The operational efficiency of a signal-controlled intersection is expressed in terms of various measures of performance (operating characteristics). The choice of an appropriate measure of performance is fundamental to signal timing calculation methods (Section 7).

Delay and *number of stops* are the two basic measures of performance from which other (secondary) measures of performance can be derived, e.g. fuel consumption, pollutant emissions, cost (vehicle operating cost plus the value of person time). Delay for a vehicle is defined as the difference between uninterrupted and interrupted travel times through the intersection. This includes the deceleration and acceleration delays and the delay while the vehicle is idling in a stationary queue. Each delayed vehicle is considered to experience a stop. A vehicle which slows down without coming to a *complete stop* is associated with a small delay, and this case will be referred to as a *partial stop* (see Appendix G). The *queue length*, i.e. the number of vehicles in queue, is also a basic measure of performance. It is of particular importance when there is limited queue storage space.

The *capacity* and *degree of saturation* are more basic measures of performance as discussed in Section 3. The degree of saturation determines the pattern of change in delay, number of stops and queue length. The intersection performance deteriorates rapidly at degrees of saturation above 0.8 to 0.9 as shown in the graphs given in this section (*Figs 6.2* to *6.4*). The degree of saturation can therefore be used as a simple indicator of signalised intersection level of service (Akcelik 1978a and *b*, 1979*a*).

The concepts used in deriving the formulae for delay, number of stops and queue length are closely interrelated (Akcelik 1980*a*; 1981). The basic concept of the *overflow queue*, which is applicable to both undersaturated and oversaturated cases, is discussed first before the formulae for delay, stops and queue length are given below. In this section, the formulae are presented for the case of isolated, fixed-time signals. The vehicle-actuated and co-ordinated signal cases are discussed in Appendices H and I.

An example is given in Section 9.6 for the calculation of measures of performance using the formulae given in this section.

6.1 OVERFLOW QUEUE

Each of the formulae given below for predicting delay, number of stops and queue length for individual movements can be considered as having a *uniform* and an *overflow* component. The uniform component is based on the assumption of regular arrivals (constant headways), and is expressed mainly in relation to the red time. The overflow component is expressed explicitly as a function of the *average overflow queue*, i.e. the average number of vehicles left in the queue at the end of the green period. Overflow queues are due to oversaturation which may last only for a few signal cycles (low to moderate degrees of saturation), or may persist for a long period of time (high degrees of saturation).

Even if the arrival flow rate is, on average, less than the capacity (i.e. the degree of saturation, $x < 1$) there are some oversaturated cycles because of the

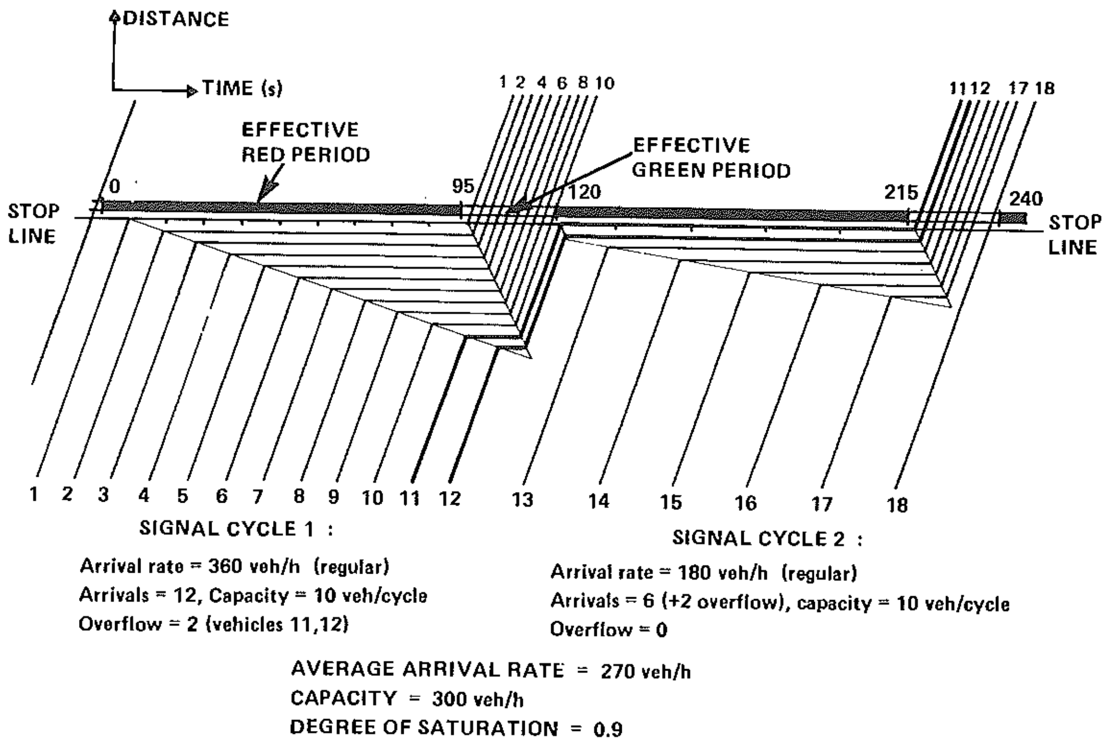


Fig. 6.1 — An example to illustrate the overflow queue concept

random fluctuations in arrival flow rates. An example is given in *Fig. 6.1* which shows the idealised time-distance trajectories of vehicles arriving during two signal cycles. The arrival rates are 360 and 180 veh/h during the first and second cycles, respectively, and they are assumed to be constant within each cycle. The saturation flow, green time and cycle time (s = 1440 veh/h, g = 25 s, c = 120 s), and hence the capacity (Q = 300 veh/h = 10 veh/cycle), are constant. The capacity of the first cycle is exceeded resulting in an overflow of two vehicles (vehicles 11 and 12). In the second cycle, these two vehicles as well as all new arrivals are cleared, i.e. there is no overflow queue. It can be seen from *Fig. 6.1* that the two vehicles left over from the first cycle cause increased delays to vehicles arriving during the second cycle (the horizontal part of a vehicle's time-distance trajectory represents the delay).

The effect of random variations in arrival flow rates is negligible for low degrees of saturation, but increases as average flow rates approach capacity resulting in larger overflow queues, and hence larger delays, numbers of stops and queue lengths. When oversaturated conditions, i.e. the arrival flows exceeding capacity ($x > 1$), persist for a long period of time, the operating characteristics become a function of the length of that period. The overflow queues grow continuously during such a period, until the average arrival flow rate drops below the capacity and the queues can be cleared during subsequent signal cycles.

The following approximate expression has been derived (Akcelik 1980*b*) for predicting average overflow queues in both undersaturated ($x < 1$) and oversaturated ($x > 1$) conditions at isolated fixed-time signals:

$$N_o = \begin{cases} \dfrac{QT_f}{4}\left(z + \sqrt{z^2 + \dfrac{12(x - x_o)}{QT_f}}\right) & \text{for } x > x_o \\ \text{zero} & \text{otherwise} \end{cases} \qquad (6.1)$$

where

N_o = average overflow queue in vehicles (where there are several lanes of vehicles, this is the total number of vehicles queued in all lanes),

Q = capacity in vehicles per hour (eqn (3.1)),

T_f = flow period, i.e. the time interval in hours, during which an average arrival (demand) flow rate, q, persists,

QT_f = throughput, i.e. the maximum number of vehicles which can be discharged during interval T_f,

x = q/Q, degree of saturation,

z = $x - 1$ (note that this has a *negative* value for degrees of saturation less than 1; also note that $zQ = q - Q$).

x_o = the degree of saturation below which the average overflow queue is approximately zero, and is given by

$$x_0 = 0.67 + sg/600 \qquad (6.2)$$

where sg is the capacity in vehicles per cycle (s = saturation flow in vehicles *per second*, g = effective green time in seconds).

The arrival flow rate, q, for calculating the degree of saturation, x, must be in vehicles per hour. However, the flow period, T_f, may have a value other than 1 h. For example, if the actual flow count during a peak period of 30 minutes (T_f = 0.5 h) is 450 vehicles, then $q = 450/0.5 = 900$ veh/h must be used (or $q = 900/3600 = 0.250$ veh/s for the delay, stop rate and queue length formulae given below).

The value of T_f does not have a significant effect on N_o for x below about 0.9. The overflow queues are due to randomness alone in this region of flow. On the other hand, the value of T_f is the main factor which determines N_o, and hence delay, number of stops and queue length, for degrees of saturation well above 1. The average oversaturation queue (given by $zQT_f/2$) is the main component of N_o in this region. For the transient region of x near 1, both the random and the oversaturation components of N_o are significant (see Akcelik 1980*b*). Similar considerations apply to other operating characteristics described below. It is recommended that a value of T_f = 1 h is used in general-purpose analyses which do not deal with oversaturated conditions during shorter intervals in specific terms.

Examples of average overflow queues at isolated fixed-time signals as a function of the degree of saturation, x, are illustrated in *Fig. 6.2* for sg = 10 and 80 vehicles (T_f = 1 h and a cycle time of c = 120 s are assumed, hence the throughput is $QT_f = 30sg$ in this example).

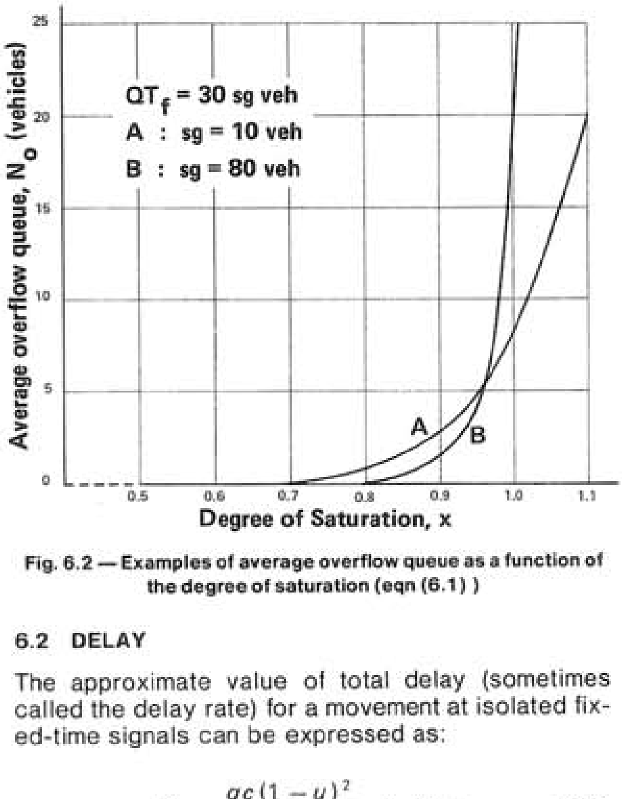


Fig. 6.2 — Examples of average overflow queue as a function of the degree of saturation (eqn (6.1))

6.2 DELAY

The approximate value of total delay (sometimes called the delay rate) for a movement at isolated fixed-time signals can be expressed as:

$$D = \frac{qc(1-u)^2}{2(1-y)} + N_o x \qquad (6.3)$$

where

D = total delay (in vehicle-hours per hour, or simply 'vehicles'),

qc = average number of arrivals in vehicles per cycle (q = flow in vehicles *per second*, c = cycle time in seconds),

u = green time ratio ($= g/c$),

y = flow ratio ($= q/s$), and

N_o = average overflow queue in vehicles given by eqn (6.1).

A general formula for the average delay per vehicle, d (in seconds) is:

$$d = D/q \qquad (6.4)$$

where D is the total delay as given by eqn (6.3) and q is the flow in vehicles *per second*. Examples of average delay at isolated fixed-time signals as a function of the degree of saturation, x, are shown in *Fig. 6.3* (based on eqns (6.1) to (6.4) with T_f = 1 h.

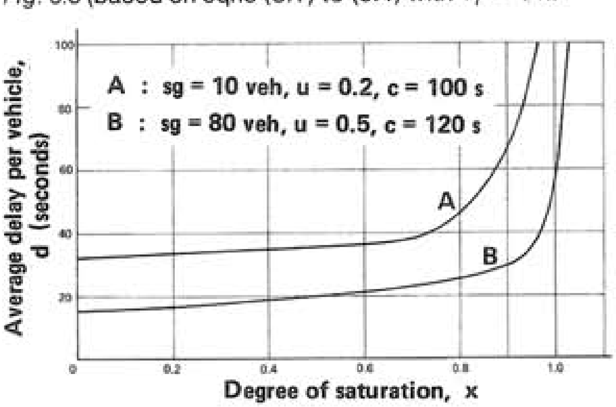


Fig. 6.3 — Examples of average delay as a function of the degree of saturation (eqns (6.3) and (6.4))

6.3 NUMBER OF STOPS

The average number of complete stops per vehicle is called the *stop rate* and denoted by h. The stop rate for a movement at isolated fixed-time signals can be calculated from:

$$h = 0.9\left(\frac{1-u}{1-y} + \frac{N_o}{qc}\right) \qquad (6.5)$$

where u, y, qc, N_o are as in eqn (6.3), and the constant 0.9 is a reduction factor to allow for partial stops, i.e. vehicles which are delayed without coming to a complete stop (see Appendix G).

It should be noted that the first term gives the proportion of stopped vehicles irrespective of how many times they are stopped (this is strictly correct for $x \leq 1$; see Akcelik 1980*b* for approximations made in deriving eqn (6.5)). The second term allows for multiple stops in oversaturated cycles using the average overflow queue as a parameter. This may give rise to stop rates greater than one whereas the maximum value of the proportion of stopped vehicles is one. *Fig. 6.1* presents an example where vehicles 11 and 12 make two stops each before they can clear the intersection. The last vehicle (vehicle 18) is unstopped. Therefore the average number of stops per vehicle for the two cycles considered is 19/18 = 1.06 whereas the proportion stopped is 17/18 = 0.94.

The effect of multiple stops becomes significant for degrees of saturation greater than about 0.8 as determined by the values of N_o (see *Fig. 6.2*). Examples of stop rate as a function of the degree of saturation calculated from eqn (6.5) for the same data in *Fig. 6.3* are shown in *Fig. 6.4*.

The number of complete stops per unit time, H, experienced by a movement with a flow rate q (vehicles per unit time) is given by the general formula:

$$H = qh \qquad (6.6)$$

where h is the stop rate as given by eqn (6.5).

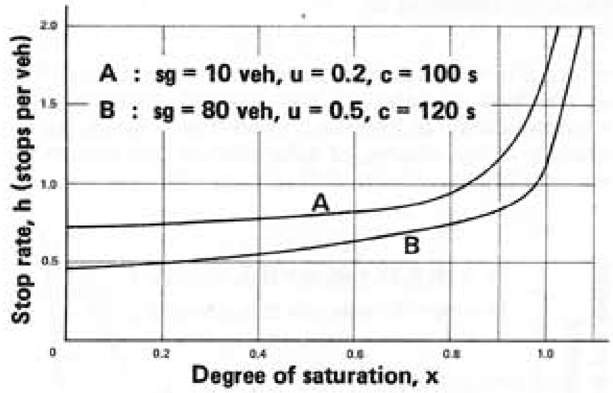


Fig. 6.4 — Examples of stop rate as a function of the degree of saturation (eqn (6.5))

6.4 QUEUE LENGTH

The *average* number of vehicles in the queue at the start of the green period (measured as idealised arrivals at the stop line) is given by:

$$N = qr + N_o \qquad (6.7)$$

where N and N_o are in vehicles, qr is the queue based on the assumption of regular arrival headways (q = arrival flow rate in vehicles *per second*, $r = c - g$ = effective red time in seconds, qr in vehicles), and N_o is the average overflow queue (eqn (6.1)).

Eqn (6.7) is based on a theoretical model which assumes that vehicles join the queue when they reach the stop line. However, because of the finite extension of the queue, vehicles join the back of the queue earlier, and hence, this formula underestimates the queue length.

The maximum back of the queue, N_m, is reached some time after the start of the green period. This queue is given by:

$$N_m = \frac{qr}{1 - y} + N_o \qquad (6.8)$$

where qr and N_o are as in eqn (6.7), and y is the flow ratio.

The maximum back of the queue N_m, corresponds to the physical end of the queue as perceived by the drivers. While the back of the queue increases to N_m, the front of the queue will be moving forward. At the time when N_m is reached, all vehicles will be moving, some beyond the stop line, i.e. will have already cleared the intersection. For example, in *Fig. 6.1*, the maximum back of the queue is 12 in signal cycle 1 and 7 in signal cycle 2. For this example, an average value of $N_m = 11.5$ is estimated from eqn (6.8) using a flow period of $T_f = 1$ hour (q = 270 veh/h, s = 1440 veh/h, r = 95 s, g = 25 s, c = 120 s, y = 0.1875, x = 0.90).

The maximum back of the queue, N_m, corresponds to the average number of vehicles stopped per cycle. Therefore, the stop rate given by eqn (6.5) can be obtained from N_m by applying the correction factor of 0.9:

$$h = 0.9\,\frac{N_m}{qc} = 0.9\left(\frac{qr}{qc(1-y)} + \frac{N_o}{qc}\right)$$

$$= 0.9\left(\frac{1-u}{1-y} + \frac{N_o}{qc}\right) \qquad (6.9)$$

where u, y, N_o, qc are as in eqn (6.5).

Equations (6.7) and (6.8) relate to the average queue length. However, it may be desirable to design the queue storage distances (e.g. the required length of a turning lane) on the basis of a critical queue length, N_c, which will be exceeded only in a negligible number of signal cycles. As a simple method which applies to both undersaturated and oversaturated cases, the critical queue length can be calculated from

$$N_c = 2\,N_m \qquad (6.10)$$

where N_m is given by eqn (6.8). The required storage distance can then be calculated as (jN_c/n), where j is the average queue space per vehicle (e.g. 7.2 m) and n is the number of lanes.

Better prediction accuracies will be obtained if the performance formulae given in this chapter are applied on a lane-by-lane basis.

6.5 PEDESTRIANS

The formulae to calculate pedestrian delays, stops and queues have been derived from the formulae given earlier by assuming zero flow ratios and zero overflow queues for pedestrian movements ($y = 0$, $N_o = 0$). This is justified on the basis of very high pedestrian saturation flows (hence, low y and x, and high sg values).

The average delay in seconds per pedestrian:

$$d = r^2/2c \qquad (6.11)$$

where

r = effective red time (including the flashing don't walk period) in seconds, and

c = cycle time in seconds.

Eqn (6.11) indicates that the pedestrian delays are only due to the red time effect and smaller cycle times will produce smaller delays to pedestrians. The longest delay a pedestrian will experience during a signal cycle is r seconds.

The number of pedestrians stopped at traffic signals:

$$H = qr/c \qquad (6.12)$$

where

q = pedestrian arrival rate (pedestrians per second or pedestrians per hour),

r, c = as in eqn (6.11), and

H = in the same units as q.

The number of pedestrians in queue at the start of green (walk) period:

$$N = qr \qquad (6.13)$$

where q is in pedestrians per second, r is in seconds, and N is the number of pedestrians.

Eqns (6.12) and (6.13) indicate that no pedestrians are stopped during the green (walk) period, i.e. they assume that the pedestrian queues are discharged instantly. They also imply that pedestrian queues are always cleared (no overflow).

6.6 SECONDARY MEASURES OF PERFORMANCE

Total delay and number of stops calculated from the above formulae can be used for calculating other statistics such as fuel consumption, pollutant emissions and cost. The general formula is:

$$E = f_1 C + f_2 D_s + f_3 H \qquad (6.14)$$

where

E	=	the statistic of interest, e.g. fuel consumption in L/h,
C	=	total amount of travel (= cruising distance x flow) in veh-km/h (distance in km, flow in veh/h,
D_s	=	total stopped delay (idling) time in veh-h/h,
H	=	total number of complete stops per hour,
f_1, f_2, f_3	=	factors (average rates) for cruise, delay and stops, respectively

For example, for fuel consumption, f_1 is the fuel consumption rate in L/veh-km while cruising, f_2 is the fuel consumption rate in L/veh-h while idling, and f_3 is the excess fuel consumption rate in litres per complete stop (this is associated with deceleration-acceleration per stop, and is found by subtracting the consumption when deceleration-acceleration distance is travelled at the cruising speed from the consumption during a complete stop-and-go cycle with no idling).

Equation (6.14) can be re-written as:

$$E = f_1 C + f_2 D + f_3' H \qquad (6.15)$$

where

D	=	total *stop-line* delay in veh-h/h,
f_3'	=	$f_3 - (f_2\, d_h/3600)$ = adjusted excess fuel consumption rate per complete stop, and
d_h	=	acceleration-deceleration delay for a complete stop-and-go cycle in seconds (e.g. 12 s).

Eqn (6.15) allows for the difference between the stopped delay (idling) time, D_s, and the model (stop-line) delay, D, which includes the deceleration-acceleration delays of stopped vehicles as explained in Appendix G.

Eqns (6.14) and (6.15) represent an *elemental model* of fuel consumption, cost, etc., which can be used for any traffic control facility by employing measured or simulated (computed) values of the amount of travel, delay and number of stops for that facility (for a detailed discussion, see Akcelik 1981).

For fuel consumption, typical values of the parameters of elemental model are f_1 = 0.08 – 0.12 L/veh-km, f_2 = 1.50 – 2.40 L/veh-h and f_3' = 0.01 – 0.04 L/stop. These values are presented here to give a rough idea, and should not be used without considering the basis of their derivation carefully. In particular, it should be noted that the parameters f_1 and f_3 (or f_3') of the elemental model are dependent on the cruising speed, and all rates are dependent on the composition (vehicle mix) of the particular movement under consideration (see Akcelik 1981).

6.7 STOP PENALTY

The relative values of the rates for idling and stops, f_2 and f_3' in the elemental model, are related to the *stop penalty* concept (Huddart 1969; Robertson, Lucas and Baker 1980). Using the rates f_2, f_3' in the same units as above, the stop penalty is given by:

$$K = 3600 \frac{f_3'}{f_2} \qquad (6.16)$$

Assuming the cruise component (the amount of travel) is unaffected by signal timings, a measure of performance defined as $(D + KH)$ to determine optimum signal timings is equivalent to a measure expressed by eqn (6.15). Therefore, this performance measure can be used to compute signal settings which minimise fuel consumption, cost, pollutant emissions, etc. (see Section 7). Typical stop penalty values which correspond to the fuel consumption rates mentioned above are in the range 20 to 60. As a rough guide, the stop penalty for cost (vehicle operating cost including fuel and oil consumption, tyre wear, etc. plus the value of person time) can be calculated as half the value of the corresponding stop penalty for fuel consumption. The typical stop penalty values for cost are therefore in the range from 10 to 30.

6.8 DISCUSSION

The following should be noted regarding the use of the formulae given in this section.

(a) It must be emphasised that the formulae given in this section are only *approximate* expressions based on various simplifying assumptions. The consideration of small prediction errors (in the sensitive and uncertain region of near-capacity conditions, in particular) is unlikely to be very productive as the accuracy of field measurements are also bound to be limited (e.g. see

Reilly, Gardner and Kell 1976; Richardson 1979; Sagi and Campbell 1969). The expressions given in this section are considered satisfactory mainly because they reflect the basic mechanism of signalised intersection operations. However, the approximate nature of these formulae must always be kept in mind when translating the results to signal design and control practice.

(b) In using some of the formulae, care must be taken not to mix units, i.e. flows and saturation flows (q, s) in hourly units and signal timings (c, g) in seconds.

(c) The formulae for total delay, stop rate and queue length (eqns (6.3), (6.5) and (6.7)) are not directly applicable to the case where a movement receives *two green periods per cycle* (e.g. a turning movement which receives opposed and unopposed periods as discussed in Appendix F.2). However, these formulae can be used by modifying the uniform (first) term to allow for this case (see Example 7 in Appendix D).

(d) The formulae given in this section are for predicting the performance of individual vehicle and pedestrian *movements*. The overall *intersection* performance can be expressed as the sum of all individual movement operating characteristics. Total delay, D, and total number of stops, H, provide meaningful statistics with this respect. Although the queue length is meaningful on an individual movement basis, summation for the intersection is a useful indicator of the relative effects of alternative control policies. The secondary measures of performance can also be calculated as a sum of the individual movement values. The intersection delay can be calculated in terms of person-delay by multiplying the total delay value of individual movements by appropriate occupancy values before they are added up (e.g. 1.3 for cars, 30 for buses, 1.0 for pedestrians, etc.).

For further reading on signal delay and queue length models, see the list given at the end of Appendix G.

7. SIGNAL TIMINGS

The calculation of signal timings (namely cycle time, green times, and offsets for co-ordinated signals) which yield satisfactory operating conditions is determined by the selection of a performance function to represent intersection operating conditions (see Section 6). The traditional method of calculating signal timings for an isolated intersection uses the vehicle delay as a measure of performance (Webster 1958; Webster and Cobbe 1966; Miller 1963, 1964 and 1968*b*). However, vehicle stops, queue length, and person (pedestrian, bus passenger, etc.) delays and stops should also be considered when determining signal timings (cycle time, in particular).

Vehicle stops are important when factors such as vehicle operating costs (fuel consumption, wear and tear), air pollution, annoyance to drivers and safety are considered. In particular, the contribution of vehicle stops to total fuel consumption and pollutant emissions is significant (Akcelik 1981). Similarly, a large proportion of accidents at traffic signals could be attributable to the need to stop vehicles (Huddart 1969).

A performance function which combines delays and stops through the use of a *stop penalty* can be employed for computing signal timings which minimise fuel consumption, cost, etc. by choosing an appropriate value of the stop penalty as described in Section 6.7. Eqn (7.1) allows for the calculation of an *approximate optimum* cycle time for isolated intersections using the stop penalty as a parameter. More exact solutions can be obtained using a computer program such as SIDRA (Akcelik 1979*b*).

A different approach is to calculate a *practical* cycle time, i.e. a minimum cycle time required to achieve various maximum acceptable degree of saturation constraints (see Section 3.3). A practical cycle time formula is also given in this section.

A prerequisite for the calculation of signal timings is to determine which movements are critical. A critical movement identification method is described in detail in Section 4. Once the critical movements are known, the intersection lost time, flow ratio and green time ratio (L, Y, U) can be calculated as the sum of the corresponding critical movement parameters (ℓ, y, u). These are the parameters to be used in the cycle time and green time calculations. The critical movement identification method described in Section 4 makes allowance for the minimum green time constraints determined by vehicle or pedestrian requirements. When using the cycle time and green time formulae given in this section, it is important to note that the minimum green time value which exceeds the normal green time requirement of any critical movement is included in the intersection lost time, L, and that the flow and green time ratios (y, u) for such movements are not included in the respective intersection parameters (Y, U).

The cycle time and green times determined using the methods given in this section are of direct use for fixed-time signal controllers (isolated or co-ordinated) and can be used for choosing vehicle-actuated controller settings as discussed in Appendix H. Additional calculations required for co-ordinated signals are described in Appendix I.

7.1 CYCLE TIME

7.1.1 Approximate Optimum Cycle Time

The cycle time which *approximately* minimises a performance measure defined as $P = D + KH$, where K is the stop penalty, and D and H are the total delay and number of stops, respectively, for all *critical* movements at an isolated intersection, (excluding any movement whose minimum green time exceeds the normal green time requirement), can be calculated from the following formula developed by the author:

$$c_o = \frac{(1.4 + k)L + 6}{1 - Y} \tag{7.1}$$

where

c_o = approximate optimum cycle time in seconds,

L = intersection lost time in seconds (see eqn (2.7)),

Y = intersection flow ratio (see eqn (3.9)), and

k = $K/100$ is the stop penalty parameter.

The typical stop penalty values, K, are given in Section 6.7. The following values of the stop penalty parameter can be used in eqn (7.1) to calculate various cycle time values:

$k = 0.4$ for minimum fuel consumption,

$k = 0.2$ for minimum cost (including the value of delay time), and

$k = 0$ for minimum delay.

Furthermore, eqn (7.1) can be used to calculate a cycle time which approximately minimises the sum of the critical movement queue lengths (excluding any movement with a minimum green time exceeding the green time required normally). For this purpose, use $k = -0.3$. Note that this solution corresponds to reducing the maximum stationary queue length in an average cycle (N given by eqn (6.7)).

It should be noted that eqn (7.1) is an *average* expression which does not make an explicit allowance for such factors as the minimum saturation flow for any critical movement or the relative values of the critical movement flow ratios. Therefore, the cycle time obtained from eqn (7.1) may be different from the true (theoretical) optimum value in a specific case. On the other hand, the performance function ($P = D + KH$) has a flat response near the optimum cycle time, i.e. it does not change significantly with the use of a slightly shorter or longer cycle time.

An important aspect of eqn (7.1) is that it represents a multiplicity of control/design objectives through the use of a stop penalty parameter. In practice, the adoption of a particular strategy depends on the conditions of a particular site during a particular time period. For example, at an intersection of high speed roads during off-peak (low flow) conditions, the minimum fuel consumption strategy with a large weight on stops is reasonable. On the other hand, at a city centre intersection during peak (heavy flow) conditions, a minimum queue strategy would be reasonable, in particular if queue storage spaces are limited.

It should be mentioned that with a value of $k = 0$, eqn (7.1) gives cycle times which are very close to the approximate minimum-delay solutions given by the Webster (1958) and Miller (1968*b*) formulae.

It is seen from eqn (7.1) that a higher value of the stop penalty parameter, k, will give a longer cycle time. This has the advantage of reducing the number of stops, and hence fuel consumption, pollutant emissions, etc. However, there are reasons for using a slightly shorter cycle time:

(a) Eqn (7.1) gives an approximate optimum cycle time for only the critical movements which contribute to the Y value. If the delays and stops to other movements are also included in the performance function, a smaller cycle time will be obtained. In particular, a smaller cycle time than that given by eqn (7.1) will decrease delays to minor movements, including pedestrians.

(b) Eqn (7.1) is based on the assumption that the saturation flow rate is retained throughout the green period. On the other hand, the saturation flow may fall off during the green period as in the case of a short lane (see Section 5). Shorter cycle times give better performance in such cases.

(c) In the case of co-ordinated signals, a common cycle time to be used for all intersections in the area is determined by the cycle time of the critical intersection. Because the other intersections require smaller cycle times, the performance of traffic in the area as a whole can be improved by using a smaller cycle time, at the expense of some loss in the performance of the critical intersection.

In conclusion, the traffic engineer is faced with the choice of a cycle time considering a multiplicity of factors relevant to a particular case. To aid this choice, a practical cycle time which indicates a minimum level of desirable performance can be calculated as described below.

7.1.2 Practical Cycle Time

The minimum cycle time which ensures that the degrees of saturation of all movements are below specified *maximum acceptable degrees of saturation*, $x < x_p$ (see Sections 3.3 and 4.1), is called the practical cycle time, and is given by

$$c_p = L/(1 - U) \tag{7.2}$$

where

L = intersection lost time in seconds (see eqn (2.7)), and

U = intersection green time ratio (see eqn (3.8)).

As discussed in Section 4 (eqns (4.2) to (4.6)), if the required time for a critical movement is determined by the pedestrian clearance or a similar vehicle minimum green time requirement ($t_m = G_m + I$) rather than the maximum acceptable degree of saturation ($uc + \ell$, where $u = y/x_p$), then L should include t_m and U should exclude u. The procedure given in Section 4 ensures that L and U (also Y for eqn (7.1)) are calculated correctly in such cases. Under very low flow conditions, this may be true for all movements, in which case U is zero and L is the sum of the t_m values of all critical movements, and eqn (7.2) is reduced to the absolute minimum cycle time formula.

$$c_m = \Sigma\, t_m = \Sigma(G_m + I) \tag{7.3}$$

where the summation is for all critical movements (G_m = displayed minimum green time, I = intergreen time).

Various suggestions for selecting the maximum acceptable degrees of saturation (x_p) are given in Section 3.3. The use of $x_p = 0.9$ is recommended as a general-purpose value. Different x_p values can be chosen for different movements as a measure of priority and restraint. For example, it is possible to choose a lower x_p value for the main road traffic and a higher value for the side road traffic in a co-ordinated signal system for better progression on the main road. Similarly, unequal degrees of saturation can be used for bus priority, or entry (gating) control purposes.

It should be noted that, the larger the values of x_p chosen, the smaller the value of cycle time obtained from eqn (7.2) because this means that the capacity conditions are approached closer. In fact, eqn (7.2), is also applicable for oversaturated flow conditions, in which case the use of $x_p > 1$ is unavoidable.

7.1.3 Choice of a Cycle Time

A cycle time calculated using either eqn (7.1) or eqn (7.2) can be used in practice. However, it is recommended that both cycle times are calculated and a suitable cycle time, c, between c_p and c_o is chosen for use. A further constraint is a maximum acceptable cycle time, c_{max} (e.g. 120 s), hence the chosen cycle time should ideally satisfy the condition $c \leq c_{max}$.

The approximate optimum cycle time formula (eqn (7.1)) is not valid for co-ordinated signals. In this case, a practical cycle time can be calculated using eqn (7.2) for each intersection in the co-ordinated area, the largest of which can be used as a basis of choosing a common area cycle time (see Appendix I).

7.2 GREEN TIMES

The calculation of green times for a chosen cycle time can be carried out in the following steps:

(a) calculate the critical movement green times,

(b) calculate the non-critical movement green times, and

(c) determine the phase green times.

A detailed description of each step follows. It is recommended that the calculations are carried out using a critical movement search diagram (see Sections 2 and 4), which helps to visualise the relationship between the movements and phases, and between the critical and non-critical movements, in particular when there are overlap movements.

The method will be explained with the aid of examples in *Fig. 7.1* (which is the example illustrated in *Figs 2.1* to *2.5* in Section 2) and *Fig. 7.2* (which presents a multiple overlap case). Also see Appendix C for the calculation of green times for the example given in Section 4.

The following general movement-phase time relationship is the basis of various formulae given below for the calculation of green times:

$$\Sigma\,(g + \ell) = \sum_{i}^{k-1} (G + I) \qquad (7.4)$$

where $\Sigma\,(g + \ell$ is the total time for several movements receiving right of way during phases i to $k-1$ consecutively, (g, ℓ) are movement green and lost times, and (G, I) are phase green and intergreen times (i = starting phase, k = terminating phase). For example, in *Fig. 7.1*:

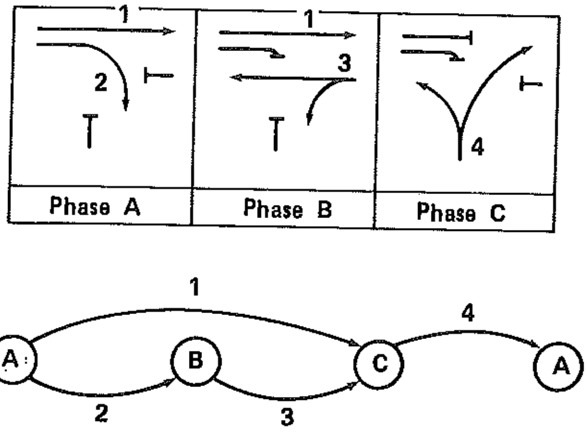


Fig. 7.1 — A simple overlap movement case

$$g_1 + \ell_1 = G_A + I_A + G_B + I_B = g_2 + \ell_2 + g_3 + \ell_3$$

and $g_4 + \ell_4 = G_C + I_C$

Similarly, in *Fig. 7.2*:

$$g_1 + \ell_1 + g_3 + \ell_3 = G_A + I_A + G_B + I_B + G_C + I_C$$

$$= g_2 + \ell_2 + g_4 + \ell_4$$

and $g_5 + \ell_5 = G_D + I_D = g_6 + \ell_6$

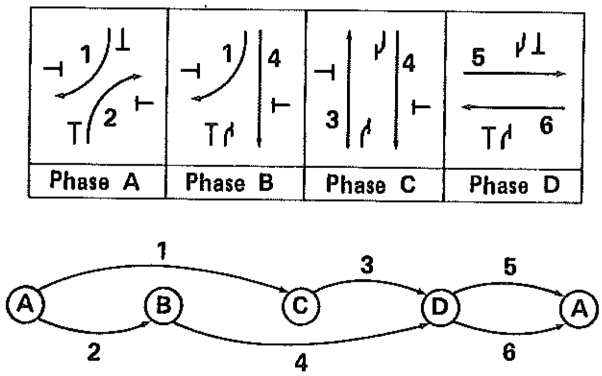


Fig. 7.2 — A multiple overlap movement case

7.2.1 Critical Movement Green Times

For a given cycle time, c, the total available (effective) green time is $(c - L)$, where L is the sum of critical movement lost times. The total available green time can be distributed to the critical movements according to the formula:

$$g = \left(\frac{c - L}{U}\right) u \qquad (7.5)$$

where u and U are the movement and intersection green time ratios, respectively, calculated as part of the critical movement identification procedure described in Section 4.

This formula is applicable to both cases of equal and unequal degrees of saturation. For the case of equal degrees of saturation, the use of (y, Y) instead of (u, U) will give the same result, which is the traditional method as described by Webster (1958) and Miller (1968*b*).

The formula does not apply to a movement whose required time is determined by the minimum green time, g_m, which is included in L. In this case:

$$g = g_m \quad (G = G_m) \qquad (7.6)$$

7.2.2 Non-critical Movement Green Times

In simple cases where there are no overlap movements, the calculation of critical movement green times is sufficient to determine all other (non-critical) movement green times as well as all phase green times. In this case, the effective green time of a non-critical movement, which receives right of way in the same phase as a critical movement whose green and lost times are g_c and ℓ_c, is given by:

$$g = (g_c + \ell_c) - \ell \qquad (7.7)$$

where ℓ is the lost time for the movement in question.

For example, in *Fig. 7.2*, assuming that movement 5 is the critical one in phase D, the effective green time for movement 6 can be calculated as $g_6 = (g_5 + \ell_5) - \ell_6$. Eqn (7.7) also applies to overlap movements which have the same starting and terminating phase numbers.

In a single overlap movement case (as in *Fig. 7.1*) where the overlap movement is critical, the green times for non-overlap movements can be calculated by treating the critical movement time as a *sub-cycle time*, $c^* = g_c + \ell_c$, where g_c and ℓ_c are the critical movement green and lost times. Then the total available green time is $(c^* - L^*)$, where L^* is the sum of non-overlap movement lost times. This time can be distributed to the non-overlap movements in the same way as in eqns (7.5) and (7.6), i.e.

$$g = \left(\frac{c^* - L^*}{U^*}\right) u \qquad (7.8)$$

or,

$$g = g_m \qquad (7.9)$$

whichever is applicable (U^* is the sum of non-overlap movement green time ratios).

As for eqn (7.5), if any movement in the sub-cycle being considered has a minimum time value, t_m, which exceeds the normal time requirement, the total lost-time, L^*, should include t_m, and U^* should exclude the green time ratio, u, of this movement. The procedure described in Section 4.3 makes allowance for this case also. The green time for such movements are given by eqn (7.9).

For example, in *Fig. 7.1*, if movement 1 is critical (in phases A and B), then

$$c^* = g_1 + \ell_1 \,, \quad L^* = \ell_2 + \ell_3 \,, \quad U^* = u_2 + u_3$$

are normally used to calculate g_2 and g_3. However, suppose the required time for movement 3 is determined by the minimum time, $t_{m3} = \ell_3 + g_{m3}$. In this case,

$$L^* = \ell_2 + t_{m3} \,, \quad U^* = u_2$$

and the green times are

$$g_2 = c^* - L^* \text{ and } g_3 = g_{m3}$$

A more general case is when there are multiple overlaps as shown in *Fig. 7.2*. Eqns (7.8) and (7.9) can be used in this case also by calculating the sub-cycle time as

$$c^* = \Sigma g_c + \Sigma \ell_c \,,$$

i.e. the total time for all critical movements. For example, in *Fig. 7.2*, if movements 1 and 3 are critical (in phases A, B and C),

$$c^* = g_1 + g_3 + \ell_1 + \ell_3 \,, \quad L^* = \ell_2 + \ell_4 \,,$$

$U^* = u_2 + u_4$ are used to calculate g_2 and g_4.

There may be a case when all non-critical movement green times are subject to $g = g_m$. Then there will be an excess time which can be allocated to these movements in any way desired.

In the case when the non-overlap movements are critical, the non-critical overlap movement green time can be calculated from:

$$g = (\Sigma g_c + \Sigma \ell_c) - \ell \qquad (7.10)$$

where g_c and ℓ_c are the critical (non-overlap) movement green and lost times, and ℓ is the overlap movement lost time. For example, in *Fig. 7.1*, if movements 2 and 3 are critical, the overlap movement 1 green time is

$$g_1 = (g_2 + \ell_2 + g_3 + \ell_3) - \ell_1$$

7.2.3 Phase Green Times

The displayed green time for a phase can be calculated from

$$G = (g + \ell) - I \qquad (7.11)$$

where $(g + \ell)$ is the time allocated to a movement which receives right of way during that phase only, and I is the intergreen time of that phase.

For example, in *Fig. 7.1*, the green time for phase A is given by $G_A = (g_2 + \ell_2) - I_A$.

In multiple overlap cases, there may be a phase with no single (non-overlap) movement. In this case, the phase green time can be calculated using eqn (7.4). For example, Phase B in *Fig. 7.2* is such a phase, and its time can be calculated as

$$G_B = (g_1 + \ell_1) + (g_3 + \ell_3) - (G_A + I_A) - (G_C + I_C) - I_B$$

7.3 PHASE CHANGE TIMES

Once all phase green times are known, phase change times in a signal cycle can be calculated from

$$F_i = F_{i-1} + (I + G)_{i-1} \qquad (7.12)$$

where

F_i	=	change time for i th phase,
F_{i-1}	=	change time for the previous phase, and
$(I + G)_{i-1}$	=	sum of intergreen and green times of the previous phase.

As explained in Section 2, phase change times can be calculated by setting the time for the first phase, $F_A = 0$, and applying eqn (7.12) successively. As a final check, the first phase change time can be calculated from the last phase change time, and this must be equal to the cycle time, e.g. in *Fig. 7.1*, $F_A = F_C + I_C + G_C$ (see *Figs 2.2* and *2.4* in Section 2 for this example).

The phase change times are of particular use in the prepartion of signal plans for co-ordinated systems because they describe signal offsets as well as green times in this case. The phase change times calculated using eqn (7.12) can be modified by adding an *offset* value to allow for progression (see Appendix I).

It is also useful to note that the effective green time for a movement can be calculated directly from known phase change times. As explained in Section 2, this is given by:

$$g = F_k - F_i - \ell \qquad (7.13)$$

where F_k and F_i are the change times for the starting and terminating phases, and ℓ is the movement lost time. Similarly, the displayed green time for a movement is given by:

$$G = F_k - F_i - I \qquad (7.14)$$

where I is the intergreen time for the movement (intergreen time of the starting phase). If the first phase change time is not zero, as might be in the case of co-ordinated signals, F_i may be greater than F_k. In this case, (F_k + c) must be used instead of F_k in the above equations.

8. DESIGN CONSIDERATIONS

8.1 GENERAL

Two important aspects of traffic signal design are:

(a) to *predict* traffic performance under a *given* set of conditions related to intersection geometry, road environment, lane arrangements, signal phasing and timings, and the level and composition of arrival (demand) flows; and

(b) to plan for the best possible (*optimum*) traffic performance by studying *changes* in the conditions mentioned above which can be achieved through various design actions.

A method which allows prediction but not optimisation is not satisfactory in a design context. The methods presented in this report have been developed with this point in mind.

However, traffic signal design must be considered in a wider traffic system management context, and this may lead to the adoption of non-optimal solutions in terms of traffic performance. A traffic system management approach involves the consideration of a number of often-conflicting objectives such as safety, operational efficiency, fuel consumption, air pollution, public transport priority, property access, and the consideration of the problems of road user groups other than vehicles, namely pedestrians and cyclists, and the problem of traffic intrusion into residential areas (see Akcelik 1981). The design objectives and constraints for a particular site (a single intersection, an arterial route or an area) should be clearly defined with due consideration to these problems. The techniques given in this report are useful in this context. For example, the practical cycle time calculation method allows for various priority and restraint objectives to be achieved by specifying different degrees of saturation (congestion levels) for different movements. The optimum cycle time calculation method involves a decision about the performance function to be used. More sophisticated solutions can be obtained using computer programs such as SIDRA and TRANSYT/6N (Akcelik 1979*b*) which allow for different weightings for different movements and for different road user groups.

Furthermore, the level of detail and accuracy of the method to be used for analysis must be considered in relation to the design objective. For planning and preliminary design purposes (e.g. to test a signalisation option), simple and easy to use methods and relatively low levels of accuracy may be acceptable. For detailed geometric design purposes (e.g. to decide the number and type of lanes), more detailed methods and higher levels of accuracy are needed. Finally, for the purposes of operational design (e.g. to determine the best signal phasing and timings), the highest levels of detail and accuracy are required, limited only with the accuracy of the input traffic flow information.

The methods given in this report are more relevant to geometric and operational design than to preliminary design. However, suggestions for simple analyses have been included wherever possible. For example, the designer may:

(a) assume that all lost times are equal to intergreen times and ignore differences between the displayed and effective green times;

(b) use a general average value of e_o = 3 for estimating opposed turn saturation flows and avoid iterative calculations;

(c) ignore non-critical movements (e.g. traffic in under-utilised lanes);

(d) use the intersection degree of saturation as a measure of performance and avoid detailed calculations of green times, delays, number of stops, etc.; and

(e) use the practical cycle time formula only.

The decision about the method to be used must naturally be made by the traffic engineer who is responsible for the final design recommendations. Similarly, the responsibility for collecting traffic flow information for input to the design process lies with the traffic engineer. The use of measured saturation flows, rather than relying on estimated saturation flows based on general averages, is a particularly important decision in this respect. A related problem is the selection of the design period which is discussed below.

8.2 DESIGN PERIOD

Traffic signal design calculations should normally (and conveniently) be based on the average weekday peak hour traffic flows. However, a shorter peak period (e.g. peak half hour) can be chosen for operational (phasing and timing) design purposes. In general terms, the choice of the design period depends on the peaking characteristics of arrival (demand) flows (e.g. shorter peak periods in smaller cities), the type of control (isolated vehicle-actuated, or fixed-time co-ordinated, etc.), as well as general planning and design policies (the level and duration of tolerable congestion in relation to the function of the road).

It is important that separate calculations are carried out for morning and evening 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). At some sites the most congested period may occur outside the weekday morning and evening peak periods, e.g. at intersections in shopping areas, or on recreational routes. The off-peak congestion problem may be because the capacity is reduced by allowing parking or right turns which are banned during peak periods. The adequacy of design should also be checked for such periods.

Signal design calculations should be based on actual flow counts, measured saturation flows and lost times wherever possible. In the case of new signalisation, or intersection improvements through minor modifications, signal design calculations should be carried out for flows expected at the time of opening, and designs should be checked for flows expected in the near future.

The intersection flow pattern and lane utilisation characteristics are likely to change in time due to gradual changes in trip characteristics, or due to changes in the surrounding network conditions. New signalisation and improvements to existing signals may themselves affect the flow conditions (e.g. traffic attraction may result from the introduction of an unopposed right-turn phase). It is therefore strongly recommended that the intersection conditions are re-examined and necessary steps are taken to rectify any problems:

(a) within one month after a new signal installation is opened;

(b) within three months when minor modifications to existing signals are implemented; and

(c) at least once a year for existing signals (more frequently for co-ordinated signals, say at least every six months).

The design calculations for a completely new intersection on a new road should also be based on peak period flows expected at the time of opening (as predicted by a traffic assignment model) with allowance for flow conditions expected in the future. Because of limitations in flow predictions, calculations at a preliminary design level would be sufficient. However, detailed calculations should be carried out using the flow data collected at the intersection after it is opened to traffic.

It is useful to calculate spare capacities, i.e. the amount of flow increases which can be accommodated before the capacity is reached if the existing intersection conditions (intersection geometry and environment, lane arrangements, signal phasing, traffic composition, lane utilisation, etc.) were to remain unchanged. The following formula can be used to calculate a *percentage spare capacity* (*PSC*):

$$PSC = \left(\frac{U_{max}}{U} - 1\right)100 \qquad (8.1)$$

where

U_{max} = $(c_{max} - L)/c_{max}$ is the maximum possible intersection green time ratio (c_{max} = a specified maximum permissible cycle time, L = intersection lost time), and

U = intersection green time ratio required to achieve specified maximum acceptable (practical) movement degrees of saturation (x_p).

The values of L and U must be as determined using the critical movement identification technique described in Section 4 (calculated as sums of the relevant critical movement values with due allowance for minimum green time constraints). Eqn (8.1) allows for a control strategy whereby different degrees of saturation are allocated to different movements. For the example in Section 4, L = 12 s, U = 0.85, and using c_{max} = 120 s, U_{max} = (120 − 12)/120 = 0.90 is found. Therefore, eqn (8.1) gives PSC = (0.90/0.85 − 1)100 = 6 per cent. In the case of equal degrees of saturation (the same X_p for all movements), $U = Y/X_p$ where Y is the intersection flow ratio. To derive a general formula for this case X_p = 0.95 (see Section 3.3) and c_{max} = 120 s can be used in eqn (8.1). The resulting formula is $PSC = (95 - 100Y - 0.8L)/Y$. For example, Y = 0.70, L = 20 s gives PSC = (95 − 70 − 0.8 x 20)/0.70 = 13 per cent. A negative PSC value shows that the maximum acceptable degrees of saturation are exceeded even with the present flow levels.

It should be expected that neither the flow conditions nor the intersection conditions (both physical and control) will remain the same in the long term, say beyond five years. This should be kept in mind when using eqn (8.1).

8.3 DESIGN PROCEDURE

A recommended design procedure which brings together various methods given in the other sections of this report is described below. The purpose is to provide guidance regarding the steps to follow in signal design calculations rather than define a strict procedure.

(a) Choose the design period (Section 8.2). Obtain the intersection flow counts for this period. Convert flows to hourly units if a design period other than an hour is used.

(b) Take a trial (or the existing) intersection layout (geometry and lane arrangements) and signal phasing (Section 2 and Appendix A). Establish the cases of, and data for, opposed turns (Section 5.6), lane under-utilisation (Section 5.7), short lanes (Section 5.8) and any special capacity problems.

(c) Describe movements from each approach road according to the turning manoeuvres, lane arrangements, lane under-utilisation, if any, and the signal phasing (Sections 2 and 5.1).

(d) Determine intergreen and minimum green times (Section 2, Appendix B).

(e) Choose the practical (maximum acceptable) degree of saturation, x_p, for each movement, according to a priority and restraint strategy if unequal degrees of saturation are to be used. If the equal degree of saturation method is to be used, choose the same X_p for all movements (Section 3.3).

(f) Measure or estimate the movement saturation flows and lost times (Sections 2 and 5, Appendices E and F). If opposed turns are allowed, calculate initial estimates of saturation flow assuming unopposed turns. If there are short lanes, calculate initial estimates of saturation flow assuming full lane length, i.e. ignoring any short lane effects. Always use flows and saturation flows in vehicle units (not tcu's).

(g) Identify the critical movements and calculate signal timings using the procedure described in detail in Section 4. The individual movement flow ratios (y) and required green time ratios ($u = y/x_p$) are calculated as part of this procedure. The intersection lost time, flow ratio and green time ratio (L, Y, U) are calculated as the sum of the corresponding critical movement values (ℓ, y, u) with due allowance for minimum green time constraints. Practical and optimum cycle times (c_p, c_o) are calculated. The latter involves a decision about the design/control strategy, i.e. minimum delay, minimum fuel consumption, etc. (Section 7). A cycle time, c, is chosen between c_p and c_o, also to satisfy a maximum cycle time constraint of $c \leq c_{max}$ (normally 120-150 s). For co-ordinated signals, the intersection with the largest c_p value is the critical one. The cycle time chosen for this intersection is used for all intersections in the area (Appendix I). In all cases, it is sufficient to choose a cycle time rounded up to the nearest multiple of 5 or 10 s. For the cycle time chosen, movement and phase green times and phase change times are calculated (Section 7). Finally, movement degrees of saturation, x, are calculated for the chosen cycle time and green times, and checked if $x \leq x_p$ is satisfied for each movement. Assuming correct calculations, $x > x_p$ is possible if $c = c_{max} < c_p$ is chosen, meaning that the desirable maximum degrees of saturation can not be achieved without increasing the cycle time beyond a practical limit ($x > 1$ indicates oversaturated conditions). For preliminary design purposes, the procedure can be simplified by using the same X_p for all movements (equal-degree-of-saturation method) and calculating an intersection degree of saturation, X, for the chosen cycle time from eqn (3.12) without the need to calculate green times.

(h) Revise the saturation flows (and lost times, where relevant) in the case of opposed turns using the cycle time and green times determined in Step (g). Repeat steps (g) and (h) until no significant changes are found in signal timings and saturation flows. A judgement is required on traffic engineer's part about what constitutes an 'insignificant change'. This should be decided according to the objective of the design work and the accuracy of the input data. For example, if estimated rather than measured saturation flows are being used, a high degree of accuracy should not be sought. The recommended method of choosing a cycle time in the range from c_p to c_o, and approximated to the nearest 10 s, helps reduce the number of iterations since the cycle time can be kept constant after one or two iterations (see Example 7 in Section 9 and Examples 6 and 7 in Appendix D). In the case of short lanes, it is necessary to establish whether the short lane effects occur (Section 5.8). A fixed green time solution can be sought to minimise the short lane effects (Appendix F.3).

(i) Repeat the calculations for both morning and evening peak periods, also any off-peak period where relevant.

(j) Wherever possible, calculate detailed performance statistics (delay, number of stops, queue length, measures of performance which combine delay and stops using a stop penalty, etc.) for each movement, and the summation for the intersection, for each period considered (Section 6).

(k) Repeat the calculations for several alternative intersection layouts and signal phasings. Choose the best design as the one which yields the minimum value of the chosen performance measure (minimum delay, minimum delay and stops, etc. using the statistics calculated in Step (j)). Consider not only the intersection totals but also the individual movement performances. A design which gives the best total intersection performance at the expense of unacceptable levels of performance for some movements (e.g. pedestrians) should not be readily accepted as the best design. Different performance measures can be used for different periods, and different designs can be found to be more suitable for different periods. Furthermore, alternative designs should also be judged in terms of safety. For example, in terms of delays and stops, a solution with opposed turns may yield better results than one with unopposed turns, but the latter is preferable with regards to safety. Therefore, a judgement must be made by the traffic engineer considering all these factors before a recommendation is made.

For planning and preliminary design purposes, the calculation of green times and detailed performance statistics may be avoided by using the intersection degree of saturation, X, from eqn (3.12) as a simple measure of performance (Y and L to be used in the calculation of X are the values calculated in step (g) with due allowance for any minimum green time constraint using the method given in Section 4).

8.4 MEASURES TO IMPROVE OPERATING CONDITIONS

The techniques presented in this report indicate various measures which can be taken to improve the performance of traffic at a signalised intersection. The following is a brief summary of such measures.

(a) Improvements to intersection geometry and environment such as larger turning radii, better channelisation and pavement markings, improved visibility and reduction of interference from standing vehicles and pedestrians.

(b) Provision of an adequate number and width of lanes (the provision of an additional lane is the most effective way of increasing capacity).

(c) Provision of adequate lane lengths by constructing turn slots of generous length and prohibiting parking or standing on kerb lanes to minimise short lane effects.

(d) Increasing kerb lane utilisation (see *Fig. 5.2* for causes of lane under-utilisation, e.g. provision of adequate lane length at the exit side, suitable locations for bus stops and use of bus bays, prohibition of parking or standing on approches or exits will help).

(e) Better signal phasing to suit varying traffic demands during different periods (peak and off-peak) accompanied by suitable lane arrangements (see Appendix A).

(f) Better signal timings, not only to achieve near optimum operating conditions under normal conditions, but also to achieve substantial increases in capacities by reducing short lane effects (shorter green times in cases where saturation flow falls off during a long green period).

(g) Signal co-ordination to eliminate downstream congestion so as to improve the conditions at the upstream intersection.

(h) Right turn prohibition during periods when the above measures are not sufficient to achieve satisfactory operating conditions (with due consideration to the network effects of the prohibition).

The examples given in Section 9 illustrate the effectiveness of some of the measures listed above.

9. NUMERICAL EXAMPLES

9.1 EXAMPLE 1 — SATURATION FLOW ESTIMATION (Equal lane utilisation)

The problem is (i) to estimate the saturation flows for a level approach road 8.6 m wide under the three alternative lane arrangements shown in *Fig. 9.1*; and (ii) to calculate the movement flow ratios and degrees of saturation (y and x values) in each case. The three alternatives to be considered are:

(a) Two shared 4.3 m lanes with *opposed* right turns from the second lane.

(b) Three lanes with *opposed* right turns only from the third lane:
Lane 1: 3.0 m shared through and left-turn lane;
Lane 2: 3.0 m through lane;
Lane 3: 2.6 m exclusive right-turn lane.

(c) Three lanes with *unopposed* right turns from the third lane (separate phase):
Lane 1: 2.9 m shared through and left-turn lane;
Lane 2: 2.9 m through lane;
Lane 3: 2.8 m shared through and right-turn lane.

Arrival flows are:

	Left	*Through*	*Right*	*Total*
CAR/h	100	730	190	1020
HV/h	10	40	30	80
Total	110	770	220	1100

The site conditions are almost ideal for free movement of vehicles. However, there is some pedestrian interference to left-turning vehicles.

The opposing movement flow and saturation flow values are 600 veh/h and 3200 veh/h, respectively. In all cases, the common phase green time is G = 40 s, the cycle time is c = 80 s. Assume that the intergreen times are I = 5 s. Treat the displayed green as the effective green ($g = G$, hence the lost time, $\ell = I = 5$) for all movements except the opposed movement in Case (b). The average number of opposed right turners which can depart after the green period is n_f = 1.8 veh/cycle. Equal lane utilisation is assumed in all cases, i.e. the available capacities of all lanes are fully utilised. It is sufficient to approximate the saturation flows to the nearest 10 tcu/h or veh/h during the estimation process.

9.1.1 Case (a)

The total approach flow is described as a single movement under this lane and phasing arrangement. Choose the base saturation flow for each lane from *Table 5.1* and adjust for lane width (eqn (5.2)) as shown in *Table 9.1(a)*. The movement saturation flow in tcu's is found as the sum of the two lane saturation flows: s_{tcu} = 1780 + 1890 = 3670. To adjust this for traffic composition, calculate first the opposed right-turn equivalent, e_o, from eqn (5.5) since this is a shared lane case.

The opposing movement parameters are:

s = 3200 veh/h = 0.889 veh/s; sg = 0.889 x 40 = 35.6 veh; q = 600 veh/h = 0.167 veh/s; qc = 0.167 x 80 = 13.3 veh; $s - q$ = 0.889 − 0.167 = 0.722 veh/s; and $x = qc/sg$ = 13.3/35.6 = 0.37.

The unsaturated part of the green period for opposing flow from eqn (5.10) is g_u = (35.6 − 13.3)/0.722 = 31 s. The opposed turn saturation flow from *Fig. 5.1*, or from eqn (F.9) in Appendix F, is s_u = 0.183 veh/s = 660 veh/h.

Therefore, from eqn (5.5):

$$e_o = \frac{0.5 \times 40}{0.183 \times 31 + 1.8} = 2.7$$

For HV's, use e_o + 1 = 3.7

The traffic adjustment factor calculated from eqn (5.4) using the above through car equivalents, together with others taken from *Table 5.2*, is f_c = 1.44 as shown in *Table 9.1(b)*. Therefore, the movement saturation flow is estimated to be $s_{veh} = s_{tcu}/f_c$ = 3670/1.44 = 2550 veh/h. The flow ratio (eqn (3.3)) is $y = q/s$ = 1100/2550 = 0.43, and the degree of saturation (eqn (3.4)) is $x = y/u$ = 0.43/0.50 = 0.86, where the green time ratio $u = g/c$ = 40/80 = 0.50. Because this is larger than the opposing movement degree of saturation (x = 0.37), the movement in question is the critical one in the phase.

9.1.2 Case (b)

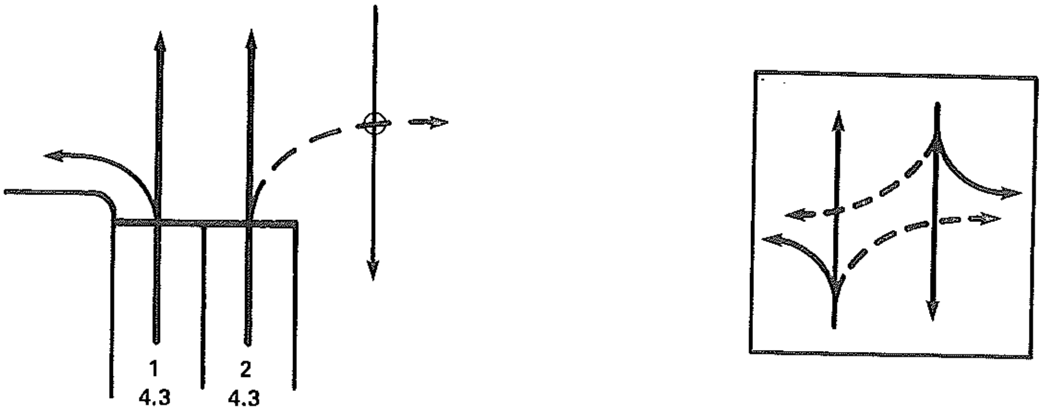
In this case, the opposed right turns will be described as a separate movement because of an exclusive lane. Call this Movement 1, and the traffic in lanes 1 and 2 together Movement 2, and estimate saturation flows separately.

Movement 1

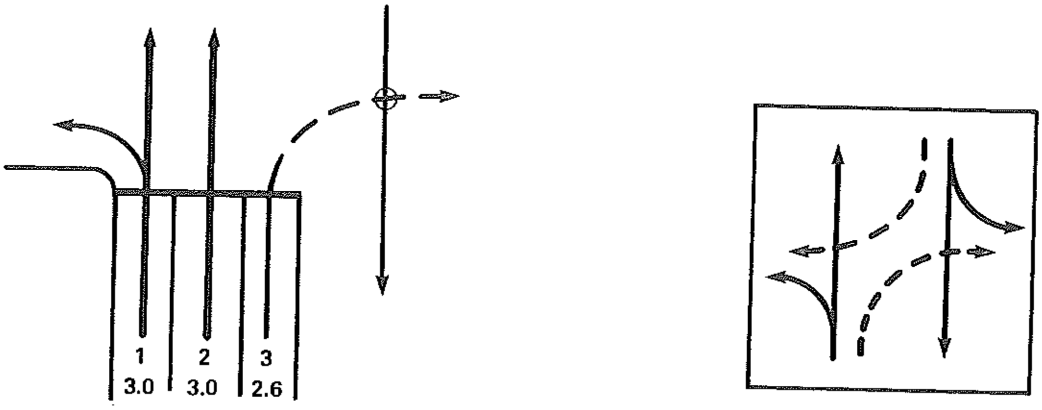
From eqn (5.6), the effective saturation flow for this opposed turn movement is s_o = 1800/2.7 = 670 veh/h, where e_o = 2.7 is as calculated above.

Alternatively, the saturation flow is taken as $s = s_u$ = 660 veh/h (as determined above) and the effective green time is calculated from eqn (5.7) using g_u = 31 s as determined above: g_o = 31 + (1.8/0.183) = 41 s. The corresponding lost time from eqn (5.8) is ℓ_o = 40 + 5 − 41 = 4 s. The movement flow ratio, green time ratio and degree of saturation are y = 220/660 = 0.33, u = 41/80 = 0.51 and x = 0.33/0.51 = 0.65. To check, the capacity from eqn (5.9) is Q = 670 x 40/80 = 660 x 41/80 = 340 veh/h and x = 220/340 = 0.65. The closeness of the values of g_o and g in this example makes it difficult to show the difference between the two methods, i.e. using s_o and g against using s_u and g_o. As a variation on this example, assume that the opposing movement saturation flow is 1700 veh/h. Therefore, g_u = 18 s, e_o = 3.9 and g_o = 28 s are found. Using e_o, s_o = 460 veh/h, y = 220/460 = 0.48 and u = 40/80 = 0.50 are obtained, whereas using g_o, y = 220/660 = 0.33 and u = 28/80 = 0.35 are found. Although the same capacity, Q = 0.50 x 460 = 0.35 x 600 = 230 veh/h, and hence x = 220/230 = 0.96 is found, the predictions of delay, stop rate and queue length will be

(a) Two shared lanes with opposed right-turns



(b) Three lanes with opposed right-turns from an exclusive lane



(c) Three lanes with unopposed right-turns from a shared lane

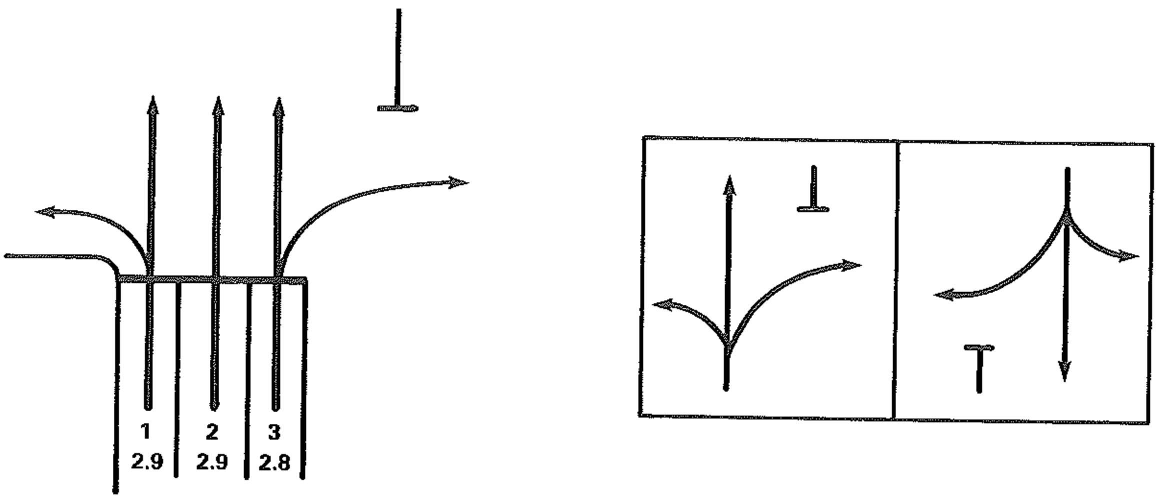


Fig. 9.1 — Lane arrangements and phasing descriptions for Example 1

TABLE 9.1 (a)

EXAMPLE 1, CASE (a): BASE SATURATION FLOW (tcu/h)

Lane Number	Environment and Lane Type	Sat. Flow (Table 5.1) tcu/h	Lane Width m	Factor, f_w (eqn 5.2)	Sat. Flow tcu/h
1	*A,3*	*1700*	*4.3*	*1.045*	*1780*
2	*A,2*	*1810*	*4.3*	*1.045*	*1890*

$s_{tcu} = 1780 + 1890 = 3670$

TABLE 9.1 (b)

EXAMPLE 1, CASE (a): TRAFFIC COMPOSITION ADJUSTMENT FACTOR (f_c)

	Left		Through		Right		Total
	CAR	HV	CAR	HV	CAR	HV	
Flow count, q_i (veh)	*100*	*10*	*730*	*40*	*190*	*30*	*1100* = q
Equivalent, e_i (tcu/veh)	*1.25*	*2.5*	1	2	*2.7*	*3.7*	
Weighted flow, $e_i q_i$ (tcu)	*125*	*25*	*730*	*80*	*513*	*111*	*1584* = $\Sigma e_i q_i$

$f_c = 1584/1100 = 1.44$ tcu/veh

$s = 3670/1.44 = 2550$ veh/h

TABLE 9.2 (a)

EXAMPLE 1, CASE (b): BASE SATURATION FLOW (tcu/h)

Lane Number	Environment and Lane Type	Sat. Flow (Table 5.1) tcu/h	Lane Width m	Factor, f_w (eqn 5.2)	Sat. Flow tcu/h
1	*A,3*	*1700*	*3.0*	*1.0*	*1700*
2	*A,1*	*1850*	*3.0*	*1.0*	*1850*

$s_{tcu} = 1700 + 1850 = 3550$

TABLE 9.2 (b)

EXAMPLE 1, CASE (b): TRAFFIC COMPOSITION ADJUSTMENT FACTOR (f_c)

	Left		Through		Right		Total
	CAR	HV	CAR	HV	CAR	HV	
Flow count, q_i (veh)	*100*	*10*	*730*	*40*	—	—	*880* = q
Equivalent, e_i (tcu/veh)	*1.25*	*2.5*	1	2			
Weighted flow, $e_i q_i$ (tcu)	*125*	*25*	*730*	*80*	—	—	*960* = $\Sigma e_i q_i$

$f_c = 960/880 = 1.09$ tcu/veh

$s = 3550/1.09 = 3260$ veh/h

TABLE 9.3(a)

EXAMPLE 1, CASE (c): BASE SATURATION FLOW (tcu/h)

Lane Number	Environment and Lane Type	Sat. Flow (Table 5.1) tcu/h	Lane Width m	Factor, f_w (eqn 5.2)	Sat. Flow tcu/h
1	*A,3*	*1700*	*2.9*	*0.96*	*1630*
2	*A,1*	*1850*	*2.9*	*0.96*	*1780*
3	*A,2*	*1810*	*2.8*	*0.94*	*1700*

s_{tcu} = *1630 + 1780 + 1700 = 5110*

TABLE 9.3(b)

EXAMPLE 1, CASE (c): TRAFFIC COMPOSITION ADJUSTMENT FACTOR (f_c)

	Left		Through		Right		Total
	CAR	HV	CAR	HV	CAR	HV	
Flow count, q_i (veh)	*100*	*10*	*730*	*40*	*190*	*30*	*1100* = q
Equivalent, e_i (tcu/veh)	*1.25*	*2.5*	1	2	*1*	2	
Weighted flow, e_iq_i (tcu)	*125*	*25*	*730*	*80*	*190*	*60*	*1210* = Σe_iq_i

f_c = *1210/1100 = 1.10 tcu/veh*

s = *5110/1.10 = 4650 veh/h*

TABLE 9.4

EXAMPLE 2, MOVEMENT 1: TRAFFIC COMPOSITION ADJUSTMENT FACTOR (f_c)

	Left		Through		Right		Total
	CAR	HV	CAR	HV	CAR	HV	
Flow count, q_i (veh)	*100*	*10*	*122*	*7*	–	–	*239* = q
Equivalent, e_i (tcu/veh)	*1.25*	*2.5*	1	2			
Weighted flow, e_iq_i (tcu)	*125*	*25*	*122*	*14*	–	–	*286* = Σe_iq_i

f_c=*286/239 = 1.20 tcu/veh*

s_{tcu}=*1630, s =1630/1.20 = 1360 veh/h*

TABLE 9.5

EXAMPLE 2, MOVEMENT 2: TRAFFIC COMPOSITION ADJUSTMENT FACTOR (f_c)

	Left		Through		Right		Total
	CAR	HV	CAR	HV	CAR	HV	
Flow count, q_i (veh)	–	–	*608*	*33*	*190*	*30*	*861* = q
Equivalent, e_i (tcu/veh)			1	2	*1*	*2*	
Weighted flow, e_iq_i (tcu)	–	–	*608*	*66*	*190*	*60*	*924* = Σe_iq_i

f_c = *924/861 = 1.07 tcu/veh*

s_{tcu} = *3480, s = 3480/1.07 = 3250 veh/h* different because of the differences in y and u values. The method of using g_o is preferred since this reflects a more realistic departure pattern for the opposed turns in exclusive lanes (see *Fig. F. 1* in Appendix F which illustrates the difference between the two methods).

Movement 2

The saturation flow for the traffic in lanes 1 and 2 can be estimated in a way similar to Case (a), but excluding the right-turning traffic. As seen in *Tables 9.2(a)* and *9.2(b)*, s_{tcu} = 3550 and f_c = 1.09 are found. Therefore the saturation flow is s_{veh} = 3550/1.09 = 3260 veh/h. The movement flow ratio is y = 880/3260 = 0.27, and degree of saturation is x = 0.27/0.50 = 0.54.

A higher degree of saturation for the opposed right-turn movement (x = 0.65) indicates that Movement 1 is the more critical movement in the phase.

9.1.3 Case (c)

Traffic in all lanes will be described as a single movement because equal lane utilisation is assumed. The calculations are similar to Case (a), except that the right-turns are unopposed, hence e = 1 for cars and 2 for HV's (from *Table 5.2* for normal turns). From *Tables 9.3(a)* and *9.3(b)*, s_{tcu} = 5110 and f_c = 1.10, and hence the movement saturation flow is s_{veh} = 5110/1.10 = 4650 veh/h. The flow ratio and degree of saturation are y = 1100/4650 = 0.24 and x = 0.24/0.50 = 0.48.

9.1.4 A Comparison

The three lane and phasing arrangements considered above can be compared in terms of the degrees of saturation obtained in each case. The decrease in the critical degree of saturation from 0.86 in Case (a) to 0.65 in Case (b) indicates that Case (b) represents a better lane arrangement. On the other hand, a further decrease to 0.48 in Case (c) does not necessarily indicate a better solution because this corresponds to a different phasing arrangement (the opposing movement requires additional time). For a satisfactory comparison, calculate the 'required movement times' from eqn (4.2) using c = 80 and x_p = 0.9, i.e. t = 80(y/0.9) + ℓ = 89 y + ℓ:

Case (a):	t = 89 x 0.43 + 5 = 43
Case (b):	t = 89 x 0.33 + 4 = 33 (opposed right-turn movement)
Case (c):	t = 89 x 0.24 + 5 = 26

The opposing movement requires t = 89 x (600/3200) + 5 = 22 s. In Cases (a) and (b), this does not affect the total time required for the intersection because the opposing movement shares the same phase with the movements considered and it has a smaller t value. In other words, it is not the critical movement in the phase. However, a separate phase is allocated to the opposing movement in Case (c), therefore the contribution to the total time required for the intersection is Σt = 26 + 22 = 48 (also note Σy = 0.43, $\Sigma \ell$ = 10). Therefore Case (c) represents the worst alternative considered in terms of operational efficiency (but not necessarily for safety).

9.2 EXAMPLE 2 — SATURATION FLOW ESTIMATION (Under-utilised lanes)

We now wish to estimate the saturation flows for Case (c) in Example 1, assuming that the kerb lane is under-utilised.

Treat the traffic using the kerb lane and the traffic using the other two lanes as two separate movements (Movements 1 and 2, respectively). As calculated in Example 1, the saturation flows in tcu's are s_{tcu} = 1630 for Movement 1 and s_{tcu} = 1780 + 1700 = 3480 for Movement 2. Find flows, traffic composition adjustment factors, saturation flows in veh's, and y, x values for each movement.

Movement 1

The number of through vehicles using the kerb lane can be estimated from $q_{T1} = q_T/2n$ where q_T = total through flow and n = number of lanes. For this example, n = 3, therefore q_{T1} = (730/6 = 122 CAR's) + (40/6 = 7 HV's) = 129 veh/h. Therefore, the traffic composition adjustment factor is f_c = 1.20 and the saturation flow is s_1 = 1630/1.20 = 1360 veh/h as calculated in *Table 9.4*. The movement flow ratio and degree of saturation are y_1 = 239/1360 = 0.18 and x_1 = 0.18/0.50 = 0.36 (where $u = g/c$ = 0.50 as in Example 1).

Movement 2

As for Case (c) in Example 1, but excluding the left-turning flow and part of the through flow using the kerb lane, f_c = 1.07 is found as shown in *Table 9.5*. Therefore, s_2 = 3480/1.07 = 3250 veh/h, y_2 = 861/3250 = 0.26 and x_2 = 0.26/0.50 = 0.52 are found.

The utilisation ratio of the kerb lane from eqn (5.13) is

$$\rho = y_1/y_2 = 0.18/0.26 = 0.69,$$

i.e. 69 per cent of the kerb lane capacity is utilised.

In this case, the critical movement for the approach road is Movement 2 with x = 0.52. Compared with Case (c) in Example 1, the critical degree of saturation is increased from 0.48 to 0.52 due to kerb lane under-utilisation.

If wanted, the two movements can be combined using the lane utilisation ratio calculated above so as to describe all traffic from the approach road as a single movement:

$$s = \rho s_1 + s_2 = 0.69 \times 1360 + 3250 = 4180 \text{ veh/h}$$

Since the total flow is q = 1100 veh/h, the flow ratio is y = 1100/4180 = 0.26 (= y_2 as found above). If ρ = 0.69 was known in advance, the same result would be obtained as follows: s_{tcu} = 0.69 x 1630 + 3480 = 4600 tcu/h, f_c = 1.10 as found in *Table 9.3(b)* for Case (c) in Example 1, hence s = 4600/1.10 = 4180 veh/h.

9.3 EXAMPLE 3 — LANE FLOWS

The problem is to calculate lane flows for Case (a) in Example 1 using the method described in Section 5.7 and assuming equal lane utilisation.

For the solution of this problem, it is better to use initially flows and saturation flows in tcu's in eqn (5.15) or eqn (5.16) because the lane traffic compositions are not known. Note that $q_{tcu} = \Sigma e_i q_i$ and s_{tcu} are already calculated in Example 1:

Lane 1: s_1 = 1780 tcu/h.
Lane 2: s_2 = 1890 tcu/h.
Movement: s = 3670 tcu/h, q = 1584 tcu/h, y = 0.43.

The turning volumes in tcu's are:

q_L = 150 tcu/h, q_T = 810 tcu/h, and q_R = 624 tcu/h.

The lane flows are

$$q_1 = q_L + q_{T,1} \text{ and } q_2 = q_R + q_{T,2}$$

where q_L and q_R are left-turning and right turning flows, and $q_{T,1}$ and $q_{T,2}$ are through flows in lanes 1 and 2, respectively.

Because the effective green times are the same for both lanes, use eqn (5.16) to calculate lane flows:

$q_1 = y\,s_1 = 0.43 \times 1780 = 770$ tcu/h,
$q_2 = y\,s_2 = 0.43 \times 1890 = 810$ tcu/h.

Therefore,

$q_{T,1} = q_1 - q_L = 770 - 150 = 620$ tcu/h,
$q_{T,2} = q_2 - q_R = 810 - 624 = 190$ tcu/h.

It is seen that a smaller proportion of through traffic (23 per cent) use the second lane because of its reduced capacity due to opposed right turns (this is in accordance with the 'equal lane utilisation' assumption).

Now calculate lane flows and saturation flows in real vehicle units. Because $q_T = 770$ veh/h $= 810$ tcu/h, the through traffic composition factor is $f_{c,T} = 810/770 = 1.05$. Therefore,

$q_{T,1} = 620/1.05 = 590$ veh/h
$q_{T,2} = 190/1.05 = 180$ veh/h

The lane flows are:

$q_1 = 110 + 590 = 700$ veh/h,
$q_2 = 220 + 180 = 400$ veh/h.

The lane traffic composition factors are

$f_{c,1} = 770/700 = 1.10$,
$f_{c,2} = 810/400 = 2.03$.

The lane saturation flows are

$s_1 = 1780/1.10 = 1620$ veh/h,
$s_2 = 1890/2.03 = 930$ veh/h.

The lane capacities are (using $u = g/c = 0.50$):

$Q_1 = 0.50 \times 1620 = 810$ veh/h,
$Q_2 = 0.50 \times 930 = 465$ veh/h.

To check the results, calculate lane degrees of saturation:

$x_1 = q_1/Q_1 = 700/810 = 0.86$,
$x_2 = q_1/Q_1 = 400/465 = 0.86$.

Hence the lane flows are obtained correctly to give equal degrees of saturation $x_1 = x_2 = x = 0.86$. This is the same as the movement degree of saturation calculated in Example 1 without knowing the lane flows (note that $s_1 + s_2 = 1620 + 930 = 2550$ veh/h is the movement saturation flow calculated in Example 1, and the movement capacity is $Q = 1275$ veh/h $= Q_1 + Q_2$).

It should be noted that the lane flow calculations in this example are for given signal timings. Because the opposed turn equivalent, e_o, is a function of signal timings, the method given in Section 5.7 indicates that the distribution of flows between lanes also depends on signal timings in this case. As a further example, solve this problem using the same data as in Example 1, Case (a), except that the cycle time is increased to $c = 100$ s (ANSWERS: $e_o = 3.0$, $s = 2450$ veh/h, $q_1 = 730$ veh/h, $q_2 = 370$ veh/h, $s_1 = 1620$ veh/h, $s_2 = 830$ veh/h, $Q_1 = 650$ veh/h, $Q_2 = 330$ veh/h, $Q = 980$ veh/h, $x_1 = x_2 = x = 1.12$, i.e. oversaturation is predicted).

The lane flows and saturation flows in Case (b) are:

Lane 1: $q_1 = 400$ veh/h, $s_1 = 1500$ veh/h,
Lane 2: $q_2 = 480$ veh/h, $s_2 = 1760$ veh/h,
Lane 3: $q_3 = q_R = 220$ veh/h, $s_3 = s_R = 660$ veh/h.

In Case (c):

Lane 1: $q_1 = 340$ veh/h, $s_1 = 1420$ veh/h,
Lane 2: $q_2 = 400$ veh/h, $s_2 = 1700$ veh/h,
Lane 3: $q_3 = 360$ veh/h, $s_3 = 1530$ veh/h.

9.4 EXAMPLE 4 — SHORT LANE SATURATION FLOW

Estimate the movement saturation flows for Cases (a) and (b) in Example 1 with parking allowed on the approach road. The effective kerb lane length available for queueing is 60 m. It shall be assumed that the average queue space per vehicle is 7.2 m.

9.4.1 Case (a)

The data required for eqn (5.20) to calculate the critical queueing distance are $q = 1100$ veh/h $= 0.306$ veh/s, $r = c - g = 80 - 40 = 40$ s, $y = 0.43$ (with full saturation flow as found in Example 1), and the number of lanes, $n = 2$. Therefore,

$$D_c = 7.2 \times 0.306 \times 40/2\,(1 - 0.43) = 80 \text{ m}.$$

Since $D < D_c$, short lane effect will occur.

From eqn (5.19), $s_1' = 500 \times 60/40 = 750$ veh/h. To compare this with $s_1 = 1780$ tcu/h, calculate s_1' in tcu/h. $Q_1 = 750\,(40/80) = 375$ veh/h. From Example 1, $q_L = 110$ veh/h, $q_L^* = 150$ tcu/h, $q_T = 770$ veh/h, $q_T^* = 810$ tcu/h. Since $q_L < Q_1$, eqn (5.21) gives:

$Q_1^* = 150 + (810/770)\,(375 - 110) = 430$ tcu/h, and the short lane saturation flow in tcu/h is: $(430/375)\,750 = 860 < s_1 = 1780$ tcu/h. The movement saturation flow is therefore $s = 860 + 1890 = 2750$ tcu/h, and using $f_c = 1.44$ from Example 1, $s = 2750/1.44 = 1910$ veh/h.

The flow ratio is $y = 1100/1910 = 0.58$, and the degree of saturation is $x = 0.58/0.50 = 1.16$, i.e. oversaturation will exist.

9.4.2 Case (b)

This applies to Movement 2 (traffic in lanes 1 and 2) with $q = 880$ veh/h $= 0.244$ veh/s and $y = 0.27$ as found in Example 1. Therefore, the critical queueing distance from eqn (5.20) is:

$$D_c = 7.2 \times 0.244 \times 40/2\,(1 - 0.27) = 50 \text{ m}.$$

Since $D > D_c$, the short lane length is large enough to contain the maximum back of the queue in an average cycle, and hence to retain the full saturation flow as in Example 1 ($s = 3260$ veh/h).

9.5 EXAMPLE 5: SHORT LANE SATURATION FLOW

In the approach road shown in *Fig. 9.2*, one of the two lanes allocated to through traffic is consistently blocked by the opposed right-turners queueing in the lane. The result is a short lane effect with an available queueing distance of 110 m. There is no interference by left-turners. The through traffic flow is 1500 veh/h with 10 per cent heavy vehicles. The through lane saturation flows are 1680 veh/h each. We wish to estimate the through movement saturation flow when:

(a) the cycle time and green time are $c = 150$ s and $g = 90$ s; and
(b) the cycle time is reduced to $c = 100$ s and the same green time ratio is retained.

We also wish to compare the degrees of saturation in Cases (a) and (b) so as to see the effect of reducing cycle time.

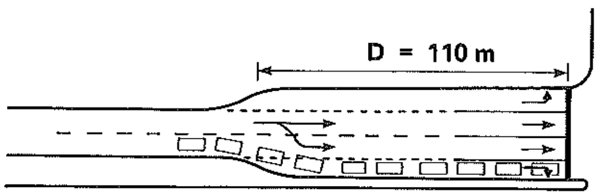


Fig. 9.2 — The approach road for Example 5

9.5.1 Case (a)

The average queue space is estimated as j = 6 x 0.90 + 12 x 0.10 = 6.6 m/veh (where 0.90 and 0.10 are the proportions of CAR's and HV's, respectively). Full movement saturation flow is s = 2 x 1680 = 3360 veh/h, and the flow ratio is y = 1500/3360 = 0.45. The red time is $r = c - g = 150 - 90 = 60$ s, and the flow rate is q = 1500 veh/h = 0.417 veh/s. Therefore, the critical distance from eqn (5.20) is

D_c = 6.6 x 0.417 x 60/2 (1 − 0.45) = 150 m.

Since D = 110 m < D_c, short lane effect will occur.

From eqn (5.18), $s_1{}'$ = 3600 x 110/(6.6 x 90) = 670 veh/h. Since $s_1{}' < s_1$ = 1680 veh/h, the movement saturation flow is $s = s_1{}' + s_2$ = 670 + 1680 = 2350 veh/h. The capacity is Q = 2350 (90/150) = 1410 veh/h, and the degree of saturation is $x = q/Q$ = 1500/1410 = 1.06 (oversaturated).

9.5.2 Case (b)

The green time ratio is u = 90/150 = 0.60, and the green time with c = 100 s is $g = uc$ = 0.60 x 100 = 60 s. Therefore, the red time is decreased to r = 100 − 60 = 40 s. The critical distance from eqn (5.20) is:

D_c = 6.6 x 0.417 x 40/2 (1 − 0.45) = 100 m.

Since $D > D_c$, full saturation flow will be obtained, hence $Q = us$ = 0.60 x 3360 = 2020 veh/h, x = 1500/2020 = 0.74. It is seen that the short lane effect and the resulting oversaturation can be prevented by decreasing the cycle time provided the time requirements of other (critical) movements at the intersection are satisfied with the smaller cycle time (see Appendix F.3).

9.6 EXAMPLE 6 — MEASURES OF PERFORMANCE

For Cases (a) and (b) in Example 5, we wish to calculate the average overflow queue, the maximum back of the queue, average delay per vehicle and the number of stops for the through movement. We also wish to calculate an estimate of the amount of fuel consumption due to delays and stops in Case (b) using f_2 = 2.20 L/veh-h and $f_3{}'$ = 0.04 L/stop in eqn (6.15). The flow period is T_f = 30 minutes, and the average queue space is 6.6 m/veh.

Firstly, determine the values of the parameters to be used in these calculations (see Section 6).

9.6.1 Case (a)

q = 1500 veh/h = 0.417 veh/s, qc = 0.417 x 150 = 62.5 veh, qr = 0.417 x 60 = 25.0 veh, s = 2350 veh/h = 0.653 veh/s, sg = 0.653 x 90 = 58.8 veh, y = 1500/2350 = 0.64, u = 90/150 = 0.60, Q = 0.60 x 2350 = 1410 veh/h, x = 1500/1410 = 1.06, QT_f = 1410 x 0.5 = 705 veh, x_o = 0.67 + (58.8/600) = 0.77, $z = x - 1$ = 1.06 − 1 = 0.06 (oversaturated case).

The average overflow queue from eqn (6.1) is

$$N_o = \frac{705}{4}\left(0.06 + \sqrt{(0.06)^2 + \frac{12(1.06 - 0.77)}{705}}\right) = 26.9 \text{ veh.}$$

The maximum back of the queue in an average cycle from eqn (6.8) is

$$N_m = 25.0/(1 - 0.64) + 26.9 = 96.3 \text{ veh.}$$

Note that 110/6.6 = 17 veh can queue in the short lane and the maximum back of the queue in the other lane is 96 − 17 = 79 veh (about 520 m from the stop line).

The total delay from eqn (6.3) is

$$D = \frac{62.5\,(1 - 0.6)^2}{2(1 - 0.64)} + 26.9 \times 1.06 = 42.40 \text{ veh-h/h}$$

The average delay from eqn (6.4) is d = 42.40/0.417 = 102 s.

The stop rate from eqn (6.5) is

$$h = 0.9\left(\frac{1 - 0.6}{1 - 0.64} + \frac{26.9}{62.5}\right) = 1.39 \text{ stops/veh}$$

The number of stops from eqn (6.6) is H = 1500 x 1.39 = 2085 stops/h.

9.6.2 Case (b)

The parameters which differ from Case (a) are:

qc = 0.417 x 100 = 41.7, qr = 0.417 x 40 = 16.7 veh, s = 3360 veh/h = 0.933 veh/s, sg = 0.933 x 60 = 56.0, y = 1500/3360 = 0.45, Q = 0.60 x 3360 = 2020 veh/h, x = 1500/2020 = 0.74, QT_f = 2010 x 0.5 = 1005 veh, x_o = 0.67 + (56.0/600) = 0.76.

Since $x < x_o$, N_o = 0. Therefore, the maximum back of the queue is N_m = 16.7/(1 − 0.45) = 30.4 veh. This corresponds to 6.6(30.4/2) = 100 m; compare this with the back of the queue which is predicted to reach about 520 m in Case (a).

The total delay is

$$D = \frac{41.7(1 - 0.6)^2}{2(1 - 0.45)} + 0 = 6.07 \text{ veh-h/h},$$

the average delay is d = 6.07/0.417 = 15 s, and the stop rate is

$$h = 0.9\left(\frac{1 - 0.6}{1 - 0.45} + 0\right) = 0.65 \text{ stops/veh}$$

Therefore, the number of stops is H = 1500 x 0.65 = 975 stops/h.

The amount of fuel consumption due to delays and stops in Case (b) can be calculated from eqn (6.15) by neglecting the fuel consumed while cruising, i.e. $E = f_2 D + f_3{}' H$ = 2.20 x 6.07 + 0.04 x 975 = 54 L/h.

9.7 EXAMPLE 7 — SATURATION FLOWS AND SIGNAL TIMINGS WITH OPPOSED TURNS

Calculate saturation flows and signal timings for the intersection shown in *Fig. 9.3(a)* which operates under a simple two-phase system as shown in *Fig. 9.3(b)*. The flows are given in *Fig. 9.3(c)*, separated into CAR's and HV's as hourly figures. The environment is almost ideal for the movement of vehicles. However, restricted turning conditions exist for left turners (small turning radii and pedestrian interference). All lanes are 3.3 m wide. All approach roads are level. The North and South approach roads have two lanes each and the East and West approach roads have four lanes each. All kerb and median lanes are shared by through and turning vehicles except for the South approach from which right turning is banned. All right turn flows are opposed. Assume that the opposed turn equivalents for the East and West approaches are fixed and $e_o = 3$ (this assumption will have little effect on the results since very light turning flows are involved). Therefore, only the opposed right turn equivalent for the North approach road will be treated as variable (use $n_f = 1.5$ for calculating e_o). Kerb lanes on the East and West approaches are under-utilised. It has been observed that only 5 per cent of through vehicles use these lanes. The minimum green times are 10 s, and intergreen times are 5 s for both phases, hence for all movements. Also assume that all movement lost times are 5 s ($\ell = I$).

9.7.1 Movement Description

Because the kerb lanes on the East and West approach roads are under-utilised, the traffic (through and left turn) using these lanes will be described as

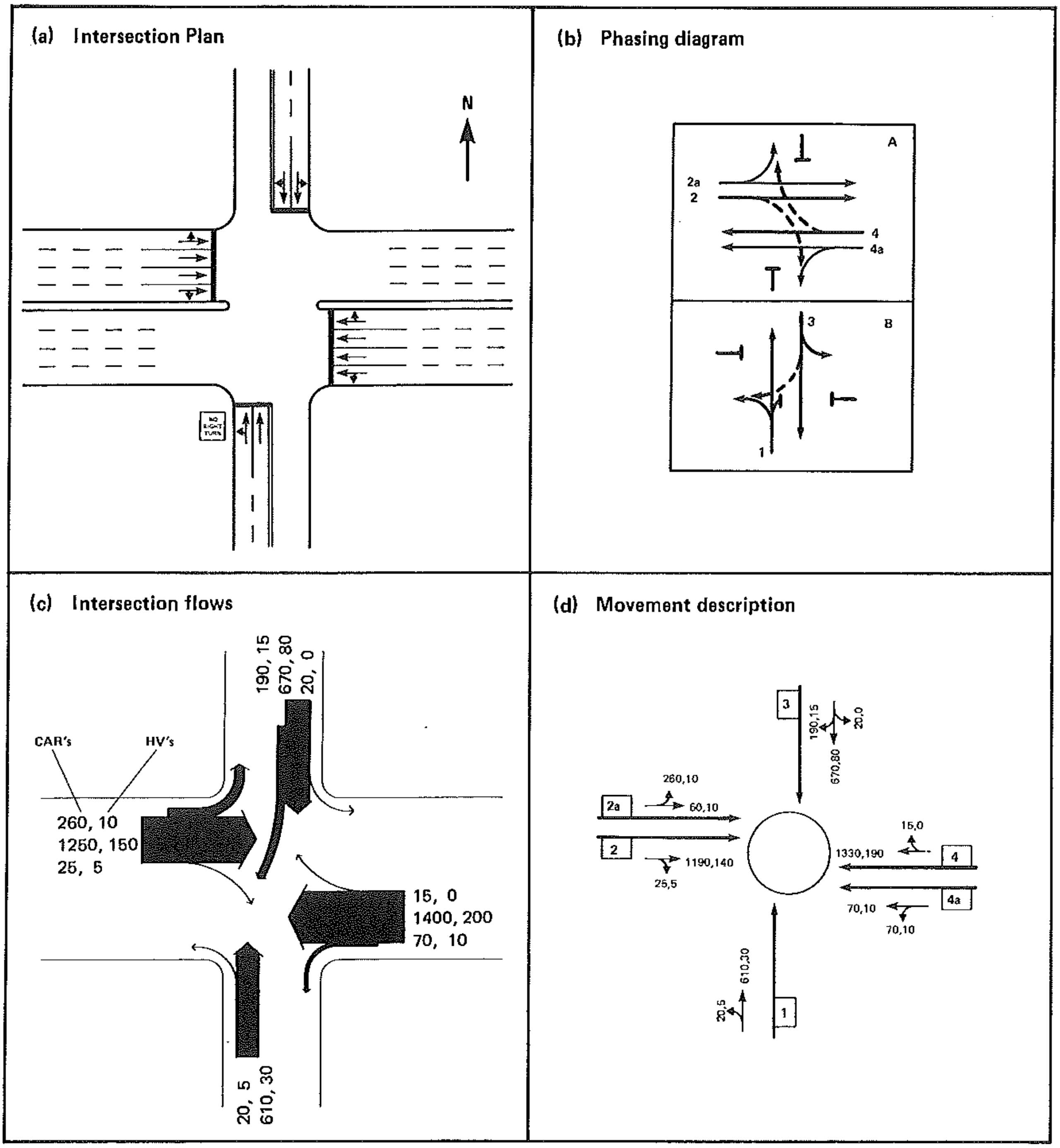


Fig. 9.3 — Intersection plan, phasing diagram, intersection flows and movement description for Example 7

separate movements. The resulting movement description for the intersection is shown in *Fig. 9.3(d)*, together with the flow data for separate movements on the East and West approaches (based on 5 per cent through flow in kerb lanes).

9.7.2 Initial Signal Timings

Saturation flows are independent of signal timings for all movements except Movement 3 because its opposed turn equivalent will be treated as a variable. For an initial estimate of Movement 3 saturation flow, assume unopposed turns. Because of normal turning conditions for right turns, the equivalents from *Tables 5.2* are 1.0 for CAR's and 2.0 for HV's for this movement. The calculations of base saturation flows in tcu/h(s_t) are given in *Table 9.6(a)*. Because all lane widths are 3.3 m in this example, all f_w = 1.0 (eqn (5.2)), and hence there is no need for lane width

TABLE 9.6(a)

EXAMPLE 7: BASE SATURATION FLOWS (tcu/h)

Movement Number	*Lane Number*	*Environment and Lane Type*	*Lane Sat. Flow**	*Movement Sat. Flow (s_t)*
1	1	A,3	1700	
	2	A,1	1850	3550
2a	1	A,3	1700	1700
2	1	A,1	1850	
	2	A,1	1850	
	3	A,2	1810	5510
3	1	A,3	1700	
	2	A,2	1810	3510
4a		As for Movement 2a		1700
4		As for Movement 2		5510

* From Table 5.1

TABLE 9.6(b)

EXAMPLE 7: TRAFFIC COMPOSITION ADJUSTMENT FACTORS (f_c)

Movement Number		*Left* CAR	*Left* HV	*Through* CAR	*Through* HV	*Right* CAR	*Right* HV	*Total*	f_c*
1	q_i	20	5	810	30	—	—	665	
	e_i	1.25	2.5	1	2				1.07
	$e_i q_i$	25	13	610	60			708	
2a	q_i	260	10	60	10	—	—	340	
	e_i	1.25	2.5	1	2				1.27
	$e_i q_i$	325	25	60	20			430	
2	q_i	—	—	1190	140	25	5	1360	
	e_i			1	2	3	4		1.15
	$e_i q_i$			1190	260	75	20	1565	
3	q_i	20	0	670	80	190	15	975	
	e_i	1.25		1	2	1	2		1.10
	$e_i q_i$	25	0	670	160	190	30	1075	
4a	q_i	70	10	70	10	—	—	160	
	e_i	1.25	2.6	1	2				1.27
	$e_i q_i$	88	25	70	20			203	
4	q_i	—	—	1330	190	15	0	1535	
	e_i			1	2	3			1.14
	$e_i q_i$			1330	380	45	0	1755	

* $f_c = (\Sigma e_i q_i)/q$

TABLE 9.6(c)

EXAMPLE 7: SATURATION FLOWS (veh/h) AND FLOW RATIOS (y)

Movement Number	s_t *(tcu/h)*	f_c	s *(veh/h)*	q *(veh/h)*	y *(= q/s)*
1	3550	1.07	3320	665	0.20
2a	1700	1.27	1340	340	0.25
2	5510	1.15	4790	1360	0.28
3	3510	1.10	3190	975	0.31
4a	1700	1.27	1340	160	0.12
4	5510	1.14	4630	1535	0.32

TABLE 9.6(d)

EXAMPLE 7: RECALCULATION OF f_c FOR MOVEMENT 3

First Iteration

	Left		Through		Right		Total
	CAR	HV	CAR	HV	CAR	HV	
Flow count, q_i (veh)	20	0	670	80	190	15	975 = q
Equivalent, e_i (tcu/veh)	1.25		1	2	2.7	3.7	f_c = 1.46
Weighted flow, e_iq_i (tcu)	25	0	670	160	513	56	1424 = Σe_iq_i

s = 3510/1.46 = 2400 veh/h

Second Iteration

	Left		Through		Right		Total
	CAR	HV	CAR	HV	CAR	HV	
Flow count, q_i (veh)	20	0	670	80	190	15	975 = q
Equivalent, e_i (tcu/veh)	1.25		1	2	2.8	3.8	f_c = 1.48
Weighted flow, e_iq_i (tcu)	25	0	670	160	532	57	1444 = Σe_iq_i

s = 3510/1.48 = 2370 veh/h

adjustments. Similarly, all $G_r = 0$, $f_g = 1.0$. Traffic composition adjustment factors (f_c) are calculated in *Table 9.6(b)*. Movement saturation flows ($s = s_f/f_c$) and flow ratios ($y = q/s$) are calculated in *Table 9.6(c)*.

The lane utilisation ratios for kerb lanes on the West and East approach roads are found $\rho_{2a} = y_{2a}/y_2 = 0.25/0.28 = 0.89$ and $\rho_{4a} = y_{4a}/y_4 = 0.12/0.32 = 0.38$, respectively. High kerb lane utilisation ratio for the West approach road is due to the heavy left turn movement.

Critical movement identification is a simple matter for this problem because there are no overlap movements and all movements are assumed to have the same lost time value ($\ell = 5$). Therefore, the movements with the highest y value in each phase are the critical ones: (2 or 4) in phase A, (1 or 3) in phase B (no need to consider Movements 2a and 4a since they represent under-utilised lanes). From *Table 9.6(c)*, Movements 4 and 3 are found critical, and the intersection flow ratio and lost time are $Y = y_4 + y_3 = 0.32 + 0.31 = 0.63$ and $L = 2 \times 5 = 10$ s.

The intersection green time ratio required for a maximum degree of saturation, $X_p = 0.90$ is $U = Y/X_p = 0.63/0.90 = 0.70$. Therefore, the practical cycle time from eqn (7.2) is $c_p = 10/(1 - 0.70) = 33$ s. The approximate cycle time for minimum delay from eqn (7.1) is $c_o = (1.4 \times 10 + 6)/(1 - 0.63) = 20/0.37 = 54$ s. Choose a cycle time of $c = 50$ s.

The total available green time is $c - L = 50 - 10 = 40$ s. The green times from eqn (7.5) using (y, Y) instead of (u, U) because of equal degrees of saturation, $g_4 = (40/0.63)0.32 = 20$ s and $g_3 = (40/0.63)0.31 = 20$ s. Since all $\ell = I$, $G_A = 20$ and $G_B = 20$ are found.

9.7.3 First Iteration

Now re-calculate the opposed turn equivalent, e_o, for Movement 3 as a function of these signal timings. Movement 1 is the opposing movement and the parameters to be used for calculating e_o are $s_u = 0.172$ veh/s (from *Fig. 5.1*, or eqn (F.9) for an opposing flow of $q_1 = 665$ veh/h), $g_u = (20 - 0.200 \times 50)/(1 - 0.200) = 13$ s (from eqn 5.10 using $g = g_1 = 20$, $y = y_1 = 0.200$) and $n_f = 1.5$ (given). Therefore, capacity per cycle for opposed turns is $s_u g_u + n_f = 0.172 \times 13 + 1.5 = 3.7$ veh/cycle and the opposed turn equivalent from eqn (5.5) is $e_o = 0.5 \times 20/3.7 = 2.7$ tcu/veh. The revised traffic composition adjustment factor for Movement 3 is $f_c = 1.46$ as shown in *Table 9.7(d)*. Therefore, $s_3 = 3510/1.46 = 2400$ veh/h and $y_3 = 975/2400 = 0.41$ are found. For the intersection, $Y = 0.32 + 0.41 = 0.73$, $U = 0.73/0.90 = 0.81$, $c_p = 10/(1 - 0.81) = 53$ and $c_o = 20/(1 - 0.73) = 74$ s are found. Choosing $c = 60$ s, the green times are $g_3 = (50/0.73)0.41 = 28$ s ($= G_B$), and $g_4 = (50/0.73)\,0.32 = 22$ s ($= G_A$).

9.7.4 Second Iteration

Using the new signal timings $g_u = 20$ s and $e_o = 2.8$ are found. Therefore, $f_c = 1445/975 = 1.48$, $s = 3510/1.48 = 2370$ veh/h, and $y = 975/2370 = 0.41$ (no change) are found. This means that no more iterations are required. The signal timings are as found at the end of the previous iteration: $c = 60$ s, $G_A = 22$ s, $G_B = 28$ s. The intersection degree of saturation from eqn (3.12) is $X = (60/50)0.73 = 0.88 < X_p$.

For further examples which illustrate signal timing calculations in more complicated cases where there are opposed and unopposed periods for the same movement during one signal cycle and there are multiple overlaps, see Examples 6 and 7 in Appendix D.

APPENDICES

APPENDIX A

SIGNAL PHASING

As discussed in Section 2, signal phasing provides the mechanism by which the basic safety and operational efficiency requirements of traffic at a signalised intersection are met. A phasing system determines how various vehicle and pedestrian movements are given right of way by green signals or walk displays. The choice of a phasing system depends on the intersection geometry (the number of approach roads and their physical relationship) and the turning traffic flow levels. The objective of signal phasing design is to minimise accidents by reducing conflicts among movements while maximising operational efficiency by reducing delay, queue length, number of stops, etc.

The design of a phasing system cannot be separated from the design of lane arrangements. The allocation of lanes to various movements from each approach road must therefore be considered carefully before the relative effectiveness of various phasing systems can be analysed.

In addition to safety and operational efficiency, the following factors must also be considered when designing a phasing system:

(a) driver understanding and acceptance of the system;

(b) uniformity at the intersections along a route, or in an area; and

(c) progression considerations for an intersection within a co-ordinated signal system.

Before discussing the detailed aspects of signal phasing design, the existence of two distinct philosophies for the implementation of phasing arrangements in signal controllers should be noted (see Huddart (1980) for an overview).

(a) Phase control philosophy: This is the traditional 'phase-dominant' philosophy whereby the control parameters (intergreen time, minimum green time, etc.) are specified for phases and apply to all movements receiving right of way during a phase. The phasing design objective with this philosophy is usually 'to minimise the number of phases' because it is generally believed that 'the intersection lost time will be decreased by decreasing the number of phases'. This is valid when there are no overlap movements but not correct otherwise (see Example 4 in Appendix D).

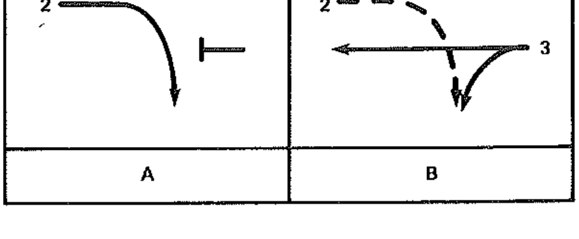
(b) Group control philosophy: This is the 'movement-dominant' philosophy whereby the control parameters are specified for movements rather than phases. It is considered that this philosophy allows for more flexible and efficient solutions, and can handle complicated phasing systems (with multiple overlap movements) associated with modern arrow signalling practices. The signal analysis techniques given in this report (Section 2 and 4, in particular) have been developed with this philosophy in mind. The objective of phasing design with the group control philosophy can be expressed as 'to maximise the amount of overlaps' (even if this increases the number of phases) because this will reduce the total time required to meet the capacity requirements of all (critical) movements at the intersection.

These two philosophies do not necessarily represent the operation of any particular type or make of controller.

An important aspect of phasing design concerns opposed (filter) or unopposed (free with no conflict) turning phases. Opposed turning phases (see Section 5.6 and Appendix F) rely on the resolution of conflicts between movements by means of priority rules, e.g. right-turning vehicles giving way to opposing through vehicles, or left-turning vehicles giving way to pedestrians. This solution helps to improve operational efficiency if used when the basic capacity requirements are met; that is, when there are sufficient gaps in the opposing stream and there is sufficient queueing space for turning vehicles. At the same time, the basic safety requirements such as adequate visibility, acceptable crossing distance, acceptable turning and opposing vehicle speeds must be met. Otherwise, unopposed turn phases must be used using green arrows and not allowing conflicting movements to run simultaneously.

In some cases, both opposed and unopposed phases are used for the same turning movement during one signal cycle (see Appendix F.2). The traditional leading turn (late release of the opposing movement) and lagging turn (early cut-off of the opposing movement) arrangements for right-turning traffic fall into this category. These are illustrated in *Fig. A.1* for a T-junction where there are no right turners from the opposite approach road. If used at a four-way intersection, the lagging turn arrangement presents a safety problem. A driver who wants to turn right from the opposite approach road and sees the signals changing to amber (at the end of filter turn Phase A in *Fig. A.1b*) might think that the signals change to amber for the opposing traffic at the same time. He might then try to 'clear' the intersection and run into an opposing vehicle to which signals would

(a) Leading turn
(late release of the opposing movement)



(b) Lagging turn
(early cut-off of the opposing movement)

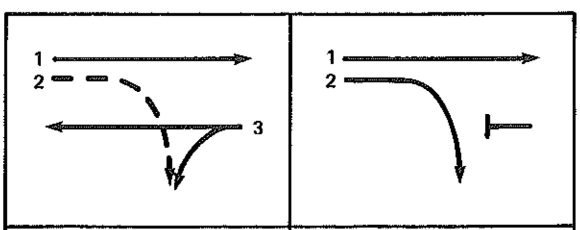


Fig. A.1 — Traditional leading and lagging turn phasing arrangements (Illustrated for a T-junction case)

still be green. This can be avoided by stopping through movements and allowing unopposed turns in both directions (a diamond turn arrangement), or by banning right turns from the opposite approach road.

More flexible and efficient solutions can be achieved with the modern group (or multi-phase) controllers using six-aspect signal displays (three circles and three arrows). The leading and lagging turn features of older controllers can be combined and variable phasing arrangements can be used to minimise time losses which may otherwise result from the changes in the flow pattern throughout the day. This helps to eliminate the need for opposed turns.

To understand how the variable phasing systems work, let us consider the intersection shown in *Fig. A.2* where all right-turning movements are separately controlled by exclusive lanes and three-aspect arrow signal displays. Assume that no opposed (filter) right turns are allowed, i.e. red arrows are shown during appropriate phases. The discussion will be presented in terms of movements 1, 2, 3 and 4 from North and South legs. The same applies to the four movements from East and West legs.

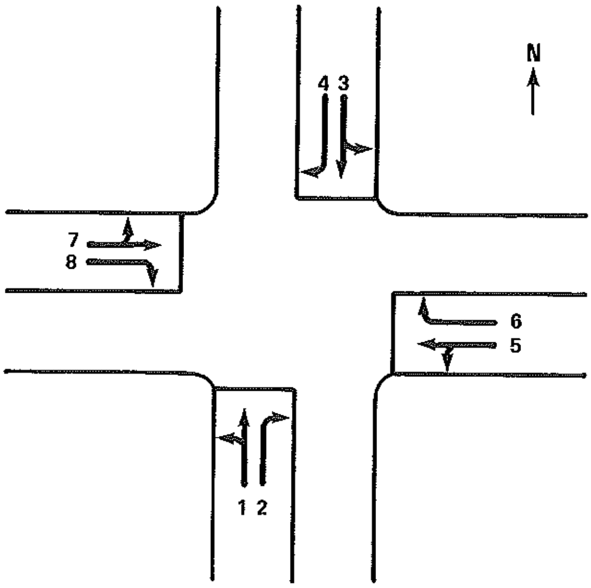
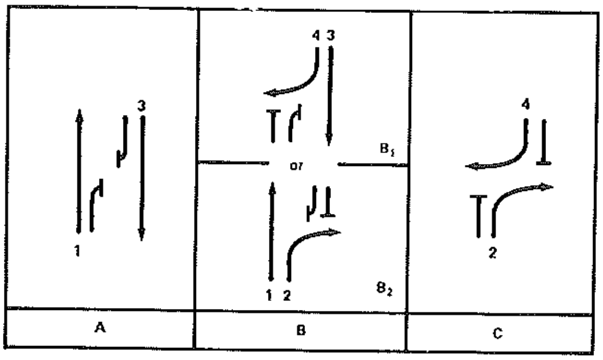


Fig. A.2 — A four-way intersection where all right-turning movements are controlled separately

A variable phasing arrangement is shown in *Fig. A.3a* which gives the sequence (A, B_1, C) or (A, B_2, C) depending on the turning and through movement flow levels. If the time required by movement 3 is longer than the time required by movement 1 ($t_3 > t_1$), the sequence (A, B_1, C) will result. If the time required by movement 1 is longer ($t_1 > t_3$), the sequence (A, B_2, C) will result. If the two movements require the same time ($t_1 = t_3$), the simple two-phase sequence (A, C) will be obtained as shown in *Fig. A.4.* The critical movement search diagrams (see Section 4) for these sequences are also given in *Figs A.3* and *A.4.* In these diagrams, phase D indicates the first phase for the movements from the East and West approach roads.

(a) Sequences A, B_1, C and A, B_2, C



(b) Critical movement search diagrams

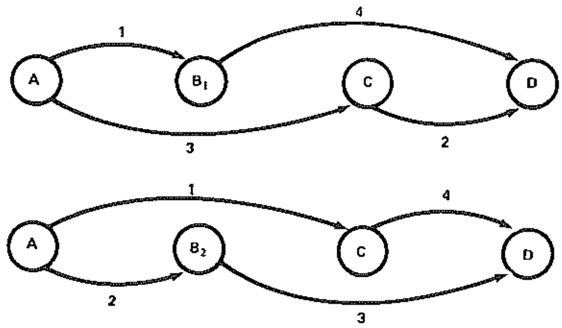


Fig. A.3 — A variable phasing arrangement

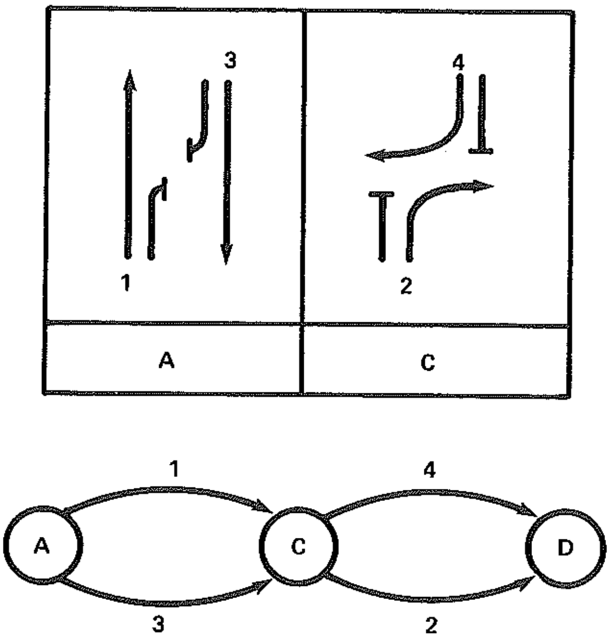
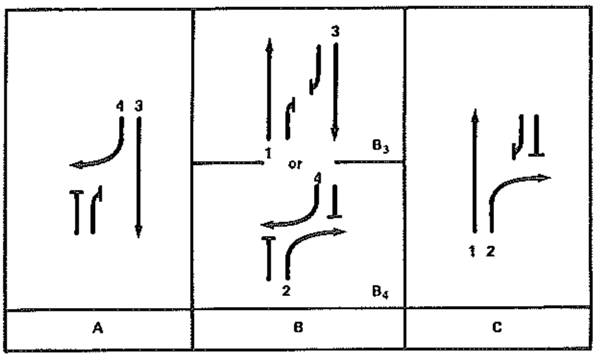


Fig. A.4 — The sequence obtained when movements 1 and 3 in Fig. A.3 require equal times

An alternative variable phasing arrangement is shown in *Fig. A.5*a. This gives the sequence (A, B_3, C) or (A, B_4, C) depending on how the times required by movements 3 and 4 compare. The simple two-phase sequence which may result from this arrangement (when $t_3 = t_4$) is shown in *Fig. A.6.* The critical movement search diagrams of these sequences are given in *Figs A.5* and *A.6.*

(a) Sequences A, B_3, C and A, B_4, C



(b) Critical movement search diagrams

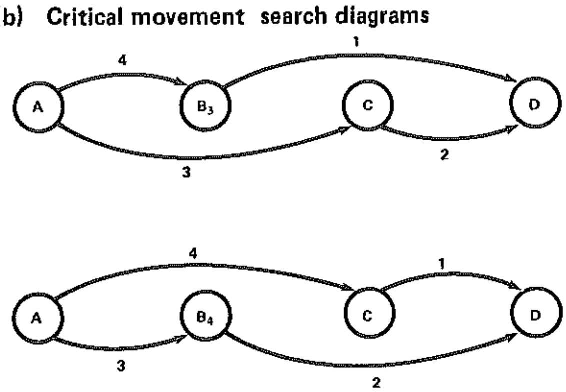


Fig. A.5 — A variable phasing arrangement

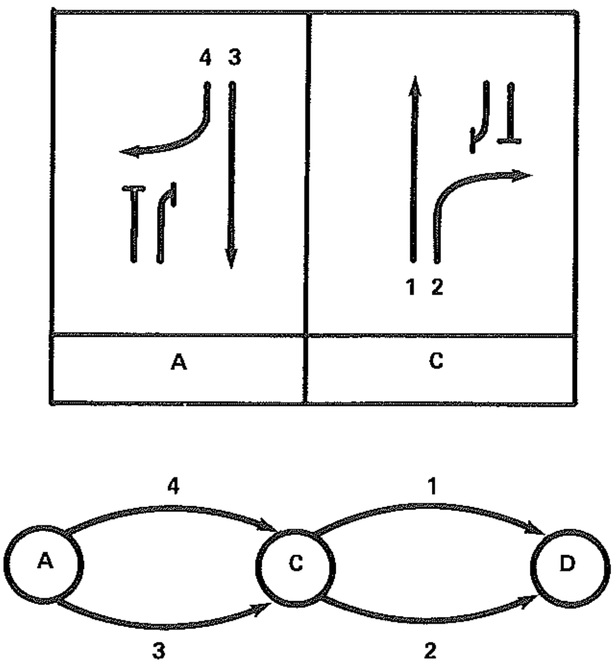


Fig. A.6 — The sequence obtained when movements 4 and 3 in Fig. A.5 require equal times

Considering the reverse phasing sequences (C, B_i, A) (i = 1 to 4), and (C, A), there is a total of twelve possible sequences. Theoretically, a fully-variable phasing system which allows for all possible twelve sequences is possible. However, because of the factors mentioned above for the design of a phasing system (driver understanding, uniformity, signal coordination constraints), a limited amount of variability may be preferable, e.g. the arrangement shown in *Fig. A.3* or *Fig. A.4*.

It can be seen from the critical movement search diagrams given in *Figs A.3* to *A.6* that, *independent of the phase sequence*, the critical movements are (1 and 4) or (3 and 2). The same is valid for the reverse sequences which are simply symmetrical. Therefore, a simple rule-of-thumb for critical movement identification in this type of *variable* phasing systems can be stated as 'the critical movements are the through (and combined left) movement and its opposing right-turn movement in one direction which has the highest total required movement time'. This rule was stated on the basis of 'critical lane flows' rather than 'critical movement times' by McInerney and Petersen (1971).

There are additional movement combinations which may produce longer time requirements when *fixed sequence* phasing systems are used. For example, the simple two-phase system shown in *Figs A.6* can have (3, 1) or (4, 2) or (4, 1) or (4, 2) as critical movements. Therefore, a variable phasing arrangement is more efficient considering possible variations in arrival flow patterns throughout the day, in particular morning peak-evening peak flow reversals. The operational efficiency of vehicle-actuated traffic signals could be significantly improved by controlling individual movements as shown in *Fig. A.2* to create variable phase sequences which can cope with changes in arrival flow patterns and rates throughout the day and from day to day.

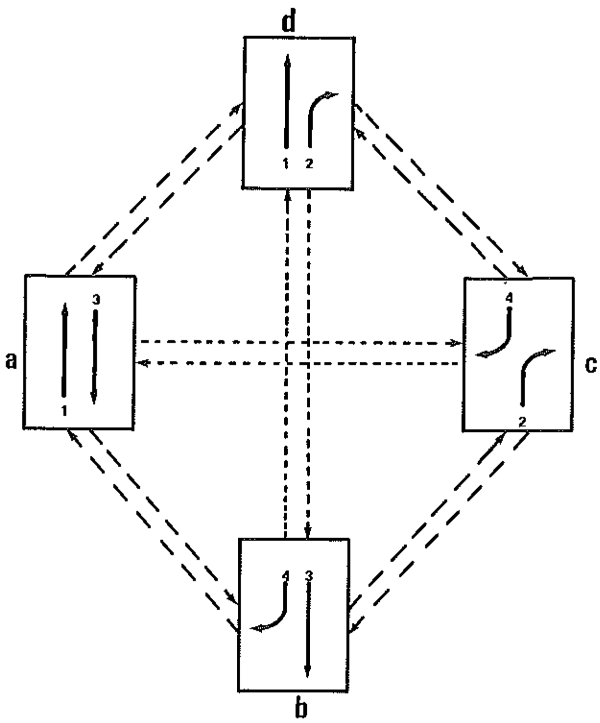


Fig. A.7 — Phase sequence generation diagram for movements from the North and South approach roads of the intersection shown in Fig. A.2

A diagram from which all possible phase sequences can be generated for movements 1, 2, 3 and 4 from North and South legs of the intersection shown in *Fig. A.2* is given in *Fig. A.7*. The twelve sequences considered above can be obtained by starting with phase a, b, c or d and proceeding along the dotted lines to the next two phases along the perimeter, or to the opposite phase to skip the intermediate phase, e.g. (a,b,c), (a,d,c), (a,c) and so on. Other sequences which were not considered above can be generated from *Fig. A.7*. These sequences involve two green periods per cycle for some movements. For example, the sequence (c,d,b) gives two green periods per cycle for movement 4. This solution is useful in practice when there is a limited queue storage space, or when the saturation flow falls off during a long green period (see Section 5).

A similar diagram can be prepared for movements 5, 6, 7 and 8 from the East and West approach roads and similar sequences can be obtained. Because the intersection shown in *Fig. A.2* does not give any overlaps between North-East and East-West movements, separate analyses are possible. Applying the general rule, the critical movements from East-West legs under variable sequence phasing arrangements are (5,8) or (6,7). Therefore, the critical movement combinations for the intersection as a whole are (1,4,5,8), (1,4,6,7), (2,3,5,8), (2,3,6,7) (see Example 5 in Appendix D).

The phase generation diagram given in *Fig. A.7* assumes that no filter turns are allowed during the through movement phase a. If filter turns are to be allowed during this phase, the sequences which can create a safety trap (associated with lagging turns as explained above) must be avoided. For example, the sequence (a,d,c) with filter turns allowed during phase a creates this problem. A preferable variable phasing sequence which can be used in this case is (c,(d or b),a). This corresponds to the reverse sequences (C,B_1,A) and (C,B_2,A) in *Fig. A.3*.

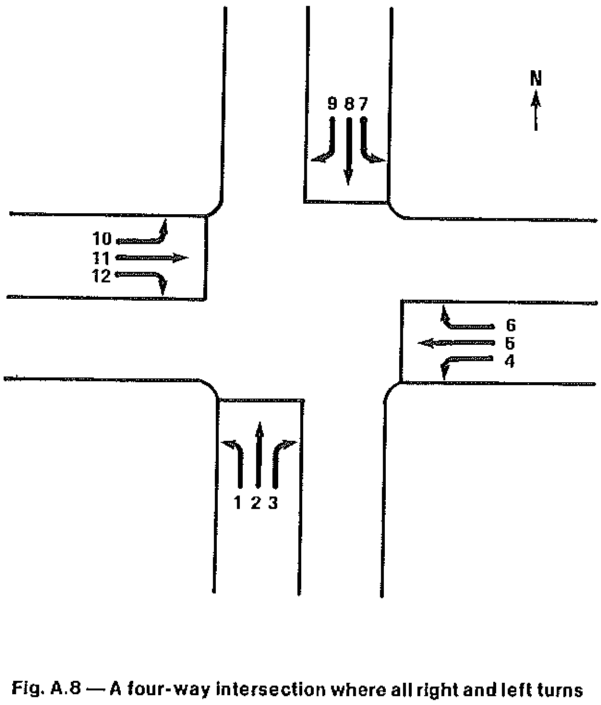


Fig. A.8 — A four-way intersection where all right and left turns are controlled separately

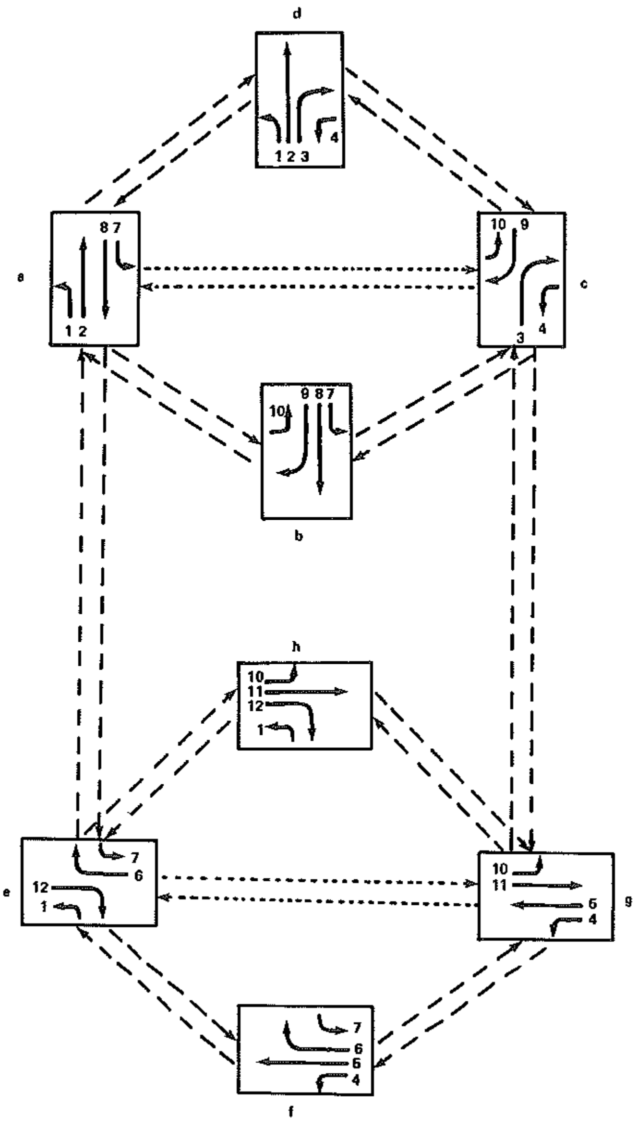


Fig. A.9 — Phase sequence generation diagram for the intersection shown in Fig. A.8

The above analysis can be extended to a case where the left-turning movements are also controlled separately as shown in *Fig. A.8*. In this case, there are overlaps between movements from the North-South and East-West legs of the intersection. This can be seen by generating various sequences from the diagram in *Fig. A.9*. The phase sequences which produce only one green period per cycle for all movements can be obtained by starting with a, c, e or g. These are (a,b,c), (a,d,c), (a,c) and their reverse sequences for the movements from the North-South legs, and (e,f,g), (e,h,g), (e,g) and their reverse sequences for the movements from the East-West legs. The complete phase sequence for the intersection is obtained by combining the North-South and East-West sequences. It can be shown that for all such sequences, the critical movement possibilities are:

(2,9,5,12), (2,9,11,6), (8,3,5,12), (8,3,11,6), (1,5,9), (2,6,10), (3,7,11), (4,8,12).

The first four are the possible combinations related to through and right turn movements as obtained by applying the general rule-of-thumb given above, and the last four are the additional possibilities related to separate control of left-turn movements. An example is given in *Fig. A.10* for the sequence (c,d,a,e,h,g), together with the corresponding critical movement search diagram.

The discussion in this Appendix has been presented in order to indicate the basic issues and some possibilities in the choice of signal phasing systems, and to demonstrate the applicability of the critical movement identification technique (described in Section 4) to complicated phasing systems. This should be understood as a discussory text rather than one which tries to describe a complete phasing design methodology, although some conclusions are useful towards this end. For further reading on this subject, the reader is referred to Stoffers (1968); Messer *et al.* (1973); Whitson and Carwell (1974); Courage *et al.* (1974); Bang and Nilsson (1976*b*); U.S. Department of Transportation (1976 and 1978); and Zuzarte Tully (1977).

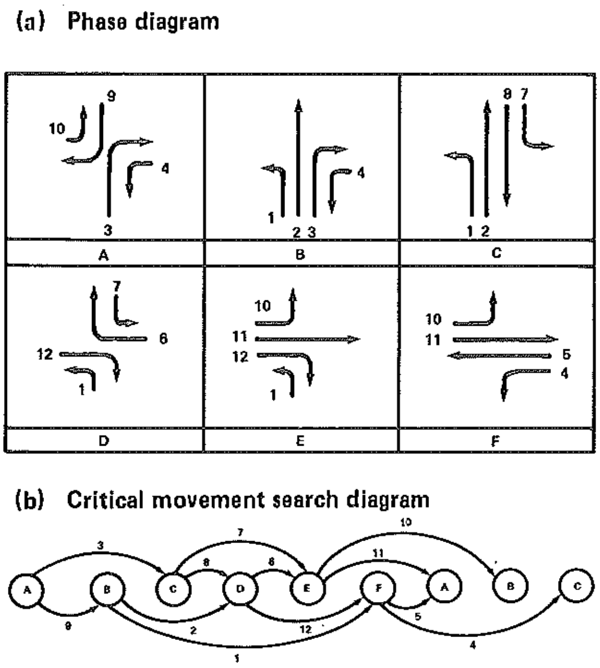


Fig. A.10 — Phase sequence c,d,a,e,h,g from Fig. A.9

APPENDIX B

MINIMUM GREEN TIME CALCULATIONS

The importance of minimum green times in critical movement identification and subsequent capacity and timing calculations is discussed in Sections 4 and 7 (see also the examples in Appendix D). The determination of displayed and effective minimum green times is discussed below.

The value of minimum displayed green time, G_m, is determined by vehicle or pedestrian requirements. Vehicle minimum green times may be chosen in the range from 6 to 10 s. For the purposes of critical movement identification and timing calculations, G_m for a pedestrian movement is the sum of the walk and clearance periods. Using a minimum walk period of 6 s, G_m can be calculated from

$$G_m = 6 + D/1.2 \qquad \text{(B.1)}$$

where D is the kerb-to-kerb crossing distance (m) and 1.2 m/s is the pedestrian walking speed during clearance. If the pedestrian clearance time overlaps with the intergreen time of the terminating phase, the value of G_m should be reduced by the amount of overlap.

If the minimum displayed green times are to be specified for movements rather than phases, as for the group controllers, the values of G_m calculated as described above can be used directly. However, if the minimum green times are specified for phases rather than movements, then the minimum green time for an overlap movement can be calculated to include the intermediate intergreen times (use eqn (2.4)). For the purpose of calculations, a zero phase minimum green time ($G_m = 0$) can be specified for a phase which has not a non-overlap movement in it. This would decrease the lost time when there is no demand for this phase. When the minimum green times are specified for movements rather than phases, this is automatically achieved.

The effective minimum green time for use in the capacity and timing calculations can be calculated from $g_m = G_m + I - \ell$ (see eqn (2.3)).

Example:

In the example given in Section 4 (also see Appendix C), G_m values for movements 6 and 7 were calculated for crossing distances of 12 m and 15 m, respectively, as shown in *Fig. B.1*, and assuming 2 s overlap with the intergreen time of the terminating phase:

$G_{m6} = 6 + 12/1.2 - 2 = 14$ s

$G_{m7} = 6 + 15/1.2 - 2 = 17$ s

For both movements, $I = 5$, $\ell = 4$ (see *Table C.2*), and from eqn (4.5), the required minimum movement times ($t_m = G_m + I$) and the minimum effective green times ($g_m = t_m - \ell$) are:

$$t_{m6} = 14 + 5 = 19, \quad g_{m6} = 19 - 4 = 15, \text{ and}$$

$$t_{m7} = 17 + 5 = 22, \quad g_{m7} = 22 - 4 = 18$$

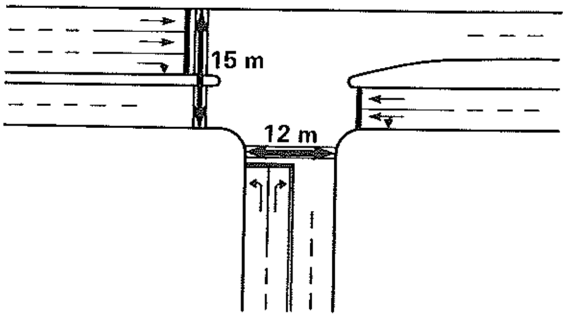


Fig. B.1 — Pedestrian crossing distances for the example in Section 4

In *Fig. B.2*, walk and clearance times for pedestrian movement 7 are shown in relation to the starting and terminating phase times (see Section 4 and Appendix C for the total intersection example).

It should be noted that the above definition of 'green time' can be changed for the purposes of calculating delays etc. (see Section 6.5) for pedestrian movements. For this purpose, the walk period only could be considered as 'green'. This can be achieved by describing the clearance period as negative end gain (excluding the part which overlaps with vehicle intergreen period), e.g. for movement 7 in *Fig. B.2*, end gain, $b = -11$. Also assuming a start loss of 1 s to give a start lag of $a = 6$ s, lost time would be $\ell_7 = a - b = 6 - (-11) = 17$, and the resulting minimum effective green time $g_{m7} = 22 - 17 = 5$ (this method has not been used in the Example in Section 4).

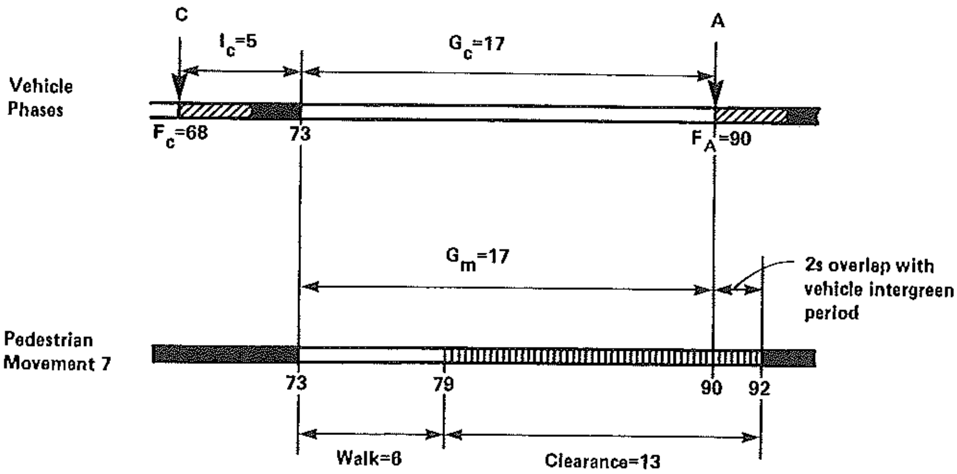


Fig. B.2 — Walk and clearance periods for pedestrian movement 7 in the example in Section 4

APPENDIX C

ADDITIONAL DATA FOR THE EXAMPLES IN SECTIONS 2 AND 4

Additional data for the examples in Sections 2 and 4 are given in *Tables C.1* and *C.2*. The details of green time calculations for the example in Section 4 are given below.

As shown in Steps 1 to 8 of the critical movement identification and signal timing calculation procedure described in Section 4, the chosen cycle time is c = 90 s, the critical movements are 3 and 4 with u_3 = 0.33, u_4 = 0.52, ℓ_3 = 4, ℓ_4 = 8 (see *Table 4.1*). Therefore, U = 0.85 and L = 12 for the intersection. Let us calculate green times using the method given in Section 7.

(a) Critical Movement Green Times

The total available effective green time is $c - L$ = 90 − 12 = 78, which will be distributed to critical movements 3 and 4 (eqn (7.5)):

$$g_3 = \frac{78}{0.85}\,0.33 = 30.3 \approx 30,$$

$$g_4 = \frac{78}{0.85}\,0.52 = 47.7 \approx 48.$$

To check, $g_3 + g_4 = 30 + 48 = 78 = c - L$.

(b) Non-Critical Movement Green Times

As seen in the critical movement search diagram (*Fig. 4.2*), the non-overlap movement 6 shares the same phase with the critical movement 3. Therefore, from eqn (7.7):

$$g_6 = (g_3 + \ell_3) - \ell_6 = 34 - 4 = 30$$

is determined. However, the calculation of other green times is more complicated because of multiple overlaps in this example. It can be seen from *Fig. 4.2* that the critical movement 4 overlaps with movements 7 (phase C) and 2 (phase A). Therefore, its time can be treated as a sub-cycle time $c^* = g_4 + \ell_4$ = 56, and distributed to these two movements. Because the time for movement 7 will be determined by its minimum time requirement (see *Table 4.1*), $t_7 = t_{m7}$ = 22, the values for use in eqn (7.8) are

$$L^* = \ell_2 + t_{m7} = 5 + 22 = 27$$

and $U^* = u_2$, and hence (eqn (7.9)):

$$g_2 = c^* - L^* = 56 - 27 = 29$$

$$g_7 = g_{m7} = 18$$

TABLE C.1

ADDITIONAL DATA FOR THE EXAMPLE IN SECTION 2

(a) Movement Data (see Fig. 2.4)

Movement	*Phase*	*Inter-green* I	*Start Loss*	*Start Lag* a	*End Lag* b	*Lost Time* $\ell_i = a - b$	*Eff. Green* g	*Movement Time* $t = g + \ell$
1	A,B	6	2	8	2	6	69	75
2	A	6	3	9	4	5	27	32
3	B	4	1	5	3	2	41	43
4	C	5	2	7	4	3	22	25

(b) Phase Data (see Figs 2.1, 2.2 and 2.4)

Phase	*Inter-green* I	*Displayed Green* G	*Change Time* F	*Green Start* $F+I$	*Green End* $F+I+G$
A	6	26	0(=100)	6	32
B	4	39	32	36	75
C	5	20	75	80	100(=0)

Cycle time, $c = \Sigma I + \Sigma G = 15 + 85 = 100$

As a result of determining the green times for movements 7 and 2 the green times for movements 5 and 1 are automatically determined. From eqn (7.7),

$$g_5 = g_7 + \ell_7 - \ell_5 = 22 - 3 = 19.,$$

From eqn (7.10),

$$g_1 = g_1 + \ell_2 + g_3 + \ell_3 - \ell_1 = 34 + 34 - 6 = 62.$$

(c) Phase Green Time and Change Time Calculations

The above calculations are best carried out by entering the values determined in a Movement Data table as shown in *Table C.2(a)*. Then the phase green time calculations can be carried out referring to this table and using a Phase Data table as shown in *Table C.2(b)*. The easiest method may be to determine the displayed green time for each movement from $G = g + \ell - I$, and assign the non-overlap movement values to the corresponding phases. In this example, $G_A = G_2$, $G_B = G_3 = G_6$ and $G_c = G_5 = G_7$. The calculation of phase change times from eqn (7.12) for this example is shown in *Table C.2(b)*. The corresponding cycle diagram is given in *Fig. C.1*.

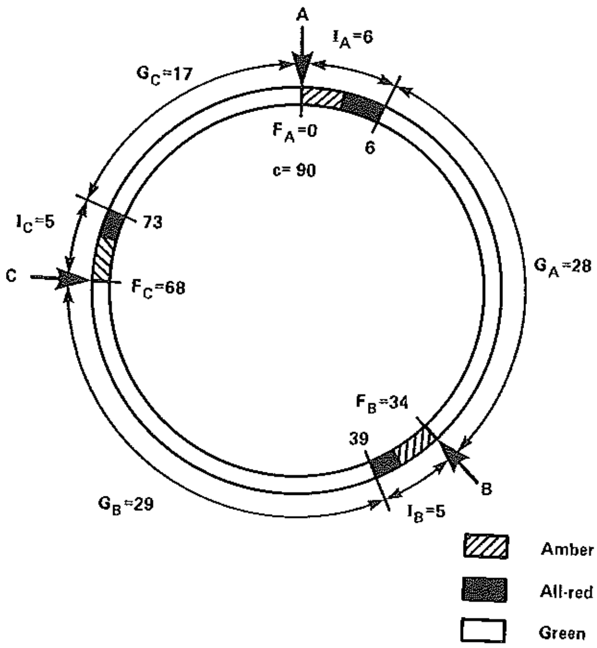


Fig. C.1 — Cycle diagram for the example in Section 4 (Table C.2b)

TABLE C.2

ADDITIONAL DATA FOR THE EXAMPLE IN SECTION 4

(a) Movement Data

Movement	*Phase*	*Inter-green* I	*Eff. green* g	*Start Loss*	*End Gain* b	*Start Lag*	*Lost Time* $\ell = a - b$	*Movement Time* $t = g + \ell$	*Displayed Green* $G = g + \ell - I$
1	A,B	6	62	2	2	8	6	68	62
2	A	6	29	3	4	9	5	34	28
3	B	5	30	2	3	7	4	34	29
4	C,A	5	48	6	3	11	8	56	51
5	C	5	19	2	4	7	3	22	17
6	B	5	30	1	2	6	4	34	29
7	C	5	18	1	2	6	4	22	17

(b) Phase Data

Phase	*Inter-green* I	*Displayed Green* G	*Change Time* F	*Green Start* $F + I$	*Green End* $F + I + G$
A	6	28	0(=90)	6	34
B	5	29	34	39	68
C	5	17	68	73	90(=0)

Cycle time, $c = \Sigma I + \Sigma G = 16 + 74 = 90$

APPENDIX D

FURTHER EXAMPLES FOR CRITICAL MOVEMENT IDENTIFICATION AND SIGNAL TIMING CALCULATIONS

D.1 EXAMPLE 1

This is a variation to the example given in Section 4 (*Figs 4.1* and *4.2*, *Table 4.1*) to illustrate the effect of changes in arrival flow pattern during the day. Assume that the example in Section 4 represents the morning peak flow pattern and that the evening peak flow pattern is an exact reversal of the morning peak pattern obtained by interchanging Movement 1 flow rate with Movement 3 flow rate and Movement 2 flow rate with Movement 4 flow rate. Assume that all other data are the same as in the example in Section 4 except for the practical degrees of saturation (x_p) which are chosen with the intention to favour heavier flows (Movement 1 in particular).

The data and calculations are given in *Table D.1*. As in the example in Section 4, the critical movements are (1,7) or (2,3,7) or (3,4). The required times are $T_{1,7} = 37 + 22 = 59$, $T_{2,3,7} = 47 + 26 + 22 = 95$, $T_{3,4} = 26 + 29 = 45$. Therefore, (2,3,7) are the critical movements in this case. Note that Movement 7 is a pedestrian movement whose minimum green time determines the required time. The intersection parameters are $L = \ell_2 + \ell_3 + \ell_7 = 5 + 4 + 22 = 31$ s, $Y = y_2 + y_3 = 0.38 + 0.20 = 0.58$ and $U = u_2 + u_3 = 0.42 + 0.22 = 0.64$. The practical cycle time (eqn (7.2)) is $c_p = 31/(1 - 0.64) = 86$ s, and the approximate optimum cycle time with a stop penalty parameter of $k = 0.2$ (eqn (7.1)) is $c_o = (1.6 \times 31 + 6)/(1 - 0.58) = 132$ s. The cycle time chosen in the range from c_p to c_o for this example is $c = 110$ s. The calculations necessary to check the validity of critical movements with this cycle time are included in *Table D.1b*.

The green times are calculated as follows (refer to *Fig. 4.2* for overlap movement green time calculations):

$c - L = 110 - 31 = 79$,
$g_2 = (79/0.64)\ 0.42 = 52$,
$g_3 = (79/0.64)\ 0.22 = 27$,
$g_7 = 18\ (= g_{m7})$,
$g_1 = (g_2 + \ell_2) + (g_3 + \ell_3) - \ell_1 = (52 + 5) + (27 + 4) - 6 = 57 + 31 - 6 = 82$,
$g_4 = (g_7 + \ell_7) + (g_2 + \ell_2) - \ell_4 = (18 + 4) + (57) - 8 = 22 + 57 - 8 = 71$,
$g_6 = (g_3 + \ell_3) - \ell_6 = (31) - 4 = 27$,
$g_5 = (g_7 + \ell_7) - \ell_5 = (22) - 3 = 19$.

These green times and the resulting degrees of saturation (x) are given in *Table D.1b*. The corresponding phase green times are $G_A = G_2 = g_2 + \ell_2 - I_A = 52 + 5 - 6 = 51$, $G_B = G_3 = g_3 + \ell_3 - I_B = 27 + 4 - 5 = 26$ and $G_C = G_5 = g_5 + \ell_5 - I_C = 19 + 3 - 5 = 17$. To check, $c = \Sigma G + \Sigma I = (51 + 26 + 17) + (6 + 5 + 5) = 110$.

D.2 EXAMPLE 2

This is based on an example by Webster and Cobbe (1966, Fig. 34). The movements and the signal phasing diagram are shown in *Figs D.1a* and *b*. The phasing diagram in *Fig. D.1b* is a condensed form of the original diagram given by Webster and Cobbe in that some 'stages' are treated as part of intergreen or combined together as a single phase in order to make the solution of the problem easier. It is assumed that opposed right turns are not allowed during Phase A so that data given by Webster and Cobbe can be used without modification (otherwise Movement 2 must be treated as one with two saturation flows per cycle as explained in Appendix F.2).

The data and calculations are given in *Table D.2*. Note that all movement lost times are calculated as $\ell = I - 1$. To allow comparison of the results with those of Webster and Cobbe, the same practical degree of saturation ($x_p = 0.90$) and the same minimum green time ($G_m = 7$) are used for all movements. These parameters are not used in Webster and Cobbe's method.

The critical movement search diagram is given in *Fig. D.1c*. Movement 6 is deleted since $t_6 < t_5$, i.e. Movement 5 is representative of Phase C. Possible critical movement combinations are (2,5,7), (1,3,5,7) and (1,4,7). The total required movement times are:

$T_{2,5,7} = 36 + 23 + 14 = 73$,
$T_{1,3,5,7} = 25 + 25 + 23 + 14 = 87$, and
$T_{1,4,7} = 25 + 52 + 14 = 91$.

Therefore, the critical movements are (1,4,7) and the intersection parameters are:

$L = \ell_1 + \ell_4 + \ell_7 = 3 + 8 + 3 = 14$ s,
$Y = y_1 + y_4 + y_7 = 0.20 + 0.40 + 0.10 = 0.70$, and
$U = u_1 + u_4 + u_7 = 0.22 + 0.44 + 0.11 = 0.77$.

The practical cycle time (eqn (7.2)) is $c_p = 14/(1 - 0.77) = 61$ s, and the approximate optimum cycle time for minimum delay ($k = 0$ in eqn (7.1)) is $c_o = (1.4 \times 14 + 6)/(1 - 0.70) = 85$ s. A cycle time of $c = 70$ s is chosen. The critical movements are valid with this cycle time as indicated by the calculations in *Table D.1b*. The total available effective green time is $c - L = 70 - 14 = 56$ and critical movement green times are:

$g_1 = (56/0.70)0.20 = 16$,
$g_4 = (56/0.70)0.40 = 32$,
$g_7 = (56/0.70)0.10 = 8$.

Non-critical movement green times are calculated as follows (see *Fig. D.1c*):

$c^* = g_4 + \ell_4 = 32 + 8 = 40$,
$L^* = \ell_3 + \ell_5 = 8 + 6 = 14$,
$c^* - L^* = 40 - 14 = 26$,
Since $y_3 = y_5$, $g_3 = g_5 = (c^* - L^*)/2 = 13$,
$g_2 = (g_1 + \ell_1) + (g_3 + \ell_3) - \ell_2 = (16 + 3) + (13 + 8) - 3 = 37$,
$g_6 = g_5 = 13$.

The movement degrees of saturation (x) calculated using these green times are given in *Table D.1b*. Note that equal degrees of saturation are obtained for the critical movements, $x_1 = x_4 = x_7 = 0.88$. This is the intersection degree of saturation which could be calculated without calculating the movement green times (eqn (3.12)): $X = (70/56)0.70 = 0.88$.

Because $\ell = I - 1$ was assumed for all movements, phase green times can be calculated from $G = g + \ell - I = g - 1$. Therefore, $G_A = g_1 - 1 = 16 - 1 = 15$, $G_B = g_3 - 1 = 13 - 1 = 12$, $G_C = g_5 - 1 = 13 - 1 = 12$, and $G_D = g_7 - 1 = 8 - 1 = 7$. To check, $c = \Sigma G + \Sigma I = (15 + 12 + 12 + 7) = (4 + 9 + 7 + 4) = 46 + 24 = 70$.

TABLE D.1

CRITICAL MOVEMENT SEARCH TABLE FOR EXAMPLE 1

(a) DATA

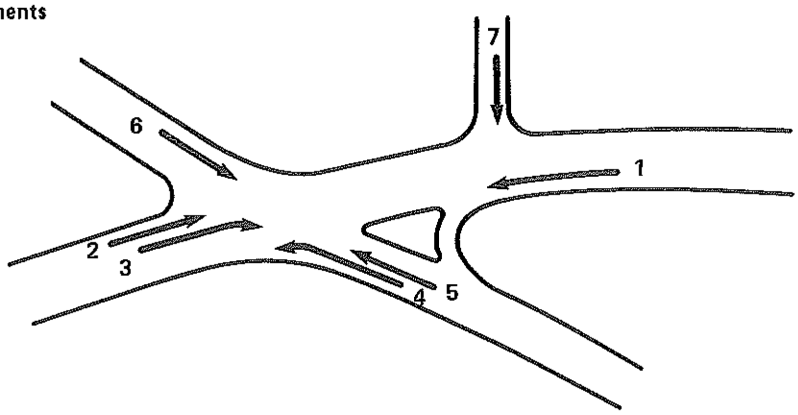
Movement	Starting Phase	Termin. Phase	Intergreen Time (I)	Min.Disp. Green (G_m)	Arrival Flow (q)	Satur. Flow (s)	Lost Time (ℓ)	Min.Eff. Green (g_m)	Prac. Deg.Sat. (x_p)
1	*A*	*C*	*6*	*8*	*920*	*3480*	*6*	*8*	*0.85*
2	*A*	*B*	*6*	*6*	*580*	*1510*	*5*	*7*	*0.90*
3	*B*	*C*	*5*	*8*	*650*	*3260*	*4*	*9*	*0.90*
4	*C*	*B*	*5*	*8*	*240*	*1240*	*8*	*5*	*0.92*
5	*C*	*A*	*5*	*6*	*170*	*1490*	*3*	*8*	*0.92*
6	*B*	*C*	*5*	*14*	*Pedestrians*		~~4~~ *19*	*15*	–
7	*C*	*A*	*5*	*17*	*Pedestrians*		~~4~~ *22*	*18*	–

(b) CALCULATIONS

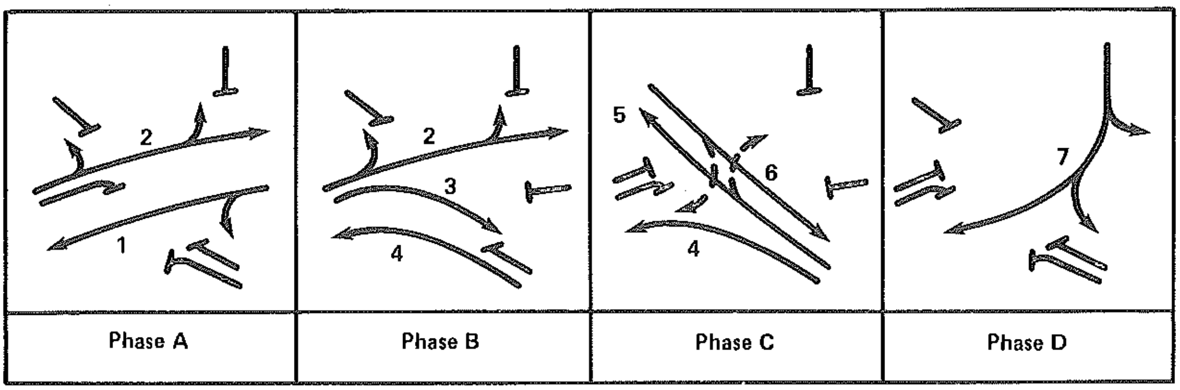
Movement	$y = q/s$	$u = y/x_p$	$100\,u+\ell$	$t_m = G_m + I$	t	Check for c = *110*			
						$uc + \ell$	t'	g	$x = (c/g)\,y$
1	*0.26*	*0.31*	*37*	*14*	*37*	*40*	*40*	*82*	*0.35*
2 *	*0.38*	*0.42*	*47*	*12*	*47*	*51*	*51*	*52*	*0.80*
3 *	*0.20*	*0.22*	*26*	*13*	*26*	*28*	*28*	*27*	*0.81*
4	*0.19*	*0.21*	*29*	*13*	*29*	*31*	*31*	*71*	*0.29*
5	*0.11*	*0.12*	*15*	*11*	*15*	*16*	*16*	*19*	*0.64*
6	–	–	–	*19*	*19*	–	*19*	*27*	–
7 *	–	–	–	*22*	*22*	–	*22*	*18*	–

* Critical movements

(a) Movements



(b) Phasing



(c) Critical movement search

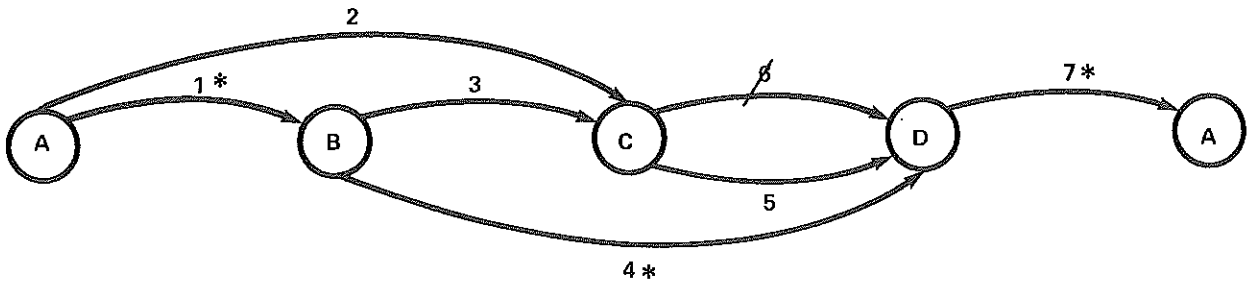


Fig. D.1 — Example 2 (after Webster and Cobbe (1966))

TABLE D.2

CRITICAL MOVEMENT SEARCH TABLE FOR EXAMPLE 2

(a) DATA

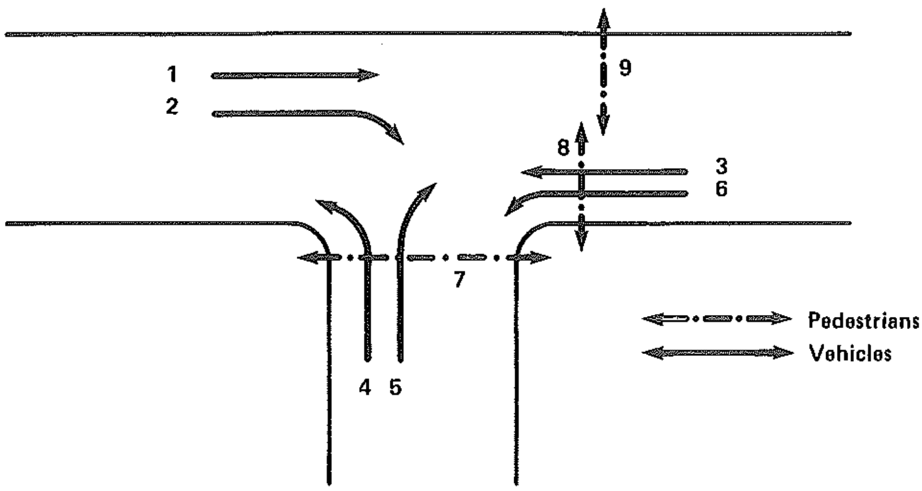
Movement	Starting Phase	Termin. Phase	Intergreen Time (I)	Min.Disp. Green (G_m)	Arrival Flow (q)	Satur. Flow (s)	Lost Time (ℓ)	Min.Eff. Green (g_m)	Prac. Deg.Sat. (x_p)
1	A	B	4	7	640	3200	3	8	0.90
2	A	C	4	7	900	3000	3	8	0.90
3	B	C	9	7	225	1500	8	8	0.90
4	B	D	9	7	640	1600	8	8	0.90
5	C	D	7	7	225	1500	6	8	0.90
6	C	D	7	7	300	3000	6	8	0.90
7	D	A	4	7	160	1600	3	8	0.90

(b) CALCULATIONS

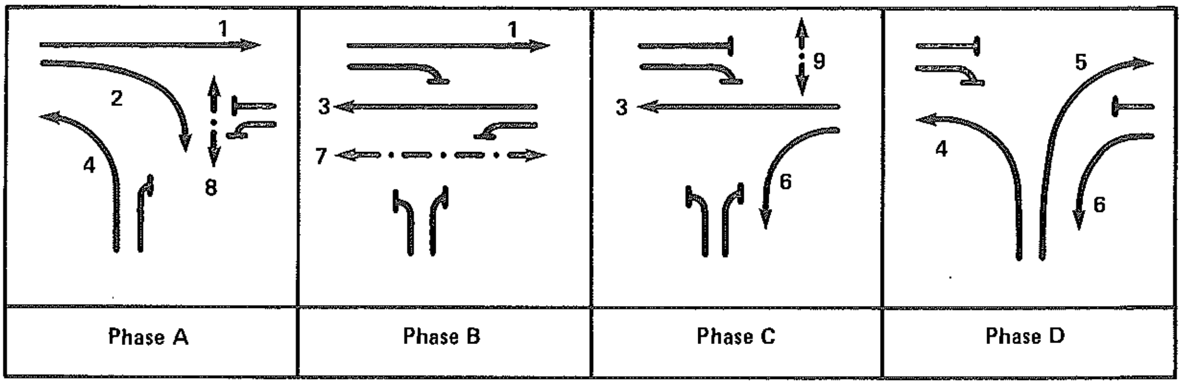
Movement	y = q/s.	u = y/x_p	100 u+ℓ	$t_m = G_m + I$	t	Check for c = 70			
						uc + ℓ	t'	g	x = (c/g) y
1 *	0.20	0.22	25	11	25	18	18	16	0.88
2	0.30	0.33	36	11	36	26	26	37	0.57
3	0.15	0.17	25	16	25	20	20	13	0.81
4 *	0.40	0.44	52	16	52	39	39	32	0.88
5	0.15	0.17	23	14	23	18	18	13	0.81
6	0.10	0.11	17	14	17	14	14	13	0.54
7 *	0.10	0.11	14	11	14	11	11	8	0.88

* Critical movements

(a) Movements



(b) Phasing



(c) Critical movement search

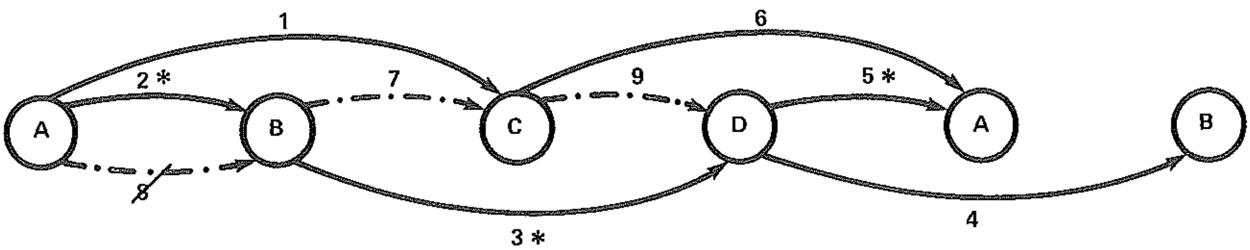


Fig. D.2 — Example 3 (after Allsop and Murchland (1978))

TABLE D.3

CRITICAL MOVEMENT SEARCH TABLE FOR EXAMPLE 3

(a) DATA

Movement	Starting Phase	Termin. Phase	Intergreen Time (I)	Min.Disp. Green (G_m)	Arrival Flow (q)	Satur. Flow (s)	Lost Time (ℓ)	Min.Eff. Green (g_m)	Prac. Deg.Sat. (x_p)
1	*A*	*C*	4	8	850	3600	4	8	0.90
2	*A*	*B*	4	8	250	1800	4	8	0.90
3	*B*	*D*	4	8	1600	4000	4	8	0.95
4	*D*	*B*	4	8	1100	3200	4	8	0.85
5	*D*	*A*	4	8	250	1800	4	8	0.90
6	*C*	*A*	4	8	600	1800	4	8	0.95
7	*B*	*C*	4	8	*Pedestrians*		~~4~~ 12	8	–
8	*A*	*B*	4	8	*Pedestrians*		~~4~~ 12	8	–
9	*C*	*D*	4	8	*Pedestrians*		~~4~~ 12	8	–

(b) CALCULATIONS

Movement	y = q/s	u = y/x_p	100 u+ℓ	t_m = G_m + I	t	Check for c = ~~45~~ 60			
						uc + ℓ	t'	g	x = (c/g) y
1	0.24	0.27	31	12	31	~~16~~ 20	~~16~~ 20	26	0.55
2 *	0.14	0.16	20	12	20	~~11~~ 14	~~12~~ 14	10	0.84
3 *	0.40	0.42	46	12	46	~~23~~ 29	~~23~~ 29	28	0.86
4(*)	0.34	0.40	44	12	44	~~22~~ 28	~~22~~ 28	24	0.85
5 *	0.14	0.16	20	12	20	~~11~~ 14	~~12~~ 14	10	0.84
6	0.33	0.35	39	12	39	~~20~~ 25	~~20~~ 25	26	0.76
7	–	–	–	12	12	–	12	12	–
8	–	–	–	12	12	–	12	10	–
9	–	–	–	12	12	–	12	12	–

* Critical movements

D.3 EXAMPLE 3

This is taken from Allsop and Murchland (1978). The movements and signal phasing are shown in *Figs D.2a* and *b*. This example is similar to the Example in Section 4. The differences are that the left turns into the arm of the T junction are separately controlled and the pedestrians crossing the main road are defined as two separate movements (8 and 9) due to the staged crossing arrangement. The phasing in *Fig. D.2b* corresponds to Allsop and Murchland's 'sequence 1,4,12,6'. The data and calculations are given in *Table D.3.*

As seen from the critical movement search diagram in *Fig. D.2c*, possible critical movement combinations are (1,6), (2,7,9,5), (1,9,5), (2,7,6), (2,3,5) and (3,4). The required times are:

$T_{1,6} = 31 + 39 = 70$,
$T_{2,7,9,5} = 20 + 12 + 23 + 20 = 64$,
$T_{1,9,5} = 31 + 12 + 20 = 63$,
$T_{2,7,6} = 20 + 12 + 39 = 71$,
$T_{2,3,5} = 20 + 46 + 20 = 86$,
$T_{3,4} = 46 + 44 = 90$.

Therefore, the critical movements are 3 and 4, and:

$L = \ell_3 + \ell_4 = 2 \times 4 = 8$,
$Y = y_3 + y_4 = 0.40 + 0.34 = 0.74$,
$U = u_3 + u_4 = 0.42 + 0.40 = 0.82$.

The practical and approximate delay-minimising cycle times are found $c_p = 8/(1 - 0.82) = 44$ and $c_o = (1.4 \times 8 + 6)/(1 - 0.74) = 66$.

This example presents a marginal case where different critical movements can be obtained for small cycle times. This is because the required times for movements 2 and 5 will be determined by their minimum green times ($g_m = 8$) under a small cycle time. For example, if c = 45 s is chosen and the required movement times (t') are re-calculated (crossed values in *Table D.3b*), (2,7,9,5) are found as critical movements since $T_{2,7,9,5} = 4 \times 12 = 48$, $T_{2,3,5} = 47$, $T_{3,4} = 45$, and so on. The practical cycle time in this case is the absolute minimum cycle time given by eqn (7.3): $c_m = 4(8 + 4) = 48$ s.

Let us choose a cycle time of c = 60 s and check the critical movements again. For this cycle time, the same total required times, $T_{2,3,5} = 14 + 29 + 14 = 57$ and $T_{3,4} = 29 + 28 = 57$ are found. It can be said that movements (2,3,5) and (3,4) are equally critical. The use of either set of critical movements make little (or theoretically no) difference in results. The green times for $c = 60$ and using (2,3,5) as the critical movements are calculated as follows:

$L = \ell_2 + \ell_3 + \ell_5 = 3 \times 4 = 12$,
$U = u_2 + u_3 + u_5 = 0.16 + 0.42 + 0.16 = 0.74$,
$c - L = 60 - 12 = 48$,
$g_2 = (48/0.74)0.16 = 10.4$,
$g_3 = (48/0.74)0.42 = 27.2$,
$g_5 = (48/0.74)0.16 = 10.4$.

Approximating to the nearest second, $g_2 = g_5 = 10$ and $g_3 = 28$ are found. The green times for other movements are (refer to *Fig. D.2c*):

$g_4 = (g_5 + \ell_5) + (g_2 + \ell_2) - \ell_4 = 14 + 14 - 4 = 24$,

to find g_7 and g_9: the total available green time is $(g_3 + \ell_3) - (\ell_7 + \ell_9) = (28 + 4) - (2 \times 4) = 24$, since both movement times are determined by minimum green times (both $t_m = 12$), divide the available green time equally: $g_7 = g_9 = 24/2 = 12$, therefore:

$g_1 = (g_2 + \ell_2) + (g_7 + \ell_7) - \ell_1 = 14 + 16 - 4 = 26$.
$g_6 = (g_9 + \ell_9) + (g_5 + \ell_5) - \ell_6 = 16 + 14 - 4 = 26$,
also note that $g_8 = g_2 = 10$.

The green times calculated above are shown in *Table D.3b*, together with the corresponding degrees of saturation. It is seen that all degrees of saturation are equal to or less than the specified maximum aceptable values (x_p in *Table D.3a*). Phase green times are easily found since $I = \ell$ is assumed for all movements:

$G_A = g_2 = 10$
$G_B = g_7 = 12$
$G_C = g_9 = 12$
$G_D = g_5 = 10$

To check, $c = \Sigma G + \Sigma I = (10 + 12 + 12 + 10) + (4 \times 4) = 44 + 16 = 60$.

D.4 EXAMPLE 4

This example is found in Zuzarte Tully and Murchland (1977). The movements at a complex intersection (Newcastle, U.K.) are shown in *Fig. D.3.* Movement 9 represents pedestrians. Three different phasing arrangements (Cases (a) to (c)) and the corresponding critical movement search diagrams are shown in *Figs D.4* to *D.6* (these cases correspond to 'phase sequences (3,8,6), (3,8,7,6) and (1,3,7,8)', respectively, in the original publication). The data which apply to all cases are given in *Table D.4.* Calculations for critical movement identification and results are given in *Tables D.5* to *D.7.* Each case is discussed separately below.

Case (a)

Possible critical movement combinations are found as (5,9,7) and (9,6) from *Fig. D.4.* Critical movements are (5,9,7) and the intersection parameters are $L = \ell_5 + \ell_9 + \ell_7 = 4 + 17 + 4 = 25$, $Y = y_5 + y_7 = 0.29 + 0.28 = 0.57$, $U = u_5 + u_7 = 0.32 + 0.31 = 0.63$. The practical and approximate minimum-delay cycle times are $c_p = 68$ and $c_o = 95$. Green times for the chosen cycle time of c = 80 are given in *Table D.5.* Phase green times for this case are $G_A = g_5 = 28$, $G_B = g_1 = 13$ and $G_C = g_7 = 27$. To check $\Sigma G + \Sigma I = (28 + 13 + 27) + 3 \times 4 = 80 = c$).

Case (b)

From *Fig. D.5*, possible critical movement combinations are found as (5,1,7), (5,9,4) and (9,6). Critical movements in this case are (5,1,7) and the intersection parameters are $L = \ell_5 + \ell_1 + \ell_7 = 4 + 12 + 4 = 20$, $Y = y_5 + y_7 = 0.57$ and $U = u_5 + u_7 = 0.63$ (see *Table D.6*). The practical and approximate minimum-delay cycle times are $c_p = 54$ and $c_o = 79$. The green times for the chosen cycle time of $c = 60$ are given in *Table D.6.* Phase green times are $G_A = g_5 = 20$, $G_B = g_1 = 8$, $G_c = g_7 - g_4 - \ell_4 = 20 - 15 - 4 = 1$ and $G_D = g_4 = 15$ (check: $\Sigma G + \Sigma I = 44 + 4 \times 4 = 60 = c$).

This example shows the possibility of a very short phase green time (G_c = 1 s) which can be achieved using a group controller whereby minimum green times are set for movements rather than phases. The difference between the phasing arrangement in Cases (a) and (b) is that, in Case (b), Movement 7 is allowed to overlap with Movement 9 for a period of $G_c + I_c = 5$ s, reducing the effect of a long minimum time for pedestrians ($t_{m9} = 17$). This reduces the intersection lost time from 25 s to 20 s and achieves a better result than given by the simpler phasing arrangement in Case (a). It must be emphasised that a better result is obtained in spite of an increased number of phases. This shows the difference in group-control and phase-control philosophies, and is an example which indicates that better phasing designs can be achieved by 'increasing overlaps' rather than 'decreasing the number of phases' (see Appendix A).

Case (c)

This phasing arrangement (*Fig. D.6*) has a simple overlap structure and the critical movements can be determined as follows. For phases A to C, either 6 or (4,8) are critical. Since $T_6 = 47 < T_{4,8} = 24 + 28 = 52$, movements 4 and 8 are critical. For phases C to A, either 9 or (7,1) are critical. Since $T_9 = 17 < T_{7,1} = 35 + 12 = 47$, movements 1,7 are critical. Therefore, $L = \ell_4 + \ell_8 + \ell_7 + \ell_1 = 3 \times 4 + 12 = 24$, $Y = y_4 + y_8 + y_7 = 0.18 + 0.22 + 0.28 = 0.68$ and $U = u_4 + u_8 + u_7 = 0.20 + 0.24 + 0.31 = 0.75$ (see *Table D.7*). The cycle times are $c_p = 96$ and $c_o = 124$. The green times for a chosen cycle time of $c = 110$ are given in *Table D.7*. Phase green times are $G_A = g_4 = 23$, $G_B = g_8 = 28$, $G_C = g_7 = 35$ and $G_D = g_1 = 8$ (check: $\Sigma G + \Sigma I = 94 + 16 = 110 = c$).

It is seen that the phasing arrangement in Case (b) is significantly better than in Case (c) which requires much longer cycle time to achieve the same intersection degree of saturation. To compare the three phasing arrangements in Cases (a) to (c), spare capacities can be calculated from eqn (8.1) using a maximum cycle time of, say, $c_{max} = 120$ s. In Case (a) $U_{max} = (120 - 25)/120 = 0.79$, PSC = ((0.79/0.63) − 1)100 = 25 per cent. In Case (b), $U_{max} = (120 - 20)/120 = 0.83$, PSC = ((0.83/0.63) − 1)100 = 32 per cent. In Case (c), $U_{max} = (120 - 24)/120 = 0.80$, PSC = ((0.80/0.75) − 1)100 = 7 per cent.

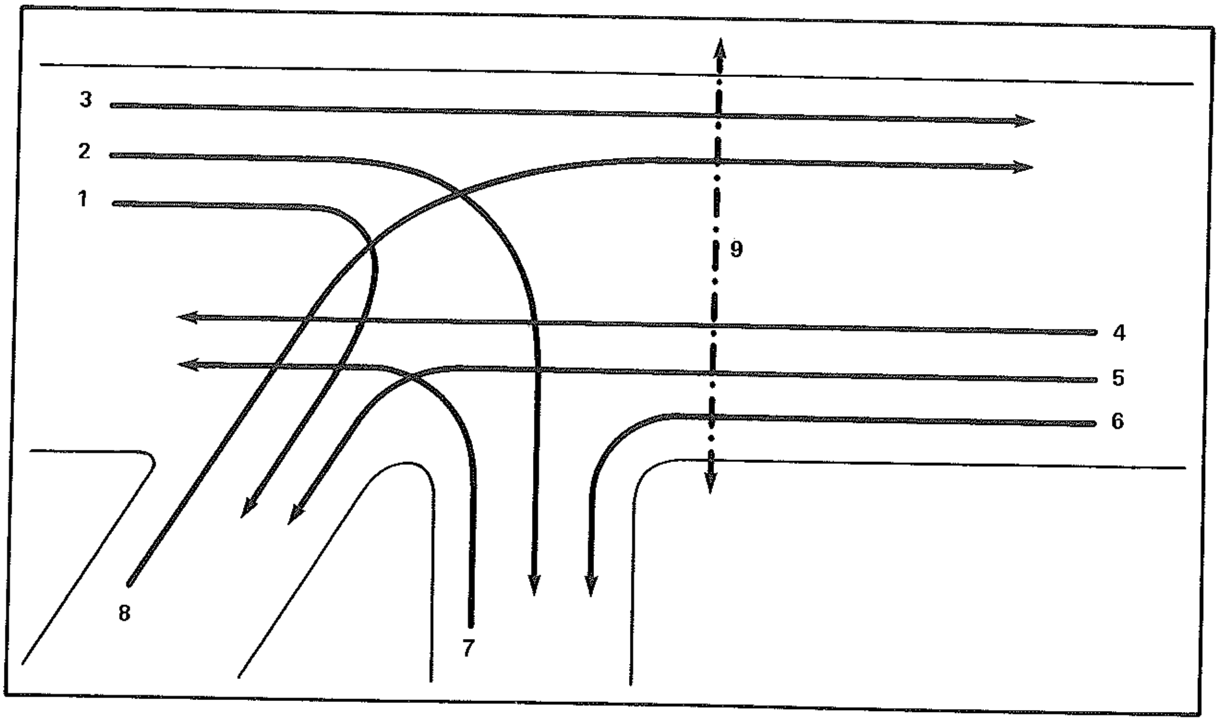


Fig. D.3 — Movements for Example 4 (after Zuzarte Tully and Murchland (1977))

TABLE D.4

CRITICAL MOVEMENT SEARCH TABLE
EXAMPLE 4 (Cases a to c)

DATA

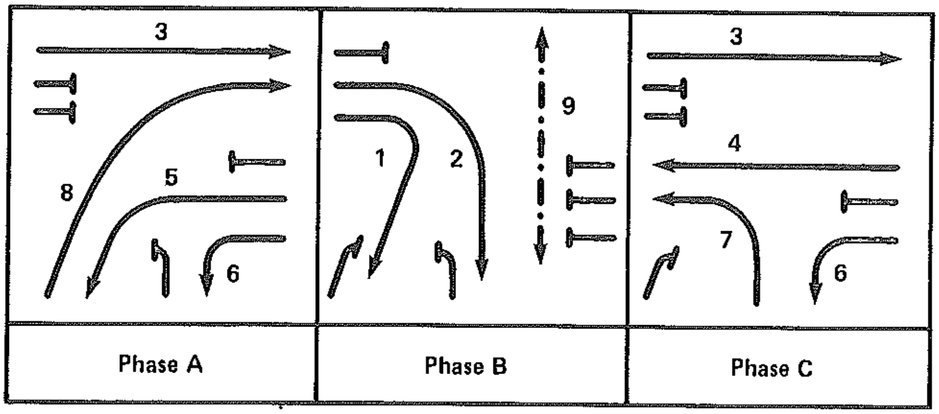
Movement	Starting Phase #	Termin. Phase #	Intergreen Time (I)	Min.Disp. Green (G_m)	Arrival Flow (q)	Satur. Flow (s)	Lost Time (ℓ)	Min.Eff. Green (g_m)	Prac. Deg.Sat. (x_p)
1	*B*	*C*	*4*	*8*	*120*	*3760*	~~4~~ *12*	*8*	*0.90*
2	*B*	*C*	*4*	*8*	*260*	*4000*	*4*	*8*	*0.90*
3	*C*	*B*	*4*	*8*	*250*	*2620*	*4*	*8*	*0.90*
4	*C*	*A*	*4*	*8*	*630*	*3500*	*4*	*8*	*0.90*
5	*A*	*B*	*4*	*8*	*870*	*2980*	*4*	*8*	*0.90*
6	*C*	*B*	*4*	*8*	*720*	*1840*	*4*	*8*	*0.90*
7	*C*	*A*	*4*	*8*	*930*	*3360*	*4*	*8*	*0.90*
8	*A*	*B*	*4*	*8*	*660*	*2970*	*4*	*8*	*0.90*
9	*B*	*C*	*4*	*13*	*Pedestrians*		~~4~~ *17*	*13*	*0.90*

For Case (a) only

Phase Data for Cases (b) and (c)

	Case (b)		Case (c)	
Movement	Starting Phase	Termin. Phase	Starting Phase	Termin. Phase
1	*B*	*C*	*D*	*A*
2	*B*	*D*	*C*	*A*
3	*D*	*B*	*A*	*C*
4	*D*	*A*	*A*	*B*
5	*A*	*B*	*A*	*C*
6	*D*	*B*	*A*	*C*
7	*C*	*A*	*C*	*D*
8	*A*	*B*	*B*	*C*
9	*B*	*D*	*C*	*A*

(a) Phasing



(b) Critical movement search

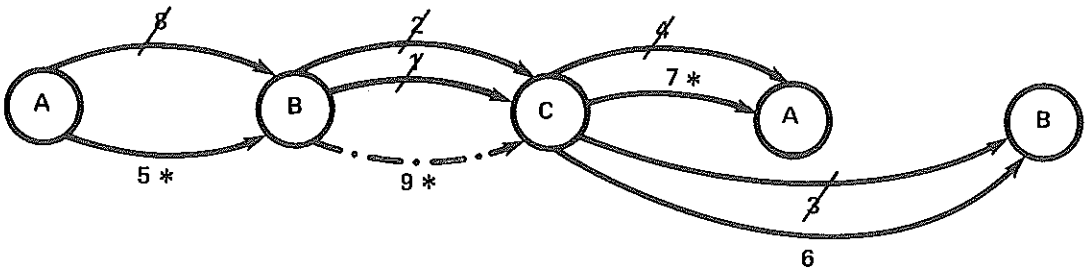


Fig. D.4 — Example 4, Case (a)

TABLE D.5

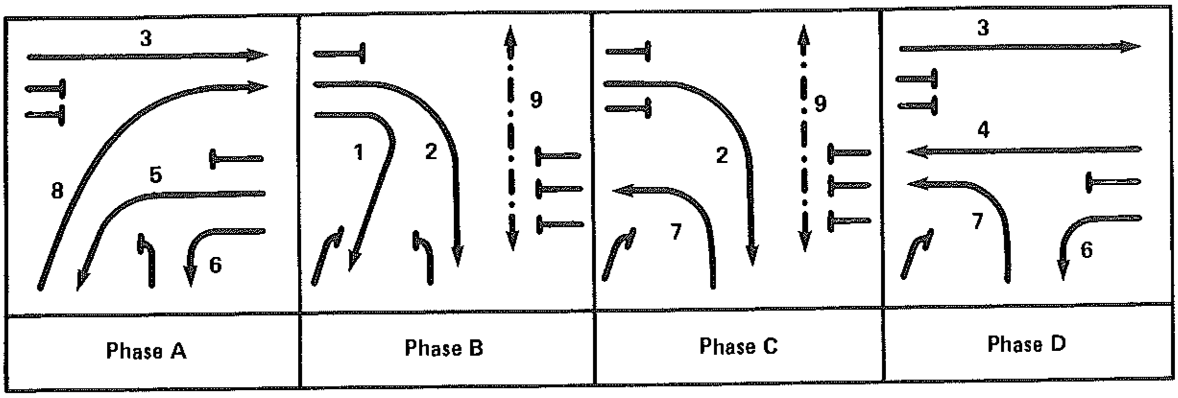
CRITICAL MOVEMENT SEARCH TABLE
EXAMPLE 4, Case (a)

CALCULATIONS

Movement	$y = q/s$	$u = y/x_p$	$100\,u+\ell$	$t_m = G_m + I$	t	Check for c = *80*			
						$uc + \ell$	t′	g	$x = (c/g)\,y$
1	*0.03*	*0.03*	*8*	*12*	*12*	*6*	*12*	*13*	*0.18*
2	*0.07*	*0.08*	*12*	*12*	*12*	*10*	*12*	*13*	*0.43*
3	*0.10*	*0.11*	*15*	*12*	*15*	*13*	*13*	*59*	*0.14*
4	*0.18*	*0.20*	*24*	*12*	*24*	*20*	*20*	*27*	*0.53*
5 *	*0.29*	*0.32*	*36*	*12*	*36*	*30*	*30*	*28*	*0.83*
6	*0.39*	*0.43*	*47*	*12*	*47*	*38*	*38*	*59*	*0.52*
7 *	*0.28*	*0.31*	*35*	*12*	*35*	*29*	*29*	*27*	*0.83*
8	*0.22*	*0.24*	*28*	*12*	*28*	*23*	*23*	*28*	*0.62*
9 *	—	—	—	*17*	*17*	—	*17*	*13*	—

* Critical movements

(a) Phasing



(b) Critical movement search

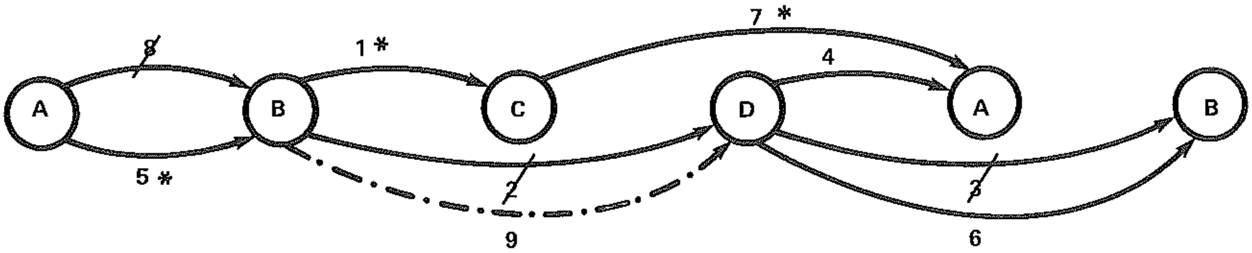


Fig. D.5 — Example 4, Case (b)

TABLE D.6

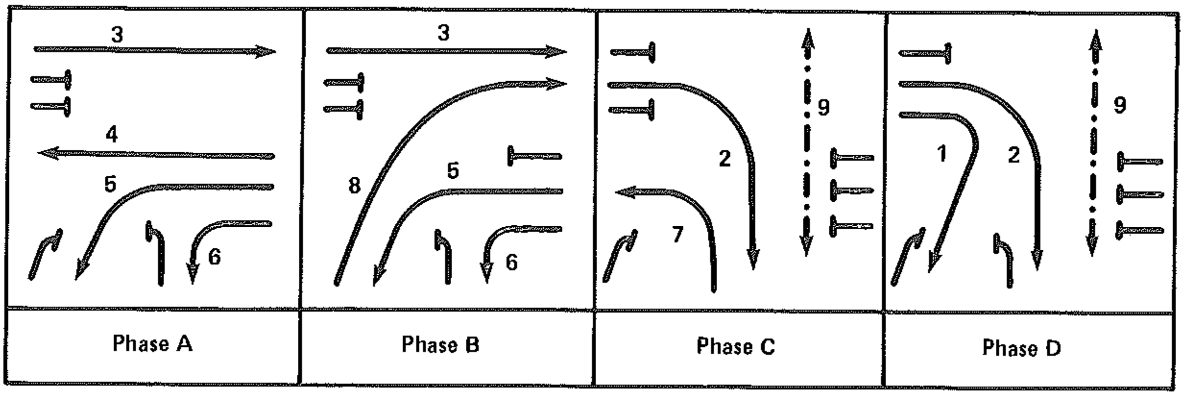
CRITICAL MOVEMENT SEARCH TABLE
EXAMPLE 4, Case (b)

CALCULATIONS

Movement	y = q/s	u = y/x_p	100 u+ℓ	t_m = G_m + I	t	Check for c = 160			
						uc + ℓ	t′	g	x = (c/g) y
1 *	~~0.03~~	~~0.03~~	8	12	12	6	12	8	0.23
2	0.07	0.08	12	12	12	9	12	13	0.32
3	0.10	0.11	15	12	15	11	12	39	0.15
4	0.18	0.20	24	12	24	16	16	15	0.72
5 *	0.29	0.32	36	12	36	23	23	20	0.87
6	0.39	0.43	47	12	47	30	30	39	0.60
7 *	0.28	0.31	35	12	35	23	29	20	0.84
8	0.22	0.24	28	12	28	18	18	20	0.66
9	–	–	–	17	17	–	17	13	–

* Critical movements

(a) Phasing



(b) Critical movement search

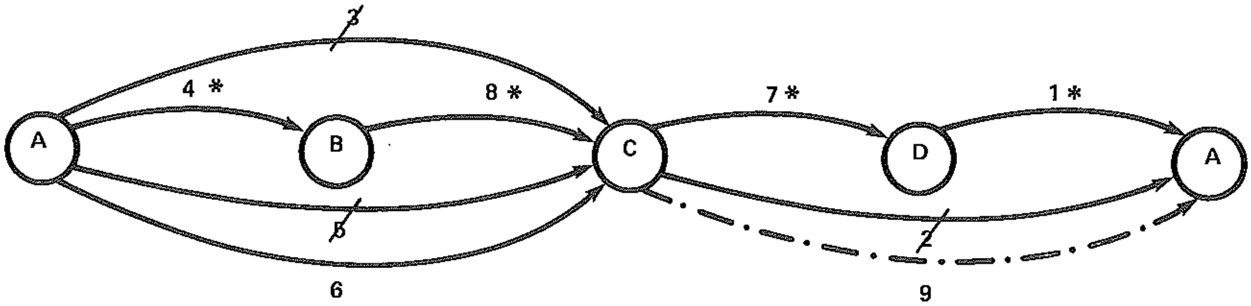


Fig. D.6 — Example 4, Case (c)

TABLE D.7

CRITICAL MOVEMENT SEARCH TABLE
EXAMPLE 4, Case (c)

CALCULATIONS

Movement	$y = q/s$	$u = y/x_p$	$100\,u+\ell$	$t_m = G_m + I$	t	Check for c = *110*			
						$uc + \ell$	t'	g	$x = (c/g)\,y$
1 *	~~*0.03*~~	~~*0.03*~~	*8*	*12*	*12*	*7*	*12*	*8*	*0.41*
2	*0.07*	*0.08*	*12*	*12*	*12*	*13*	*13*	*47*	*0.16*
3	*0.10*	*0.11*	*15*	*12*	*15*	*16*	*16*	*55*	*0.20*
4 *	*0.18*	*0.20*	*24*	*12*	*24*	*26*	*26*	*23*	*0.86*
5	*0.29*	*0.32*	*36*	*12*	*36*	*39*	*39*	*55*	*0.58*
6	*0.39*	*0.43*	*47*	*12*	*47*	*51*	*51*	*55*	*0.78*
7 *	*0.28*	*0.31*	*35*	*12*	*35*	*38*	*38*	*35*	*0.88*
8 *	*0.22*	*0.24*	*28*	*12*	*28*	*30*	*30*	*28*	*0.86*
9	–	–	–	*17*	*17*	–	*17*	*47*	–

* Critical movements

D.5 EXAMPLE 5

This is based on an example by Messer and Fambro (1977). The movements are as shown in *Fig. A.2* of Appendix A. The variable phasing arrangement shown in *Fig. A.5* is used for both North-South (1 to 4) and East-West (5 to 8) movements. The data and calculations are given in *Table D.8.* Although a variable phasing arrangement is used, the start and end phases given in *Table D.8a* and illustrated in *Fig. D.7a* are as determined according to the required movement green times calculated in *Table D.8b* ($t_3 > t_4$ and $t_7 > t_8$). As seen from *Fig. D.7*, critical movements are (1,4) or (2,3) in phases A,B,C and (5,8) or (6,7) in phases D,E,F. Since $T_{1,4} = 25 + 10 = 35 > T_{2,3} = 12 + 18 = 30$ and $T_{6,7} = 10 + 22 = 32 > T_{5,8} = 15 + 13 = 28$, movements (1,4,6,7) are critical.

Therefore,

$L = \ell_1 + \ell_4 + \ell_6 + \ell_7 = 4 + 10 + 10 + 4 = 28$
$Y = y_1 + y_7 = 0.18 + 0.19 = 0.37$
$U = u_1 + u_7 = 0.21 + 0.22 = 0.43$

and hence

$c_p = 28/(1 - 0.43) = 49$
$c_o = (1.4 \times 28 + 6)/(1 - 0.37) = 72$ (for minimum delay).

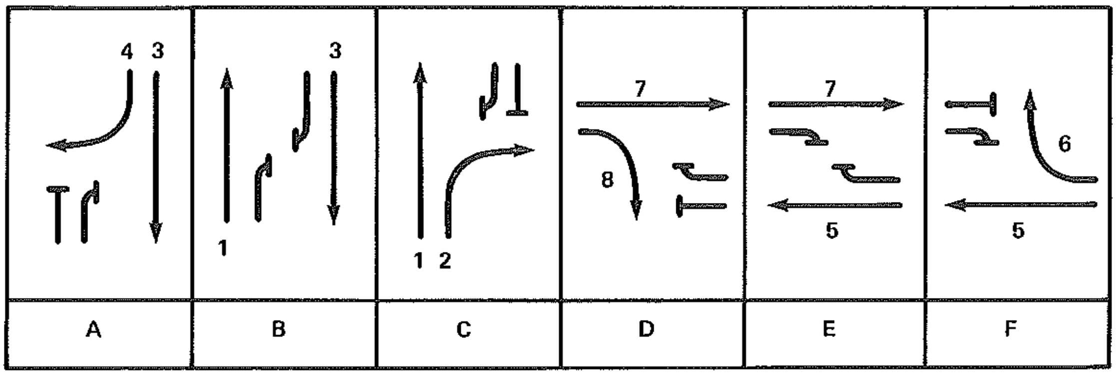
Note that a minimum cycle time of $c_m = 4 \times (6 + 4) = 40$ s satisfies the minimum green time requirements of all movements, in which case phases B and E are not necessary (zero times).

Choosing a cycle time of $c = 60$ s, the critical movements remain the same. The green times are calculated as follows. For critical movements:

$c - L = 60 - 28 = 38$
$g_4 = g_6 = 6\ (= g_m)$
$g_1 = (32/0.37)\ 0.18 = 16$
$g_7 = (32/0.37)\ 0.19 = 16$

For non-critical movements: total time for phases A,B,C is $c^* = (g_4 + \ell_4) + (g_1 + \ell_1) = 10 + (16 + 4) = 30$, lost time is $L^* = \ell_2 + \ell_3 = 10 + 4 = 14$, hence $g_3 = c^* - L^* = 30 - 14 = 16$ and $g_2 = 6\ (= g_m)$. Similarly, $g_5 = 16$, $g_8 = 6\ (= g_m)$ are found. The green times and the corresponding degrees of saturation are given in *Table D.8b.*

(a) Phasing



(b) Critical movement search

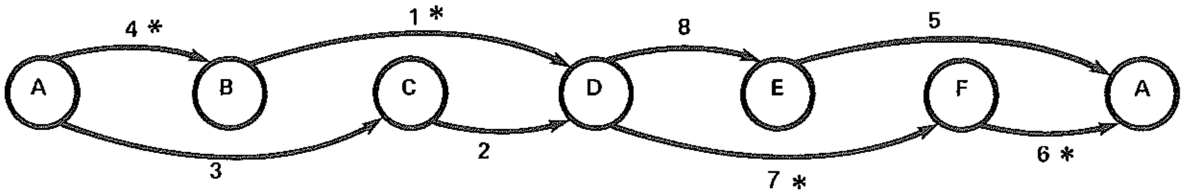


Fig. D.7 — Example 5 (after Messer and Fambro (1977))

TABLE D.8

CRITICAL MOVEMENT SEARCH TABLE
EXAMPLE 5

(a) DATA

Movement	Starting Phase	Termin. Phase	Intergreen Time (I)	Min.Disp. Green (G_m)	Arrival Flow (q)	Satur. Flow (s)	Lost Time (ℓ)	Min.Eff. Green (g_m)	Prac. Deg.Sat. (x_p)
1	*B*	*D*	*4*	*6*	*540*	*3030*	*4*	*6*	*0.85*
2	*C*	*D*	*4*	*6*	*115*	*1570*	~~4~~ *10*	*6*	*0.85*
3	*A*	*C*	*4*	*6*	*320*	*3030*	*4*	*6*	*0.85*
4	*A*	*B*	*4*	*6*	*65*	*1570*	~~4~~ *10*	*6*	*0.85*
5	*E*	*A*	*4*	*6*	*395*	*4170*	*4*	*6*	*0.85*
6	*F*	*A*	*4*	*6*	*35*	*1570*	~~4~~ *10*	*6*	*0.85*
7	*D*	*F*	*4*	*6*	*775*	*4120*	*4*	*6*	*0.85*
8	*D*	*E*	*4*	*6*	*100*	*1510*	~~4~~ *10*	*6*	*0.85*

(b) CALCULATIONS

Movement	y = q/s	u = y/x_p	100 u+ℓ	t_m = G_m + I	t	Check for c = *60*			
						uc + ℓ	t'	g	x = (c/g) y
1 *	*0.18*	*0.21*	*25*	*10*	*25*	*17*	*17*	*16*	*0.68*
2	~~0.07~~	~~0.08~~	*12*	*10*	*12*	*9*	*10*	*6*	*0.70*
3	*0.11*	*0.13*	*18*	*10*	*18*	*12*	*12*	*16*	*0.41*
4 *	~~0.04~~	~~0.05~~	*9*	*10*	*10*	*7*	*10*	*6*	*0.40*
5	*0.09*	*0.11*	*15*	*10*	*15*	*11*	*11*	*16*	*0.34*
6 *	~~0.02~~	~~0.02~~	*6*	*10*	*10*	*5*	*10*	*6*	*0.20*
7 *	*0.19*	*0.22*	*22*	*10*	*22*	*17*	*17*	*16*	*0.71*
8	~~0.07~~	~~0.08~~	*13*	*10*	*13*	*9*	*10*	*6*	*0.70*

* Critical movements

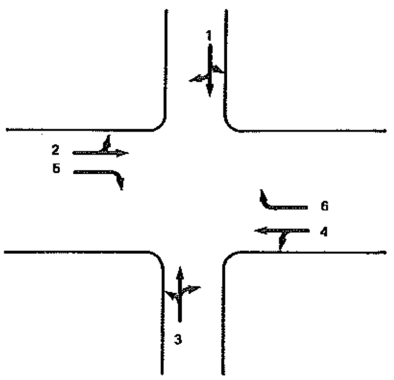
D.6 EXAMPLE 6

This is based on an example by Yagar (1975). The movements and signal phasing are shown in *Figs D.8*a and *b*. The right-turn movements 5 and 6 are allocated exclusive lanes. They are allowed to make filter (opposed) turns during Phase B and free (unopposed) turns during Phase C, i.e. this is a case of two saturation flows per movement (see Appendix F.2). The saturation flows and lost times given in *Table D.9*a for Movements 5 and 6 are for the unopposed conditions. It will be assumed that these movements are stopped at the end of Phase B before they are shown green arrows during Phase C but the number of departures at the end of Phase B green period is negligible ($n_f = 0$). Filter right-turns which are part of Movements 1 and 3 will be treated as having fixed opposed turn equivalents because of low turning volumes. The saturation flows given in *Table D.9*a for these movements allow for opposed turn effects.

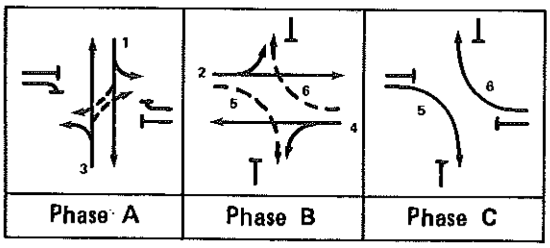
Assume initially that Movements 5 and 6 can depart only during Phase C, i.e. neglect the opposed turns in Phase B. The calculations based on this assumption are given in *Table D.9b*. From *Fig. D.8*c, the critical movements are easily found as (1,2,5). Therefore, $L = 2 + 3 + 3 = 8$, $Y = 0.22 + 0.38 + 0.16 = 0.76$ and $U = 0.24 + 0.42 + 0.18 = 0.84$. Hence, the practical cycle time is:

$c_p = 8/(1 - 0.84) = 50$ s,

(a) Movements



(b) Phasing



(c) Critical movement search

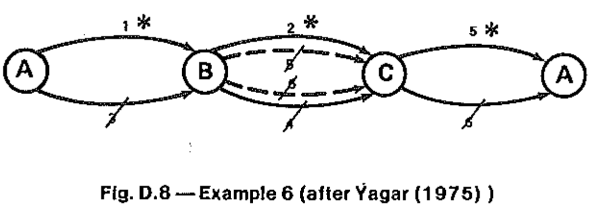


Fig. D.8 — Example 6 (after Yagar (1975))

and the approximate minimum-delay cycle time is

$c_o = (1.4 \times 8 + 6)/(1 - 0.76) = 72$ s.

Choosing a cycle time of $c = 60$ s, the green times are found as:

$c - L = 60 - 8 = 52$,
$g_1 = (52/0.76)0.22 = 15$, $G_A = g_1 + \ell_1 - I_A = 15 + 2 - 4 = 13$,
$g_2 = (52/0.76)0.38 = 26$, $G_B = g_2 + \ell_2 - I_B = 26 + 3 - 4 = 25$,
$g_5 = (52/0.76)0.16 = 11$, $G_C = g_5 + \ell_5 - I_C = 11 + 3 - 4 = 10$,
$g_3 = g_1 = 15$, $g_4 = g_2 = 26$, $g_6 = g_5 = 11$.

The crossed values in *Table D.9b* correspond to these timings.

Saturation flows and lost times of Movements 5 and 6 can now be revised using the initial timings according to the method given in Section 5.6. Calculations are given below for Movement 5 which is the more critical right turn movement. The opposing movement is Movement 4 which has the parameters, $q = 480$ veh/h $= 0.133$ veh/s, $qc = 0.133 \times 60 = 8.0$, $s = 2500$ veh/h $= 0.694$ veh/s, $sg = 0.694 \times 26 = 18.0$ and $s - q = 0.561$ veh/s. From eqn (5.10), $g_u = (18.0 - 8.0)/0.561 = 18$ s, and from *Fig. 5.1*, $s_u = 0.21$ veh/s $= 750$ veh/h. From eqn (5.7), the effective green time for Movement 5 in Phase B is found as $g_B = g_u = 18$ (since $n_f = 0$). From eqn (5.8), the lost time is $\ell_B = (25 + 4) - 18 = 11$. The saturation flow is $s_B = s_u = 0.21$ veh/s.

To calculate the revised signal timings according to the method given in Appendix F.2 (eqn (F.11)),

$s_B\,g_B + s_c\,g_c = 0.21 \times 18 + (1500/3600)11 = 8.4$ veh,
$q/(s_B\,g_B + s_c\,g_c) = (240/3600)/8.4 = 0.008$,
$y_B = 0.008 \times 18 = 0.14$, $y_c = 0.008 \times 11 = 0.09$

are found. The required times under $c = 60$ are

$t_B = (0.14/0.9)\,60 + 11 = 20$, $t_c = (0.09/0.9)\,60 + 3 = 9 = G_m + I$.

It is seen from *Table D.9b* that Movement 2 is still the critical one in Phase B. The required time for Movement 5 in Phase C is decreased because of the additional capacity provided in Phase B. The revised timings are calculated as follows:

$L = 2 + 3 + 9 = 14$, $Y = 0.22 + 0.38 = 0.60$, $U = 0.24 + 0.42 = 0.66$,
$c_p = 14/(1 - 0.66) = 41$,
$c_o = (1.4 \times 14 + 6)/(1 - 0.60) = 64$.

Choosing a cycle time of $c = 50$,

$c - L = 50 - 14 = 36$,
$g_5 = 6 = g_m$, $G_c = 6 + 3 - 4 = 5 = G_m$,
$g_1 = (36/0.60)0.22 = 13 = g_3$,
$G_A = 13 + 2 - 4 = 11$,
$g_2 = (36/0.60)0.38 = 23 = g_4$,
$G_B = 23 + 3 - 4 = 22$.

Calculations based on these timings give $g_B = g_u = 17$, $s_B\,g_B + s_c\,g_c = 6.1$, $y_B = 0.19$ and $y_c = 0.07$ for Movement 5. The results are not affected by the changes in these parameters, and hence above timings are final. These are given in *Table D.9b*, together with the corresponding degrees of saturation. Note that the capacity for Movement 5 is $Q_5 = (6.1/50)3600 = 440$ veh/h, and the degree of saturation is $x_5 = 240/440 = 0.55$.

TABLE D.9

CRITICAL MOVEMENT SEARCH TABLE
EXAMPLE 6

(a) DATA

Movement	Starting Phase	Termin. Phase	Intergreen Time (I)	Min.Disp. Green (G_m)	Arrival Flow (q)	Satur. Flow (s)	Lost Time (ℓ)	Min.Eff. Green (g_m)	Prac. Deg.Sat. (x_p)
1	*A*	*B*	*4*	*10*	*300*	*1360*	*2*	*12*	*0.90*
2	*B*	*C*	*4*	*5*	*960*	*2500*	*3*	*6*	*0.90*
3	*A*	*B*	*4*	*10*	*300*	*1360*	*2*	*12*	*0.90*
4	*B*	*C*	*4*	*5*	*480*	*2500*	*3*	*6*	*0.90*
5	*B,C*†	*A*	*4*	*5*	*240*	*1500*	~~3~~ *9*	*6*	*0.90*
6	*B,C*†	*A*	*4*	*5*	*120*	*1500*	~~3~~ *9*	*6*	*0.90*

† Opposed in Phase B, unopposed in Phase C.

(b) CALCULATIONS

Movement	$y = q/s$	$u = y/x_p$	$100u+\ell$	$t_m = G_m + I$	t	Check for c = ~~60~~ 50			
						$uc + \ell$	t'	g	$x = (c/g)\,y$
1 *	*0.22*	*0.24*	*26*	*14*	*26*	~~16~~ *14*	~~16~~ *14*	~~15~~ *13*	*0.85*
2 *	*0.38*	*0.42*	*45*	*9*	*45*	~~28~~ *24*	~~28~~ *24*	~~26~~ *23*	*0.83*
3	*0.22*	*0.24*	*26*	*14*	*26*	~~16~~ *14*	~~16~~ *14*	~~15~~ *13*	*0.85*
4	*0.19*	*0.21*	*24*	*9*	*24*	~~16~~ *14*	~~16~~ *14*	~~26~~ *23*	*0.41*
5 *	~~0.16~~	~~0.18~~	*21*	*9*	*21*	~~14~~ *7*	~~14~~ *9*	~~11~~ *6*	*0.55#*
6	*0.08*	*0.09*	*12*	*9*	*12*	~~8~~ *5*	*9*	~~11~~ *6*	*0.33#*

* Critical movements

\# Calculated from $x = q/Q$, where $Q = (\Sigma sg)/c$

D.7 EXAMPLE 7

This example deals with a more complicated case where there are movements with two saturation flows per cycle and there are multiple overlaps. The movements and the signal phasing are shown in *Figs D.9a* and *b*. The right-turn Movements 1 and 3 have exclusive lanes. Movement 1 is unopposed during Phase A and opposed during Phase C (non-consecutive green periods). Movement 3 is unopposed during Phases A and B, and opposed during Phase C (consecutive green periods). The data are given in *Table D.10a*. The saturation flows and lost times for Movements 1 and 3 are for unopposed conditions. It will be assumed that the average number of departures at the end of Phase C green period is n_f = 1.4 veh for these movements. The saturation flows of Movements 5 and 6 include the effects of opposed turns, and will be treated as fixed values due to low turning volumes. Equal maximum acceptable degrees of saturation are chosen for all movements.

Initial Calculations

The required movement times calculated by neglecting opposed turns in Phase C are given in *Table D.10b*. As seen from *Fig. D.10a*, the critical movements are either (3,2,5) or (1,4,5). Since $T_{3,2,5}$ = 34 + 26 + 40 = 100 > $T_{1,4,5}$ = 17 + 37 + 40 = 94, (3,2,5) are the critical movements. Therefore, L = 3 x 4 = 12, Y = 0.27 + 0.20 + 0.32 = 0.79, U = 0.30 + 0.22 + 0.36 = 0.88, c_p = 12/(1 − 0.88) = 100 and c_o = (1.4 x 12 + 6)/(1 − 0.79) = 109 (minimum delay). Choosing c = 100, the green times are found as g_3 = 30, g_2 = 22, g_5 = 36, g_1 = 15 and g_4 = 37.

First Iteration

Opposed turn saturation flows and green times will be calculated for Movements 1 and 3 in Phase C using the initial signal timings. For Movement 1, s_{1c} = s_u = 0.14 veh/s (opposing flow q_4 = 840 veh/h), g_u = 10, g_{1c} = 10 + (1.4/0.14) = 20, ℓ_{1c} = 22 + 4 − 20 = 6 are found. Since s_{1A} = 1540/3600 = 0.43 veh/s and g_{1A} = 15, $s_{1A}\,g_{1A}$ + $s_{1c}\,g_{1c}$ = 9.3 veh is found, and eqn (F.11) gives,

y_{1A} = (180/3600) x (15/9.3) = 0.08
y_{1c} = (180/3600) x (20/9.3) = 0.11.

For Movement 3, s_{3c} = s_u = 0.18 veh/s (opposing flow q_2 = 600 veh/h), g_u = 2, g_{3c} = 2 + (1.4/0.18) = 10, ℓ_{3c} = 22 + 4 − 10 = 16, $s_{3A}\,g_{3AB}$ + $s_{3c}\,g_{3c}$ = 0.43 x 30 + 0.18 x 10 = 14.7 veh, and hence

y_{3AB} = (420/3600) x (30/14.7) = 0.24
y_{3c} = 420/3600) x (10/14.7) = 0.08

are found. The required movement times calculated using these parameters are shown in *Fig. D.10b*. It should be noted that t_{1A} = 14 = g_m + ℓ , i.e. the time for Phase A will be determined by the minimum green time for Movement 1.

The critical movements are unchanged and L = 3 x 4 = 12, Y = y_{3AB} + y_2 + y_5 = 0.23 + 0.20 + 0.32 = 0.75, U = 0.75/0.90 = 0.83, c_p = 12/(1 − 0.83) = 71, c_o = (1.4 x 12 + 6)/(1 − 0.75) = 91 are found. Choosing a cycle time of c = 80, the green times are found as g_{3AB} = 21, g_2 = 18, g_5 = 29, g_{1A} = 10 (min) and g_4 = 29.

Second Iteration

With the new timings, g_{1c} = 17, ℓ_{1c} = 5, g_{3c} = 10, ℓ_{3c} = 16, y_{1A} = 0.07, y_{1c} = 0.13, y_{3AB} = 0.23 and y_{3c} = 0.11 are found. The required movement times are given in *Fig. D.10c*. It is seen that Movement 3 is now critical in Phase C also. Therefore, L = ℓ_{3AB} + ℓ_{3c} + ℓ_5 = 4 + 16 + 4 = 24, Y = y_{3AB} + y_{3c} + y_5 = 0.23 + 0.11 + 0.32 = 0.66, U = 0.66/0.90 = 0.73, c_p = 24/(1 − 0.73) = 89, c_o = (1.4 x 24 + 6)/(1 − 0.66) = 116. Choosing a cycle time of c = 90, the green times are g_{3AB} = 23, g_{3c} = 11, g_5 = 32, g_2 = g_{3c} + ℓ_{3c} − ℓ_2 = 11 + 16 − 4 = 23, g_{1A} = 10(min) and g_4 = 36.

Third Iteration

The revised values are g_{1c} = 23, ℓ_{1c} = 4, g_{3c} = 14, ℓ_{3c} = 13, y_{1A} = 0.07, y_{1c} = 0.15, y_{3AB} = 0.22 and y_{3c} = 0.13. The new required movement times are shown in *Fig. D.10d*. The critical movements are as in the previous iteration and L = 4 + 13 + 4 = 21, Y = 0.22 + 0.13 + 0.32 = 0.67 and U = 0.67/0.90 = 0.74 are found. As a result, c_p = 21/(1 − 0.74) = 81 and c_o = (1.4 x 21 + 6)/(1 − 0.67) = 107. Choosing the same cycle time as in the previous iteration, c = 90, the green times are g_{3AB} = 23, g_{3c} = 13, g_5 = 33, g_2 = 13 + 13 − 4 = 22, g_{1A} = 10(min) and g_4 = 35. It is considered that these timing are sufficiently close to those at the end of the previous iteration and these will be used as final timings. The corresponding phase green times are

$G_A = g_1 + \ell_1 - I_A = 10 + 4 - 6 = 8 = G_m$,
$G_B = g_{3AB} + \ell_{3AB} - (G_A + I_A + I_B) = 23 + 4 - (8 + 6 + 6) = 7$,
$G_C = g_2 + \ell_2 - I_C = 22 + 4 - 6 = 20$,
$G_D = g_5 + \ell_5 - I_D = 33 + 4 - 6 = 31$.

To check, $\Sigma G + \Sigma I$ = (8 + 7 + 20 + 31) + (4 x 6) = 90 = c.

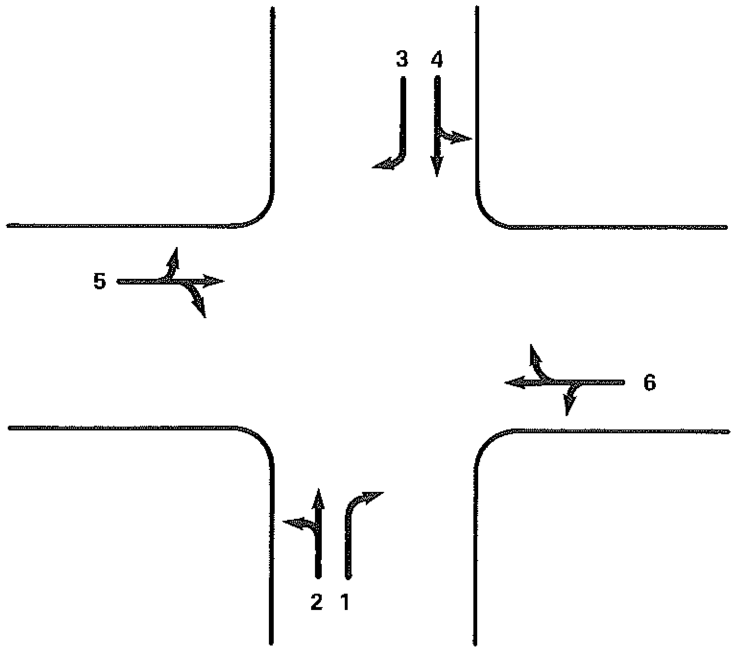
The green times and the corresponding degrees of saturation are given in *Table D.10b*. For Movement 1, Σsg = 7.4 veh, Q_1 = 300 veh/h, x_1 = 180/300 = 0.60, and for Movement 3, Σsg = 12.2 veh, Q_3 = 490 veh/h, x_3 = 420/490 = 0.86 were found.

In *Fig. D.11*, the departure flow patterns for Movement 3 and its opposing movement (Movement 2) are shown. Queue growth and decay pattern is also shown for Movement 3. As discussed in Appendix F.2, this is a case of two green periods per cycle. Formulae for the calculation of delay (uniform components represented by the shaded area in *Fig. D.11*) in this case have been given by Allsop (1977).*

* Formulae for delay, queue length and number of stops in the case of two green periods per cycle were developed and used in the SIDRA program. Refer to:

AKCELIK, R. (1990). Calibrating SIDRA. Australian Road Research Board Ltd. Research Report No. 180. (Second Edition 1992).

(a) Movements



(b) Phasing

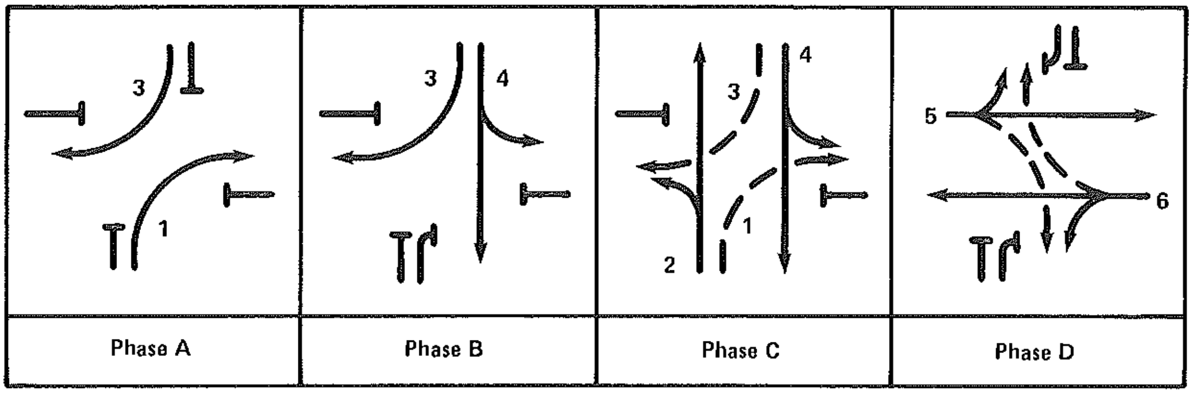


Fig. D.9 — Example 7

TABLE D.10

CRITICAL MOVEMENT SEARCH TABLE
EXAMPLE 7

(a) DATA

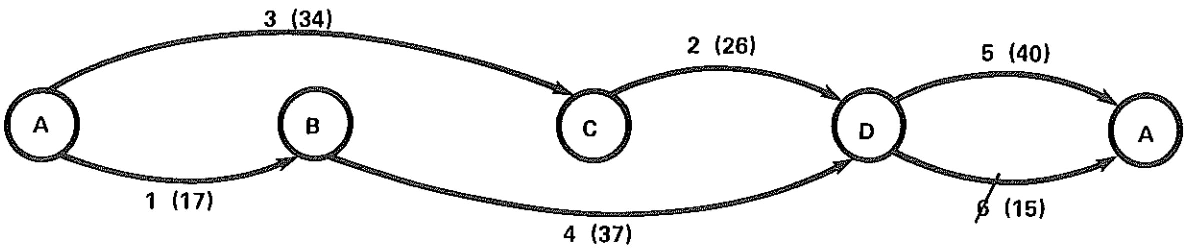
Movement	Starting Phase	Termin. Phase	Intergreen Time (I)	Min.Disp. Green (G_m)	Arrival Flow (q)	Satur. Flow (s)	Lost Time (ℓ)	Min.Eff. Green (g_m)	Prac. Deg.Sat. (x_p)
1	*A,C*	*B,D*	*6*	*8*	*180*	*1540*	*4*	*10*	*0.90*
2	*C*	*D*	*6*	*8*	*600*	*3000*	*4*	*10*	*0.90*
3	*A,C*	*D*	*6*	*8*	*420*	*1540*	*4*	*10*	*0.90*
4	*B*	*D*	*6*	*8*	*840*	*2800*	*4*	*10*	*0.90*
5	*D*	*A*	*6*	*8*	*1470*	*4600*	*4*	*10*	*0.90*
6	*D*	*A*	*6*	*8*	*430*	*4500*	*4*	*10*	*0.90*

(b) CALCULATIONS

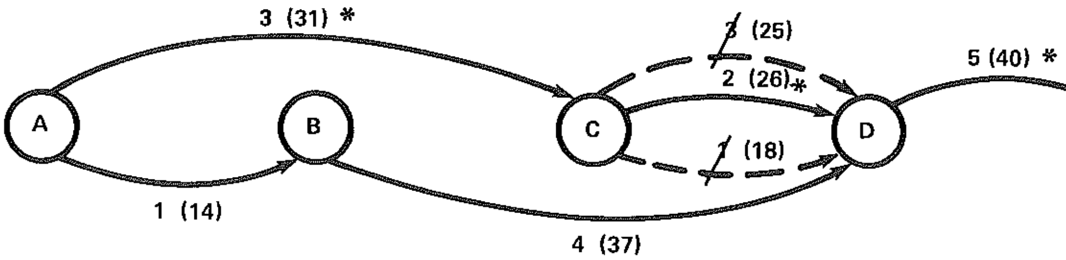
Movement	y = q/s	u = y/x_p	100 u+ℓ	t_m = G_m + I	t	Check for c = *90*			
						uc + ℓ	t′	g	x = (c/g) y
1	*0.12*	*0.13*	*17*	*14*	*17*	*11,19*	*14, 19*	*10, 22*	*0.60#*
2	*0.20*	*0.22*	*26*	*14*	*26*	*24*	*24*	*22*	*0.82*
3 *	*0.27*	*0.30*	*34*	*14*	*34*	*26,26*	*26, 26*	*23, 13*	*0.86#*
4	*0.30*	*0.33*	*37*	*14*	*37*	*34*	*34*	*35*	*0.77*
5 *	*0.32*	*0.36*	*40*	*14*	*40*	*36*	*36*	*33*	*0.87*
6	*0.10*	*0.11*	*15*	*14*	*15*	*14*	*14*	*33*	*0.27*

Calculated from x = q/Q, where Q = (Σsg) / c

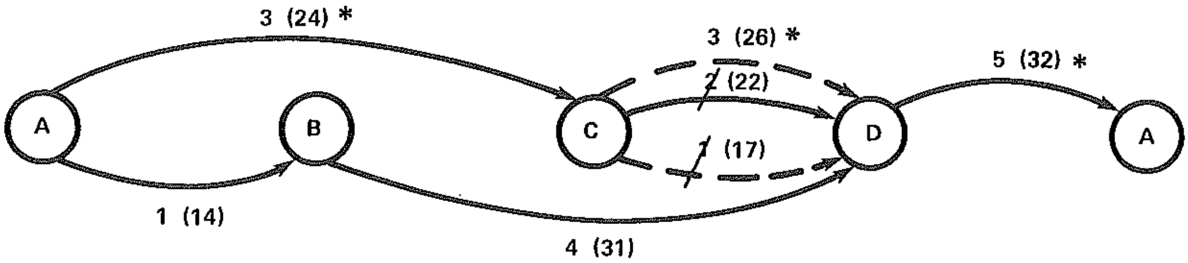
(a) Initial (c = 100)



(b) First Iteration (c = 100)



(c) Second Iteration (c = 80)



(d) Third Iteration (c = 90)

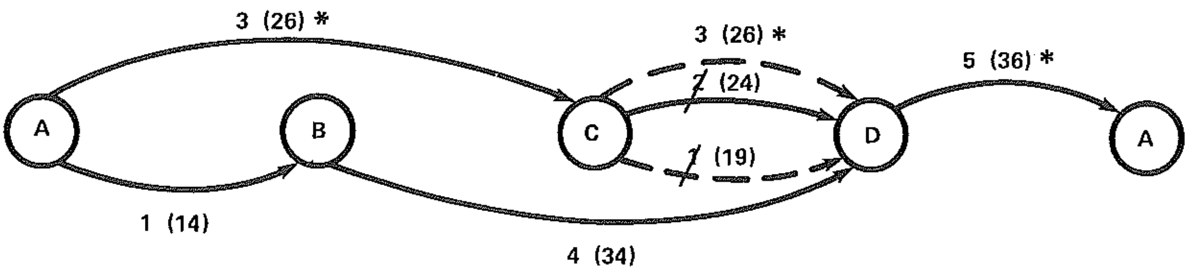


Fig. D.10 — Critical movement search diagrams for Example 7 (the required movement times are shown in brackets)

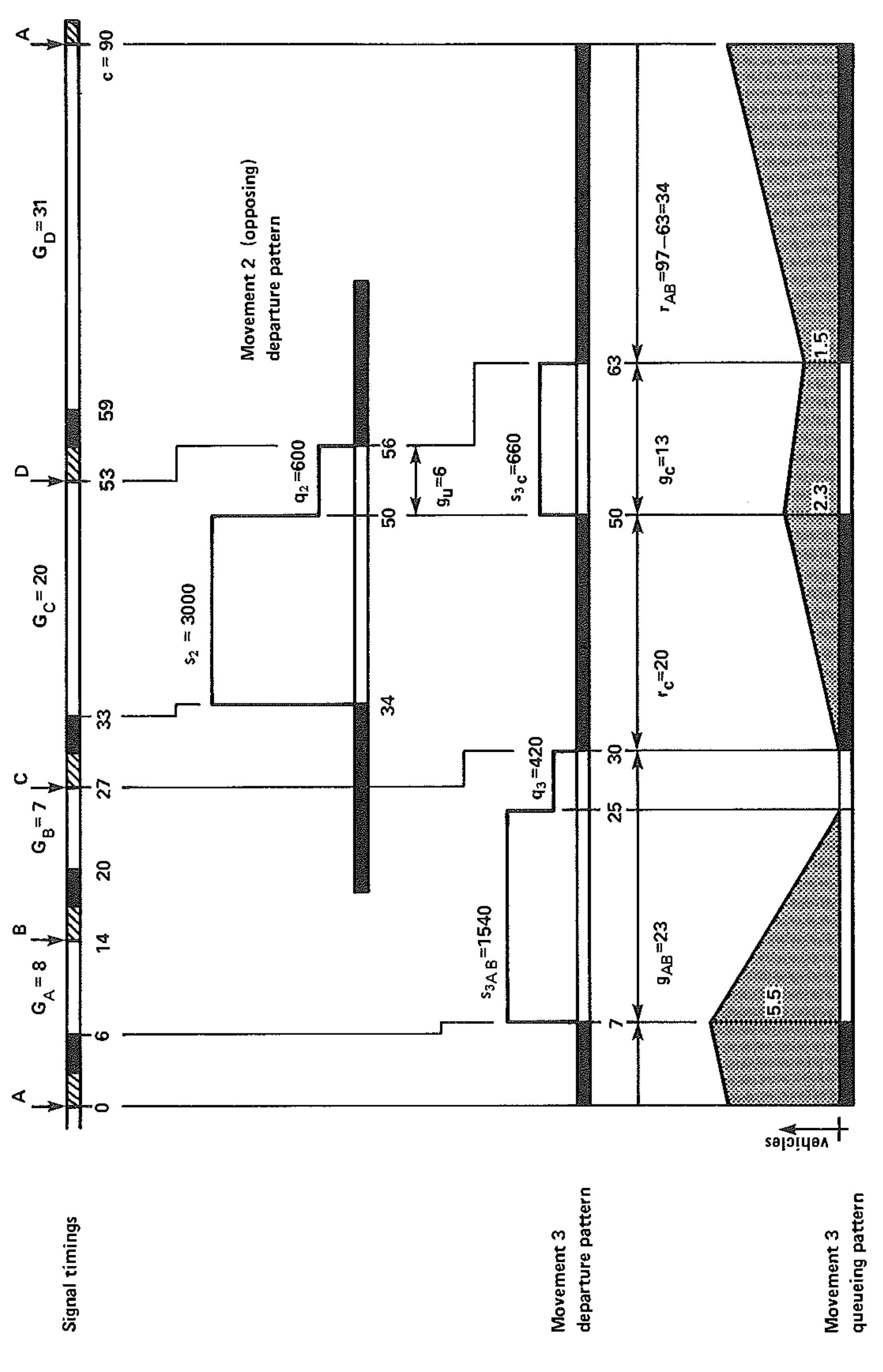


Fig. D.11 — Departure and queueing patterns for Movement 3 in Example 7 (two green periods per cycle)

APPENDIX E

A METHOD FOR MEASURING SATURATION FLOW AND LOST TIME

The method described below for measuring saturation flows and lost times in the field is based on the principles formulated in Section 2 where a basic model of saturation flow, effective green time and lost time is described.

A simple method to measure the saturation flow in vehicle units without considering the composition of traffic can be implemented using a form as shown in the example given in *Table E.1* (a blank form is given in Appendix J). The method consists of counting the number of vehicles *departing from the queue* during three separate intervals (columns 1 to 3 of *Table E.1*):

(a) First Interval: the first 10 s of the green period;

(b) Middle Interval: the rest of the green period while saturated;

(c) Last Interval: the period after the end of green, i.e. amber and the following red period.

Vehicles are counted as they cross the stop line, but a decision must be made about when the saturated period ends by observing the back of the queue. Thus, the saturation time (column 4 of *Table E.1*) must be recorded as the time to clear the vehicles which are stopped during the red period as well as the vehicles which arrive at the back of the queue and are stopped during the green period. The number of these vehicles is recorded in column 2. However, the vehicles which are not stopped are excluded from this count. The saturation period includes the first interval but not the last interval. Its maximum value is the green time (recorded in column 5), and this corresponds to a fully saturated cycle. If the saturation time is less than 10 s, the counts in that cycle must be excluded. The departures in the last interval must be counted only for fully saturated conditions, i.e. when the queue still exists at the end of the green period. If no vehicles depart in spite of saturated conditions, this must be recorded as zero in column 3 (— means that the green period is not fully saturated). The following three cases can be observed from *Table E.1*:

(a) Columns 1 to 3 have numbers recorded. This is a fully saturated cycle, i.e. saturation time = green time (e.g. cycles 1 and 2).

(b) Only column 1, or columns 1 and 2 have numbers recorded. The queue lasts for at least 10 s, but is cleared before the end of green. This is an unsaturated cycle, i.e. saturation time < green time (e.g. cycles 3 and 4).

(c) No numbers are recorded in columns 2 and 3, and the number in column 1 is deleted because the saturation has lasted for less than 10 s (e.g. cycles 5, 22).

The counts must be repeated for a number of cycles (e.g. 30 in *Table E.1*). The TOTAL and the number of SAMPLES must be calculated for each column (X_i, n_i, for column i in *Table E.1*). If there is a number recorded in a column, it is counted as a sample. The saturation flow and lost time can then be calculated from the following formulae.

The saturation flow in vehicles per second is given by:

$$s^* = \frac{X_2}{X_4 - 10n_4} \qquad \text{(E.1)}$$

For the data in *Table E.1*, $s^* = 290/(917 - 10 \times 28) = 0.455$ veh/s. The saturation flow in vehicles per hour is:

$$s = 3600\, s^* \qquad \text{(E.2)}$$

In *Table E.1*, $s = 3600 \times 0.455 = 1640$ veh/h.

The lost time in seconds can be calculated from:

$$\ell = I + 10 - \frac{1}{s^*}\left(\frac{X_1}{n_1} + \frac{X_3}{n_3}\right) \qquad \text{(E.3)}$$

where I is the intergreen time which has to be measured separately (see *Fig. 2.3* in Section 2). In *Table E.1*, $I = 5$, $X_1/n_1 = 86/28 = 3.07$, $X_3/n_3 = 16/15 = 1.07$; therefore the lost time is $= 5 + 10 - (3.07 + 1.07)/0.455 = 15 - 9.1 = 6$ s.

The average displayed green time is:

$$G = X_5/n_5 \qquad \text{(E.4)}$$

and the effective green time is:

$$g = I + G - \ell = \frac{X_5}{n_5} - 10 + \frac{1}{s^*}\left(\frac{X_1}{n_1} + \frac{X_3}{n_3}\right) \qquad \text{(E.5)}$$

For the data in *Table E.1*, $G = 1024/30 = 34$ s, and $g = 5 + 34 - 6 = 33$ s.

It should be noted that, for the measurement of saturation flow only, it is sufficient to collect the data for the middle interval and saturation time (columns 2 and 4 in *Table E.1*).

If the conversion of saturation flows from vehicle units to through car units is desired so as to obtain basic saturation flow data for the site in question, it is necessary to collect data on traffic composition, i.e. to count HEAVY VEHICLES and CARS, through and turning vehicles separately. If opposed turns exist, the opposing movement characteristics must also be measured (flow, saturation flow, green time and cycle time). Using this information, a traffic composition adjustment factor, f_c, can be calculated from eqn (5.4) with the tcu equivalents given in *Table 5.2* in Section 5. Then the saturation flow in tcu/h is $s_{tcu} = f_c\, s_{veh}$, where s_{veh} is the saturation flow calculated from eqn (E.2). The value of s_{tcu} can be compared with the general average values given in *Table 5.1* (after adjustment for lane width and gradient also, if relevant).

The measurements should be ideally made for individual lanes. The existence of lane under-utilisation can be established using this method. If the average saturation period (X_4/n_4) for a lane is significantly less than the other lanes, it is under-utilised. If it is known that several lanes are used equally, e.g. two through lanes, the saturation flow and lost time measurement method described above can be used for these lanes combined together in order to minimise the survey costs. However, the difficulty of deciding when the saturated period ends is increased in this case.

For further reading on saturation flow measurement methods, the reader is referred to Leong (1964), Miller (1968*a*), Branston and Zuylen (1978), Branston (1979), Branston and Gipps (1979) and Saito (1980).

TABLE E.1

SATURATION FLOW AND LOST TIME MEASUREMENT
(An Example)

CYCLE NUMBER	DEPARTURES FROM QUEUE (vehs)			SATURATION TIME * (s)	GREEN TIME (s)
	First Interval	Middle Interval	Last Interval		
	1	2	3	4	5
1	3	12	1	35	35
2	4	3	0	20	20
3	3	6	–	24	29
4	3	–	–	10	14
5	~~1~~	–	–	–	12
6	4	10	–	34	46
7	3	23	1	52	52
8	3	14	–	44	53
9	3	10	2	34	34
10	2	8	1	27	27
11	2	4	–	18	33
12	3	8	–	25	30
13	4	6	–	22	27
14	3	4	–	21	34
15	3	15	0	45	45
16	2	17	3	52	52
17	3	18	1	52	52
18	3	10	–	25	26
19	4	12	2	38	38
20	3	9	1	37	37
21	4	6	–	23	28
22	~~2~~	–	–	–	10
23	3	9	1	20	20
24	3	18	0	46	46
25	3	19	–	45	48
26	2	10	1	32	32
27	4	–	–	10	13
28	4	7	–	24	29
29	2	15	1	50	50
30	3	17	1	52	52
TOTAL	X_1 = 86	X_2 = 290	X_3 = 16	X_4 = 917	X_5 = 1024
SAMPLES	n_1 = 28	n_2 = 26	n_3 = 15	n_4 = 28	n_5 = 30

* Not to include the amber time, i.e. the maximum value to be the green time in col. 5.

INTERGREEN TIME (s) = 5

APPENDIX F

FURTHER ON SATURATION FLOW ESTIMATION

F.1 THROUGH CAR EQUIVALENTS

The method described in Section 5 for the calculation of a traffic composition adjustment factor (eqn (5.4)) to convert the saturation flow from a standard value in through car units (s_{tcu}) to an estimate of the real-life value in vehicle units (s_{veh}) is based on the use of through car equivalents (tcu/veh). The recommended *average* values of through car equivalents (e) for various vehicle and turn types are given in *Table 5.2.* The meaning and the basis of deriving through car equivalents are discussed below.

The through car equivalents are better understood if considered in terms of saturation headways (headway is the time interval between the passage of two consecutive vehicles, observed at the stop line in this case). The saturation (queue discharge) headway is the minimum departure headway (in seconds per vehicle) given by:

$$h = 3600/s \qquad \text{(F.1)}$$

where s is the saturation flow in veh/h.

Note that the basic model given in Section 2 (illustrated in *Fig. 2.3*) assumes a constant saturation flow rate, s, during a fully saturated effective green period. This means that the vehicles are assumed to depart from the queue with a constant headway, h.

A through car equivalent, e, is defined in such a way that if the saturation headway is (h) seconds per *through car*, it is ($e\,h$) seconds per *vehicle.* A through car is simply a passenger car which travels straight ahead. The classification of different vehicle types as CAR's and HV's mean that all 'light vehicles' are treated as CAR's, i.e. the tcu equivalent for any light vehicle is $e = 1$. In *Table 5.2*, the through car equivalent for a through HV is $e = 2$ tcu/veh. This suggests that, if a queue consisting of through cars only is discharged with constant headways of $h = 2.4$ s, a queue consisting of through HV's only will be discharged with equal headways $e\,h = 2 \times 2.4 = 4.8$ s. From eqn (F.1), the corresponding saturation flows are $3600/2.4 = 1500$ tcu/h and $3600/4.8 = 750$ veh/h, respectively.

Because of the inverse relationship between the saturation flow rate and the saturation headway, the through car equivalents are used in saturation flow calculations as

$$s_{veh} = s_{tcu}/e \qquad \text{(F.2)}$$

In the above example, $s_{veh} = 1500/2 = 750$.

Consider now the average queue discharge headway for a mixture of through cars and through HV's. The total discharge time is ($q_1\,h_1 + q_2\,h_2$), where q_1 = number of through cars per second, q_2 = number of through HV's per second ($q = q_1 + q_2$ is the total arrival flow rate), h_1 and h_2 are the corresponding departure headways. Because $h_2 = e_{HV}\,h_1$, the average departure headway is

$$h = \frac{q_1 h_1 + q_2 e_{HV} h_1}{q} = f_c h_1 \qquad \text{(F.3,}$$

where

$$f_c = \frac{q_1 + e_{HV} q_2}{q} \qquad \text{(F.4)}$$

is the traffic composition factor which helps to express the saturation headway, h, for a mixture of through cars and through HV's in terms of the saturation headway of through cars, h_1. For example $q_1 = 900$ through cars/h, $q_2 = 100$ through HV's/h ($q = 1000$ veh/h) will give $f_c = (900 + 2 \times 100)/1000 = 1100/1000 = 1.10$. If $h_1 = 2.4$ s, $h = 1.10 \times 2.4 = 2.64$ s. The corresponding saturation flow is $s = 3600/2.64 = 1360$ veh/h.

The saturation headway, h, of a traffic stream which consists of not only different vehicle types but also different types of turning manoeuvres can be calculated similarly, i.e. $h = f_c\,h_{tcu}$, where h_{tcu} is the through car saturation headway and f_c is the traffic composition factor given by the following formula which is a generalised form of eqn (F.4):

$$f_c = \frac{\Sigma e_i q_i}{q} \qquad \text{(F.5)}$$

where $q = \Sigma q_i$ is the total arrival flow, i denotes the vehicle-and-turn type, and the through car equivalents, e_i are 1 for through cars, 2 for through HV's, 1.25 for left or right-turning cars under restricted conditions, etc., as given in *Table 5.2.*

As in eqn (F.2), the inverse relationship between the saturation flow rate and the saturation headway gives

$$s_{veh} = \frac{s_{tcu}}{f_c} \qquad \text{(F.6)}$$

A comparison of eqn (F.6) with eqn (F.2) shows that the traffic composition factor, f_c, has a similar role as the through car equivalent for a single vehicle-turn type, e. From eqn (F.5), it is seen that the traffic composition factor is a flow weighted average of through car equivalents for different vehicle-turn types in the traffic stream.

An important point about the through car equivalents is that they are related to queue discharge headways and they should not be used for any purpose other than saturation flow calculations. A common error in the past has been the calculation of traffic operating characteristics (see Section 6) using arrival flow rates expressed in tcu's, i.e. $q_{tcu} = \Sigma e_i\,q_i$. The following are some of the errors resulting from the use of this approach.

(a) The queue lengths calculated in tcu's would give misleading results if used for determining queue storage space requirements and/or short lane saturation flows because the through car equivalents are related to the queue headways (time) but not the queue spaces (distance). The degree of error is particularly high when there are opposed turns with a high e_o value.

(b) The calculation of delays and stops would lead to meaningless results if used to calculate secondary measures of performance, e.g. fuel consumption (see Section 6.6). Similarly, they would not be useful for calculating *person* delays and stops.

(c) It would be wrong to determine the opposed turn saturation flows, s_u, given in *Fig. 5.1* (or eqn (F.9) as a function of the opposing flow rate expressed in tcu's rather than vehicles because the values of s_u are related to the distribution of gaps in the opposing stream when unsaturated but not the average headways when saturated.

Such errors are prevented by a consistent use of flows and saturation flows expressed in vehicle units. This is the approach adopted in this report.

Opposed Turn Departure Characteristics

The basis of the expressions given in Section 5.6 for estimating opposed turn (left or right, giving way to pedestrians or vehicles) saturation flows is represented by the model illustrated in *Fig. F.1*. This is essentially the model used for opposed right turns by Gordon and Miller (1966), Webster and Cobbe (1966), Allsop (1977), Fambro, Messer and Andersen (1977), Peterson, Hansson and Bang (1978) and Michalopoulos, O'Connor and Novoa (1978). The model describes the opposed turn departure pattern in three different periods, namely, the saturated and unsaturated parts of the opposing movement green period and the period after the end of the green period. The opposed turn capacity per cycle is $s_u\, g_u + n_f$, where $s_u\, g_u$ and n_f are the maximum number of departures during the second and third periods, respectively. The two approximate models shown in *Fig. F.1* are used to derive the expressions given in Section 5.6 using the principle of obtaining an equivalent capacity per cycle.

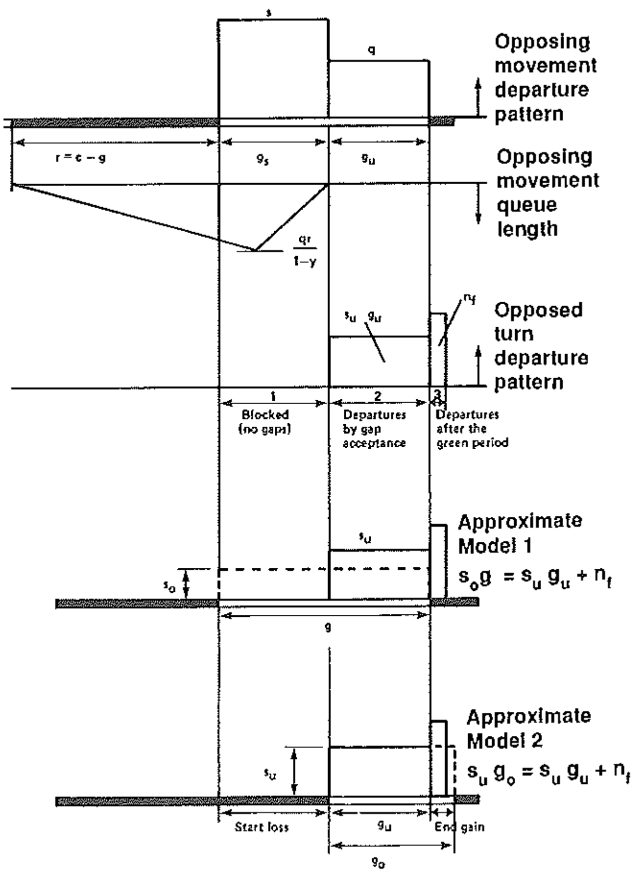


Fig. F.1 — Opposed turn model

The first model calculates an opposed turn saturation flow, s_o, during the whole of green period from $s_o\, g = s_u\, g_u + n_f$, i.e.

$$s_o = \frac{s_u\, g_u + n_f}{g} \tag{F.7}$$

To derive the opposed turn equivalent, e_o, given in eqn (5.5) for turning traffic in shared lanes, this is treated as $s_{veh} = 3600\ s_o$ and compared with a basic lane saturation flow of $s_{tcu} = 1800$. From eqn (F.2),

$$e_o = \frac{s_{tcu}}{s_{veh}} = \frac{0.5g}{s_u\, g_u + n_f} \tag{F.8}$$

For example, a through car equivalent of $e_o = 3$ means that the opposed turn saturation flow is $s_{veh} = 1800/3 = 600$ veh/h (in terms of average saturation headways, $h = 3600/1800 = 2$ s per through car and $h_o = e_o h = 6$ s per opposed turn).

The second model calculates an effective green time, g_o, by treating the blocked period as a start loss and converting n_f to an end gain value of (n_f/s_u) from $s_u\, g_o = s_u\, g_u + n_f$. Eqns (5.7) and (5.8) in Section 5 are based on this model. This model improves the estimates of delay, queue length and number of stops because it predicts a more realistic departure pattern by treating the blocked period as effectively red. It is suitable for opposed turns from exclusive lanes.

The graph in *Fig. 5.1* which gives the opposed turn saturation flow, s_u, during the unopposed part of opposing movement green period is derived from the following formula based on an exponential gap distribution assumption (Gordon and Miller 1966; Fambro *et al.* 1977; Peterson *et al.* 1978):

$$s_u = \frac{q\exp(-\alpha q)}{1-\exp(-\beta q)} \tag{F.9}$$

where

q = opposing movement flow rate (veh/s),

α = accepted critical gap (s),

β = minimum departure headway(s) for opposed turners (this corresponds to a maximum opposed turn saturation flow of $3600/\beta$ (veh/h) which could be achieved when $q = 0$).

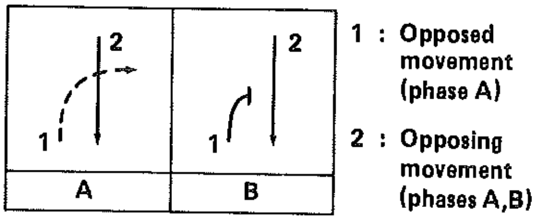
The graph in *Fig. 5.1* is based on the use of $\alpha = 5$ s and $\beta = 3$ s as used by Gordon and Miller (1966). The opposed right turn formula given by Miller (1968*a* and *b*) is equivalent to the use of these values (and $n_f = 1.5$) in eqn (F.8), but is limited to $q < 800$ veh/h. If it is considered that better opposed turn departure characteristics exist for a particular movement (e.g. due to better visibility and better geometry), $\alpha = 4.5$ s and $\beta = 2.5$ s can be used in eqn (F.9). If desired, these parameters can be measured in the field.

Typical values of e_o and g_o calculated from eqns (5.5) and (5.7) are given in *Table F.1*. These values are based on the use of $\alpha = 5$, $\beta = 3$, $n_f = 1.5$, and are for a cycle time of $c = 100$ s and opposing movement saturation flow of $s = 3000$ veh/h.

As discussed in Section 5.6, the length of the unsaturated part of the opposing movement green period to be used in eqns (5.5) and (5.7) is not exactly the same as that given by eqn (5.10) if the opposing movement is terminated later than the opposed movement. The value given by eqn (5.10) must be reduced by an amount equal to the extra time for the opposing movement in this case. If the extra time for the opposing movement is γ, and $g_u^* = (g - yc)/(1 - y)$ as given by eqn (5.10), the value of g_u to be used in eqn (5.5) or (5.7) is

$$g_u = \begin{cases} g_u^* - \gamma & \text{if } \gamma < g_u^* \\ 0 & \text{otherwise} \end{cases} \qquad \text{(F.10)}$$

The case where $\gamma < g_u^*$ is illustrated in *Fig. F.2* ($\gamma = I_B + G_B$). The case where $\gamma \geqslant g_u^*$ occurs if the saturated part of the opposing movement green period, $g_s > (I_A + G_A)$, approximately, in which case the unsaturated part of the opposing movement green period cannot be used by the opposed turners. It is also possible for the opposing movement to start before the opposed movement (phase order *B*, *A* in *Fig. F.2*). As in eqn (F.10), only the part of g_u^* which is common with the opposed movement must be used as g_u in opposed turn capacity calculations.



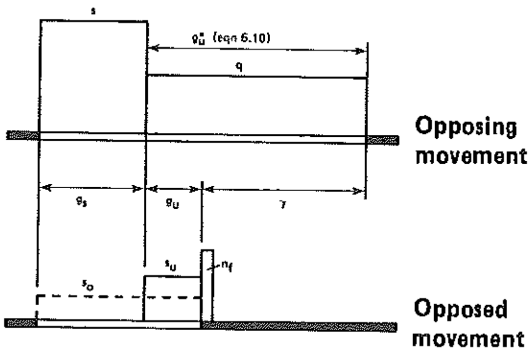


Fig. F.2 — The case where the opposed movement is stopped before the opposing movement

TABLE F.1

TYPICAL OPPOSED TURN DEPARTURE CHARACTERISTICS

($c = 100$ s, $s = 3000$ veh/h)

(a) Through car equivalent, e_o (eqn (5.5))

Green Time, g (s)	Opposing Flow, q (veh/h) 200	400	600	800	1000	1200	1400	1600
20	1.9	3.1	6.7	*	*	*	*	*
40	1.8	2.4	3.3	4.7	7.3	13.3	*	*
60	1.7	2.2	2.8	3.6	4.7	6.2	8.5	12.4

Effective opposed turn saturation flow (veh/h), $s_o = 1800/e_o$

(b) Effective green time, g_o

Green Time, g (s)	Opposing Flow, q (veh/h) 200	400	600	800	1000	1200	1400	1600
20	20	14	8	*	*	*	*	*
40	41	37	33	28	22	15	*	*
60	63	60	58	56	52	48	44	37
s_u (veh/h)	990	810	660	540	440	360	290	240

(c) Opposed turn capacity (veh/h), Q (eqn (5.9))

Green Time, g (s)	Opposing Flow, q (veh/h) 200	400	600	800	1000	1200	1400	1600
20	200	120	50	*	*	*	*	*
40	410	300	220	150	100	50	*	*
60	620	490	380	300	230	170	130	90

* The opposing movement capacity is exceeded in these cases ($qc > sg$)

F.2 TWO SATURATION FLOWS PER MOVEMENT

A turning movement may have two different saturation flow values during one signal cycle if it is allowed to filter through gaps in an opposing stream of higher priority during one phase (as an opposed movement) and to depart freely during another phase (as an unopposed movement). This applies to the following cases.

(a) The conventional fixed period leading turn (late-release) and lagging turn (early cut-off) arrangements for right turning movements (see *Fig. A.1* in Appendix A).

(b) More modern controller arrangements where the leading and lagging turn features are combined and used as separate phases of variable duration (see Appendix A, e.g. when opposed turns are allowed during phase A in *Fig. A.3(a)*).

(c) Left-turn movements opposed by pedestrians during one phase and free during another phase (e.g. movement 4 during phases C and A in *Fig. 4.1* in Section 4).

(d) Left-turn movements, under 'left turn at any time with care' or 'left turn on red' arrangements, which can depart freely during the main phase and can merge into an opposing stream during another phase.

Fig. F.3 represents a general case where the unopposed phase (A) and the opposed phase (B) may consist of more than one phase each (see Example 7 in Section D). The discussion given below applies to the reverse order B, A also (see Appendix A for a discussion of factors affecting the choice of a particular phasing order). Although *Fig. F.3* corresponds to a case where the turning traffic has an exclusive lane, the discussion applies to shared lane cases also.

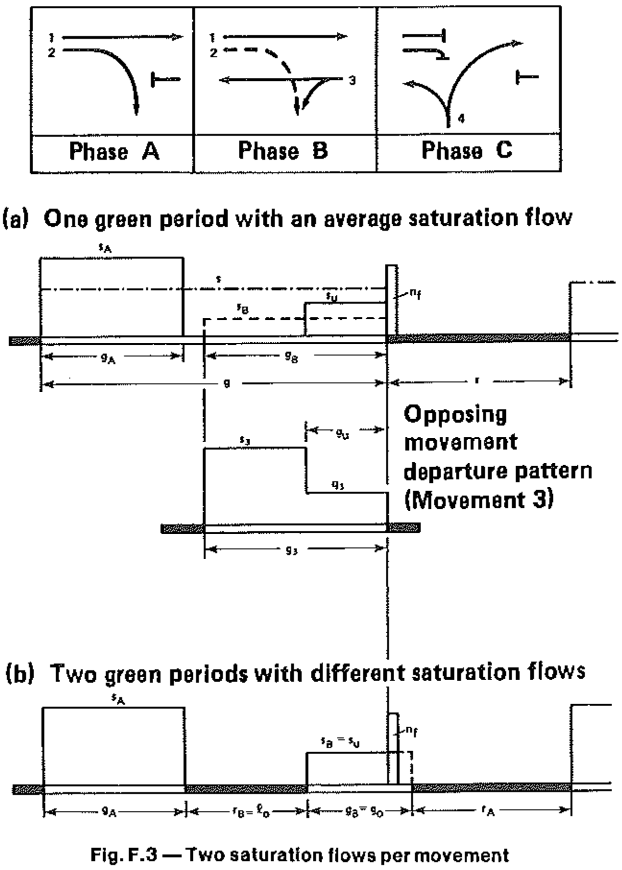


Fig. F.3 — Two saturation flows per movement

Two alternative methods for treating the problem of two saturation flows per movement are illustrated in *Fig. F.3*. The first method is to combine the opposed and unopposed periods and treat them as a single period with a capacity equivalent to the sum of the capacities available during the two periods. As shown in *Fig. F.3(a)*, this can be achieved by using an average saturation flow calculated from

$$s = (s_A\, g_A + s_B\, g_B)/g$$

where

s_A	=	normal saturation flow during the unopposed phase (A),
s_B	=	reduced saturation flow during the opposed phase (B),
g_A, g_B	=	effective green times for the unopposed and opposed phases,
g	=	$G_A + G_B + I_B$ = the green time during which the average saturation flow, s, is considered to exist.

The saturation flows (s_A, s_B) can be calculated using the general method described in Section 5, i.e. $s = s_{tcu}/f_c$, where s_{tcu} is the saturation flow in through car units and f_c is the traffic composition adjustment factor (calculated using eqn (5.4) with e_o = 1 for s_A, and with e_o given by eqn (5.5) for s_B). For an exclusive lane, $s_B = 1800/e_o$ (eqn (5.6)) is used directly. It must be noted that $n_f = 0$ must be used in the calculation of e_o, or g_o, if the opposed period is followed by an unopposed period without an intergreen (conventional early cut-off arrangement for right-turners).

An alternative method for combining the opposed and unopposed periods has been described by Allsop (1977) and Pretty (1980*b*). This method assumes that the saturation flow is s_A during a green period of length $(g_A + (s_B/s_A)\, g_B)$ if $s_A\, g_A > s_B\, g_B$. Otherwise, the saturation flow is assumed to be s_B during a green period of length $(g_B + (s_A/s_B)\, g_A)$.

The above methods are useful when the opposed and unopposed phases are *consecutive*. The advantage of having a single green period is that the formulae for delay, number of stops and queue length given in Section 6 are directly applicable. However, a different method is needed when the opposed and unopposed periods are *non-consecutive* (see Example 7 in Appendix D). This method treats the two periods separately, each with its own saturation flow, green and red times (*two green periods per cycle*). It is a general method, and is recommended for use in all cases because it facilitates an explicit treatment of minimum green times for the two periods, offers advantages in terms of critical movement identification and signal timing calculations, and provides better estimates of delay, queue length and number of stops (only the calculation of uniform terms of these statistics differ in the case of two green periods per cycle; see *Fig. D.11* in Appendix D). This method is illustrated in *Fig. F.3(b)* for a case where the opposed and unopposed periods are consecutive and an exclusive turn lane is available.

The calculation of the saturation flows (s_A, s_B) is as described above. However, in the case of opposed turns from exclusive lanes, it is recommended that $s_B = s_u$ (from *Fig. 5.1*, or eqn (F.9)) and $g_B = g_o$ (from eqn (5.7)) are used for the opposed phase. As shown in *Fig. F.3(b)*, this means that the saturated part of the opposing movement green period is treated as effectively red for the opposed movement (i.e. different lost times for the two periods).

When the two green periods are treated separately, the flow ratios must be calculated from:

$$y_A = \frac{q g_A}{s_A g_A + s_B g_B}$$

$$y_B = \frac{q g_B}{s_A g_A + s_B g_B} \qquad \text{(F.11)}$$

where q is the arrival flow rate for the opposed/unopposed movement in question (use q and s in consistent units).

Iterative Calculations

For all methods described above, it is necessary to know the signal timings in advance, and an iterative procedure is required to find a solution in which the signal timings and saturation flows are in balance. It is recommended that the initial timings are calculated assuming no opposed turns (during phase B in *Fig. F.3*). The next step is to calculate the reduced saturation flow s_B (and $g_B = g_o$ in the case of exclusive lanes) using the initial timings. The critical movement identification and signal timing calculations can then be repeated using the revised flow ratios and lost times based on the above methods. The calculations can be stopped when no significant changes are obtained in signal timings. The number of iterations can be decreased by keeping the cycle time constant after one or two iterations (see Examples 6 and 7 in Appendix D).

F.3 SIGNAL TIMINGS WHEN THE SATURATION FLOW FALLS OFF DURING THE GREEN PERIOD

In the case of a short lane, the saturation flow can fall off abruptly during the green period as discussed in detail in Section 5.8. The effective (reduced) saturation flow of the short lane can be predicted as a function of the green time using eqn (5.18) or (5.19). However, signal design calculations involve the determination of green times and the calculations may be complicated when both saturation flows and green times are variable. The following method can be used as a practical solution to this problem.

(a) Calculate full movement saturation flows assuming that short lane effects do not occur. Determine critical movements and calculate cycle time and green times accordingly (using the method described in Section 4).

(b) Determine if short lane effects occur for any critical movement using the signal timings calculated in (a) in accordance with the method described in Section 5.8. If short lane effects are found to occur, calculate the cycle time which yields the minimum intersection degree of saturation with a fixed green time which gives full saturation flow for the movement to which the short lane is allocated. Determine the green times for other movements accordingly. The formulae to implement this method and an example are given below.

Suppose the flow ratio of the movement to which the short lane is allocated is y ($= q/s$, where s is the unreduced saturation flow), the sum of flow ratios of all other critical movements is Y', and hence, the intersection flow ratio is $Y = Y' + y$ (these are as determined in (a) above). The cycle time which gives the minimum intersection degree of saturation with a fixed green time, g, for the movement whose flow ratio is y can be calculated from:

$$c = (Yg/y) + L \qquad \text{(F.12)}$$

and the corresponding intersection degree of saturation is:

$$X = Y + (y/g)\,L \qquad \text{(F.13)}$$

where L is the intersection lost time.

The fixed green time, g, to be used in the above formulae is the maximum green time which would cause no saturation flow loss. This is g_l shown in *Fig. 5.5*, and is given by:

$$g = \frac{3600\,D}{j\,s_l} \qquad \text{(F.14)}$$

where

D	=	short lane length (m),
j	=	average queue space (m/veh), and
s_l	=	unreduced saturation flow for the short lane (veh/h).

The degree of saturation, X, from eqn (F.13) is the same for all movements, i.e. the cycle time from eqn (F.12) corresponds to an equal degree of saturation solution. If X is found to be less than an acceptable maximum degree of saturation, x_p, for the movements whose flow ratio is Y', an unequal degree of saturation solution can be obtained using a cycle time given by:

$$c = \frac{L + g}{1 - U'} \qquad \text{(F.15)}$$

where $U' = Y'/x_p$ and L, g are as in eqn (F.12). The cycle time, and hence, the red time and the degree of saturation are further reduced for the movement to which the short lane is allocated using this solution.

Example

Consider a case where the movement to which a short lane is allocated has the following characteristics: q = 1200 veh/h, s = 3360 veh/h (full two-lane saturation flow), y = 1200/3360 = 0.36, D = 100 m, j = 7.2 m, s_l = 1680 veh/h, and hence, the maximum green time to give full saturation flow (from eqn (F.14)) is g_l = 3600 x 100/(7.2 x 1680) = 30 s. The other critical movements have a total flow ratio of Y' = 0.45. The total lost time for all critical movements is L = 10 s.

First calculate the cycle time which yields a maximum intersection degree of saturation, X_p = 0.90 assuming full saturation flow for the movement to which the short lane is allocated. Since $Y = Y' + y$ = 0.45 + 0.36 = 0.81 and L = 10 s, eqn (7.2) gives c = 10/(1 − 0.81/0.90) = 100 s. The total available green time is $c - L$ = 90 s, therefore the green times are g = (90/0.81)0.36 = 40 s and g' = (90/0.81)0.45 = 50 s.

Now determine if short lane effects occur with these timings. The critical queueing distance from eqn (5.20) is $D_c = 7.2 \times (1200/3600) \times (100 - 40)/2(1 - 0.36) = 113$ m $> D = 100$ m. To minimise the short lane effects, calculate the cycle time from eqn (F.12) with a fixed green time $g = g_1 = 30$ s:

$$c = (0.81 \times 30/0.36) + 10 = 78 \text{ s}.$$

The intersection degree of saturation from eqn (F.13) is

$$X = 0.81 + (0.36/30)10 = 0.93.$$

The total green time for the movements whose flow ratio is $Y' = 0.45$ is $g' = c - L - g = 78 - 10 - 30 = 38$. If $x_p = 0.98$ was acceptable for these movements, the cycle time could be reduced to $c = (10 + 30)/(1 - 0.45/0.98) = 74$ s (eqn (F.15)). The degree of saturation for the movement using the short lane would then be $x = y/(g/c) = 0.36/(30/74) = 0.89$.

Similar fixed green time methods can be used for other cases of saturation flow falling off during the green period (Section 5.10).

APPENDIX G

DECELERATION-ACCELERATION DELAY AND CORRECTION FOR PARTIAL STOPS

The analytical expressions of delay at traffic signals measure the delay at the stop line by assuming infinite deceleration and acceleration rates. This can be observed from *Fig. G.1* which shows the time-distance trajectory of a vehicle which makes a complete stop by decelerating from a constant cruising speed to zero speed and accelerating back to the cruising speed, idling for a period of time between the deceleration and acceleration manoeuvres (the slope of the time-distance trajectory is the speed of vehicle). The parameters shown in *Fig. G.1* are:

v_c	=	cruising speed,
ℓ	=	cruising distance,
t_c	=	ℓ / v_c = cruising time (uninterrupted travel time),
d	=	delay time,
t	=	$t_c + d$ = travel time (interrupted),
t_a, t_b	=	deceleration and acceleration times,
ℓ_a, ℓ_b	=	deceleration and acceleration distances,
d_s	=	stopped delay (idling) time, and
d_h	=	$d_a + d_b$ = deceleration-acceleration delay.

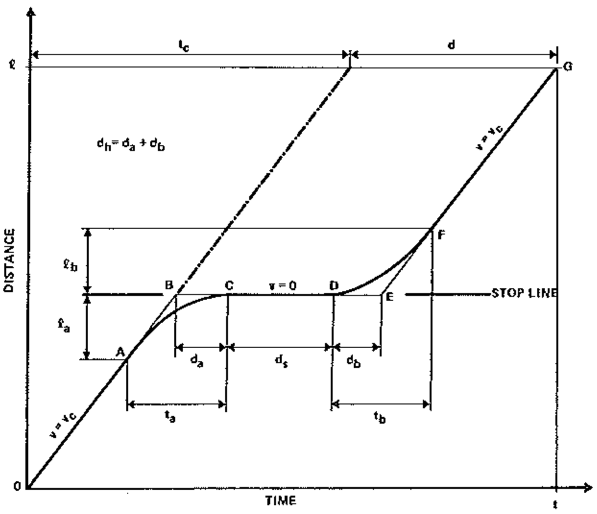


Fig. G.1 — Time-distance trajectory of a vehicle which makes a complete stop with a stopped time delay

It is seen that the delay experienced by the vehicle, i.e. the difference between the uninterrupted and interrupted travel times, can be measured at the *stop line* by assuming infinite deceleration and acceleration rates. The delay formula presented in Section 6 gives the stop-line delay based on this method. For the purposes of predicting the number of stops, and the relevant secondary measures of performance such as fuel consumption, cost, etc., it is necessary to distinguish between the stop-line delay and the actual stopped delay (idling) time. As seen in *Fig. G.1*, the stopped delay time for an individual vehicle (d_{si} = CD) is the difference between the stop-line delay (d_i = BE) and the deceleration-acceleration delay (d_h) which is part of the total deceleration-acceleration time ($t_a + t_b$). Using the average delay and stop rate values, d and h, as given by eqns (6.4) and (6.5) in Section 6, the *average* stopped delay time per vehicle, d_s, can be calculated from:

$$d_s = d - h\,d_h \qquad \text{(G.1)}$$

It should be noted that d_s, d and h are average values for all vehicles, stopped and unstopped, whereas d_h applies to stopped vehicles only. The relationship between eqns (6.14) and (6.15) which describe the elemental model given in Section 6 is based on this formula (note that $D_s = q d_s$, $D = qd$).

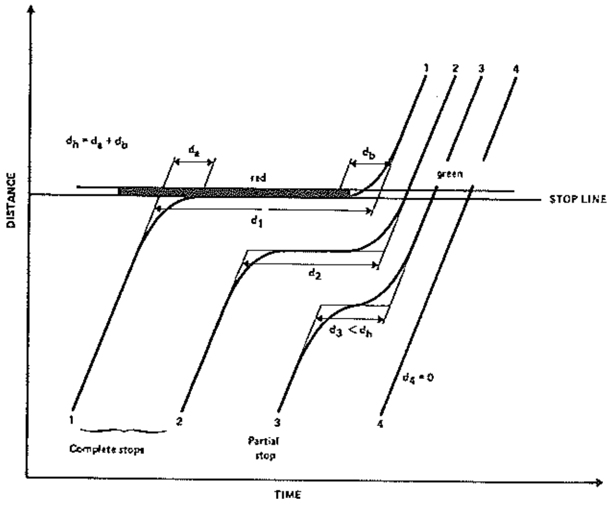


Fig. G.2 — Time-distance diagram to illustrate complete and partial stops

As a result of the stop-line method of measuring delay, a vehicle which is delayed only for a short time without coming to a full stop is counted as a stopped vehicle. To describe such a case, a vehicle whose stop-line delay is less than the deceleration-acceleration delay for a complete stop, d_h, is said to be subject to a *partial stop*. An example is given in *Fig. G.2*. Let us assume that d_h = 12 s, and delays for vehicles 1 to 4 are:

$$d_1 = 32\,\text{s},\ d_2 = 18\,\text{s},\ d_3 = 9\,\text{s},\ d_4 = 0.$$

In this case, vehicles 1 and 2 make complete stops ($d_{1,2} > d_h$), vehicle 3 makes a partial stop ($d_3 < d_h$), and vehicle 4 is unstopped. The partial stop value for vehicle 3 can be found as d_3 / d_h = 9/12 = 0.75 complete stops. The stop rate for this example is therefore

$$h = (1 + 1 + 0.75 + 0)/4 = 0.69 \text{ stops/veh.}$$

The average stop-line delay is:

$$d = (32 + 18 + 9 + 0)/4 = 14.8\ \text{s}$$

From eqn (G.1), the average stopped delay time per vehicle is

$$d_s = 14.8 - 0.69 \times 12 = 6.5\ \text{s}$$

This could, of course, be calculated as the average of individual vehicle values:

$$d_s = (20 + 6 + 0 + 0)/4 = 6.5\ \text{s}$$

The value of d_h can be estimated as follows. The time for a vehicle to decelerate from speed v down to speed v', and to accelerate back to speed v is:

$$t_a + t_b = (v - v')\left(\frac{1}{a_1} + \frac{1}{a_2}\right) \quad \text{(G.2)}$$

where the deceleration and acceleration rates, a_1 and a_2 are both positive and assumed to be constant. It can be shown that the deceleration-acceleration delay for a partial stop is equal to half the value of acceleration-deceleration time given by eqn (G.2), that is:

$$d_h' = \left(\frac{v - v'}{2}\right)\left(\frac{1}{a_1} + \frac{1}{a_2}\right) \quad \text{(G.3)}$$

Therefore, the deceleration-acceleration delay for a complete stop ($v' = 0$) is:

$$d_h = \frac{v}{2}\left(\frac{1}{a_1} + \frac{1}{a_2}\right) \quad \text{(G.4)}$$

Further, assuming $a_1 = a_2 = a$ as an average rate for all vehicles, and replacing v by the cruising speed, v_c, the simpler formula:

$$d_h = v_c / a \quad \text{(G.5)}$$

is obtained. For example, for a cruising speed of v_c = 60 km/h = 16.7 m/s and a = 1.4 m/s^2 give a deceleration-acceleration delay of d_h = 12 s.

It can be assumed that the proportion of stop to be associated with d_h' for a partial stop will vary linearly between one for $d_h' = d_h$ (complete stop) and zero for $d_h' = 0$ (unstopped vehicle), i.e. it is given by (d_h'/d_h). For example, 9/12 = 0.75 complete stops for vehicle 3 in *Fig. G.2* as discussed above (see Richardson 1979; Akcelik 1980*a*).* The stop rate for this example counting the stop made by vehicle 3 as a complete stop is 3/4 = 0.75 stops/veh whereas the stop rate with due allowance for the partial stop effect is 2.75/4 = 0.69 as found above. Therefore, the overall correction factor for partial stops in this example is 0.69/0.75 = 0.92.

As a general-purpose partial stop reduction factor, a constant value of 0.9 is used in the stop rate formula given in Section 6 (eqn (6.5)). This is a typical value which corresponds to desirable operating conditions during peak periods (degrees of saturation in the range from 0.70 to 0.90 and cycle times in the range from 80 to 120 s).

* Improved formulae for acceleration and deceleration rates and profiles were developed and used in the SIDRA program. Refer to:

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APPENDIX H

VEHICLE-ACTUATED SIGNALS

The purpose of the following discussion is to describe how the basic vehicle-actuated control mechanism works, to indicate the relevance to the vehicle-actuated control case of the methods given in Sections 6 and 7 for predicting traffic operating characteristics and for calculating signal timings, and to present suggestions for further research to facilitate better use of vehicle-actuated controllers in practice.

H.1 CONTROLLER SETTINGS

At vehicle-actuated signals, the green times, and hence the cycle time, are determined according to the vehicle demands registered by detectors. Phase sequence may be fixed or variable as discussed in Appendix A. In the case of fixed-sequence phasing, a phase can be skipped when there is no demand for it. The running phase waits in the *rest* position when no conflicting demand is present unless there is an automatic *call* for another phase.

For a given phasing system, efficient operation of vehicle-actuated signals depends on the values of various controller settings. The three basic controller settings which determine the length of the green period are *minimum green, vehicle interval* (the terms gap time, vehicle extension, unit extension, etc. are also used) and *maximum extension* (or maximum green) settings. Modern controllers have additional settings which differ according to the type of controller (Pak-Poy and Associates 1975; Staunton 1976). The location, number and other characteristics of detectors affect the choice of vehicle-actuated settings also. It is therefore difficult to give general-purpose rules for choosing the values of vehicle-actuated controller settings, which can be used for any controller and location. However, some suggestions are presented below which are based on a limited amount of research published in the literature (Grace, Morris and Pak Poy 1964; Webster and Cobbe 1966; Morris and Pak-Poy 1967; Pak-Poy and Associates 1975; Staunton 1976).

The green period allocated to a movement comprises a minimum green period and a green extension period which is subject to an upper limit (maximum green setting). In modern controllers, the minimum green period comprises a fixed period and an additional variable period which is determined by the number of vehicle actuations (after the first vehicle) during the red period. The *fixed minimum green* and *vehicle increment* settings must be chosen to be sufficiently long for the clearance of vehicles waiting *in one lane* between the detection point and the stop line. These settings must be chosen with care so as to avoid unduly long green times which result in loss of efficiency, especially when detections in more than one lane contribute to the length of the variable minimum green period.

As illustrated in *Figs H.1* and *H.2* (from Staunton 1976), detection of each additional vehicle extends the green period by the vehicle interval setting. The controller starts timing a new vehicle interval at each vehicle actuation. The green period terminates when the time between successive vehicle actuations exceeds the vehicle interval setting (*gap change*) or the total green extension time equals the maximum extension setting (*maximum change*).

The choice of the vehicle interval setting is critical in determining the length of the green period, and hence the efficiency of operation. A basic control mode uses a fixed vehicle interval setting whereas modern controllers provide facilities for automatic reduction of vehicle interval (*gap reduction*) according to the traffic flow rate on the running phase (inaptly called 'density'), the number of vehicles waiting on the red phase, waiting time on the red phase, or combinations and variations of these algorithms (e.g. 'volume-density' and 'headway-density' controllers). A summary of various gap reduction and other vehicle-actuated controller types can be found in Staunton (1976). Various recommendations for controller settings in simple (basic), volume-density, headway-density, and a 'variable-maximum' controller are given by Pak-Poy and Associates (1975). These are based on the use of delay criterion only. In an earlier publication, Morris and Pak-Poy (1967) considered the effect of vehicle interval on the percentage of stopped vehicles also. Some of the findings of this and other published work (referred to above) on the effect of vehicle interval setting are as follows.

(a) There is an optimum value of the vehicle interval setting which minimises delay.

(b) The optimum vehicle interval becomes smaller and its effect becomes more critical as traffic flows increase (*Figs H.3* and *H.4*); hence for relatively high flows it is important that the vehicle interval setting chosen is not too large.

(c) The effect of increasing the vehicle interval is to increase the cycle time and to reduce the percentage of vehicles stopped (*Fig. H.5*); using large values of vehicle interval setting is advantageous from this viewpoint especially for low flows where delays are not very sensitive to the value of the vehicle interval as seen in *Fig. H.4* (note that the results in *Figs H.3* to *H.5* were obtained with no maximum setting control).

Based on these findings, and considering the importance of vehicle stops in relation to fuel consumption, cost and safety (see Sections 6 and 7), in particular at high speed locations as emphasised by Staunton (1976), a compromise value of 3 or 4 s appears to be a good choice as a *fixed* vehicle interval setting. However, the effect on the actual number of stops rather than the proportion stopped, especially under heavy flow conditions (see Section 6.3), needs to be assessed. In practice, the intersection geometry and traffic composition as well as the number and type of lanes (turning or through) per phase at a particular site (and at different times) must also be taken into account in choosing the value of vehicle interval setting.

It is apparent that basic vehicle-actuated control with a fixed vehicle interval setting will not produce efficient operating conditions (delays and stops) during both light and heavy flow periods. Gap reduction features of modern controllers are useful for this reason. However, there are difficulties in finding and maintaining optimum adjustment of sophisticated equipment in practice as pointed out by Staunton (1976). This applies to a widely used vehicle-actuated controller type in Australia which has 'waste' and 'headway' settings in addition to the normal 'gap' (vehicle interval) setting. With this controller, when the time between successive vehicle actuations exceeds the headway setting, the excess time (waste increment) is accumulated. In addition to normal gap

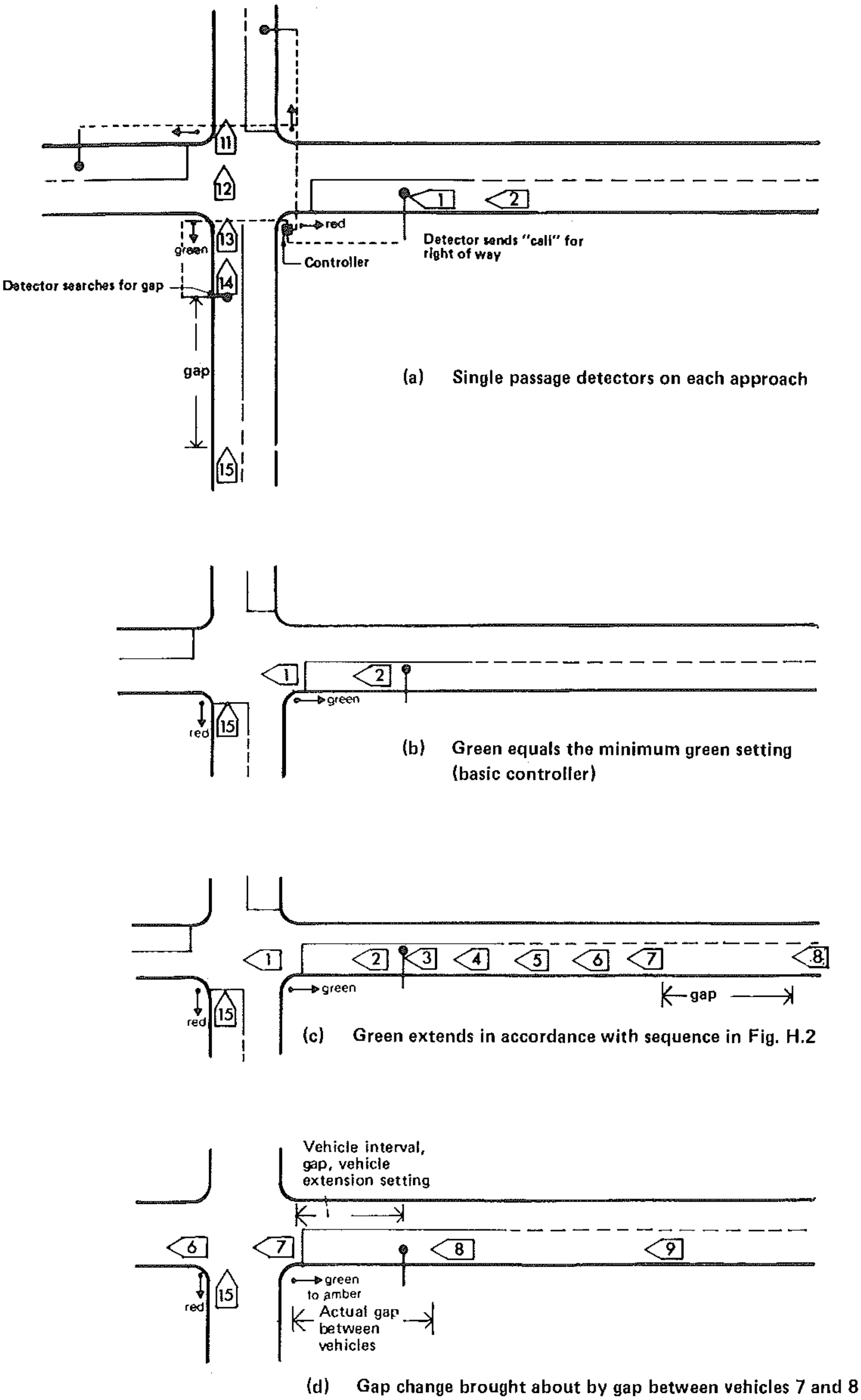

Fig. H.1 — Vehicle-actuated control by vehicle interval
(from Staunton 1976)

change, the green period is terminated when the accumulated waste increments equal the waste time setting (*density change*). Staunton points out that this corresponds to gap reduction earlier in light traffic and later in heavy traffic which may not be efficient in terms of the earlier discussion. Huddart (1980) tends to justify the 'waste' method on the basis that the controller provides a longer green time when there is more traffic, but he points out that it may be slow in identifying the end of a dense traffic platoon. Huddart suggests that an alternative controller in which the allowable gap is reduced according to the 'density' derived from the number of vehicles in the previous 10 s is more effective in immediately identifying the end of a platoon. Further research is recommended in order to develop rules, if possible, for the efficient use of waste time controllers.

The maximum green (or extension) setting (see *Fig. H.2*) corresponds to the green time calculated as explained in Sections 4 and 7 of this report. The value of the setting to be used in practice must be chosen with due consideration to traffic flows at different times (morning peak, evening peak, day off-peak, night off-peak, weekend and shopping periods) and to the peaking characteristics of traffic during peak flow periods. The choice of the 'design period' as a basis of green time calculations is important in this respect (see Section 8.2). The objective should be to obtain green times which are not too restrictive for maximum possible flow rates (e.g. during peak 10-minute period). On the other hand, long maximum settings coupled with a bad choice of the vehicle interval and other controller settings can lead to unduly long green and cycle times resulting in inefficient operation during a larger proportion of the time.

In group controllers, maximum settings can be determined directly from the calculated movement green times. In phase controllers, maximum settings for phases must be chosen in such a way that various overlap movement green times can be achieved in practice. When maximum setting means maximum extension setting rather than maximum green setting, the value to be chosen corresponds to the green time calculated less the minimum green setting including the variable part if applicable (see *Fig. H.2*).

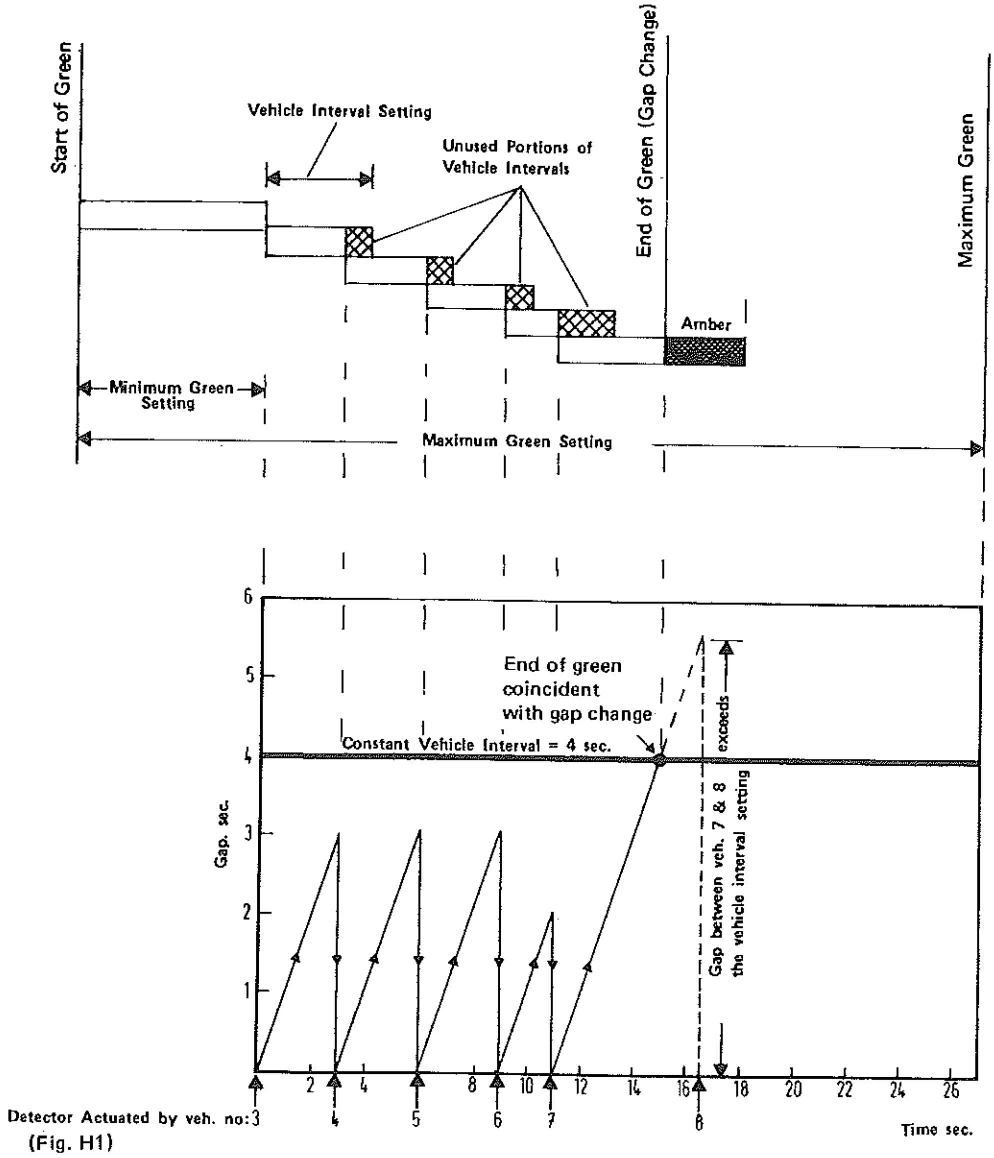

Fig. H.2 — Extension sequence in a basic vehicle-actuated controller (from Staunton 1976)

For further reading on *traffic-responsive* control (in a broader sense than vehicle-actuated control), the reader is referred to Al-Khalili (1980), Bang and Nilsson (1976*a* and *b*), de la Breteque (1979), Nip (1975) and Robertson and Bretherton (1974).

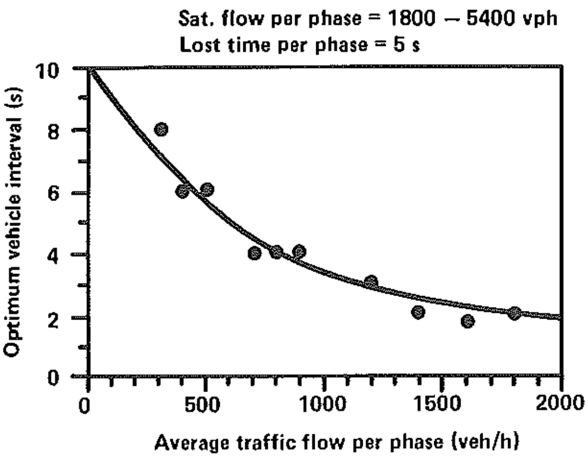

Fig. H.3 — Variation in optimum vehicle interval with average traffic flow per phase at a simple vehicle-actuated two-phase traffic signal (from Morris and Pak-Poy 1967)

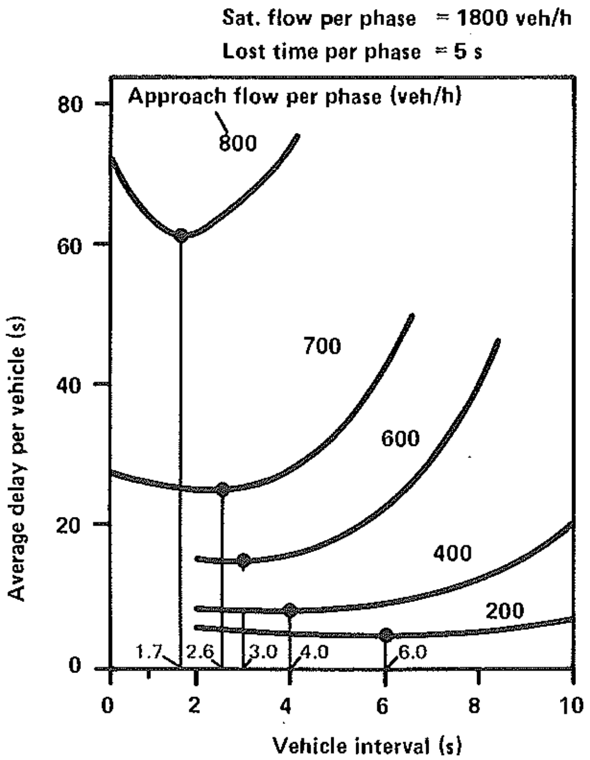

Fig. H.4 — Variation in average delay with vehicle interval at a simple vehicle-actuated two-phase traffic signal (from Morris and Pak-Poy 1967)

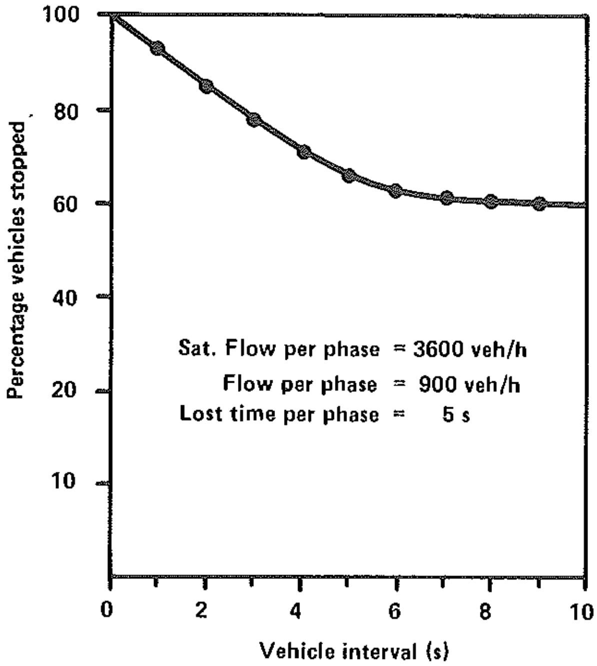

Fig. H.5 — Variation in the proportion of vehicles stopped with vehicle interval at a simple vehicle-actuated two-phase traffic signal (from Morris and Pak-Poy 1967)

H.2 OPERATING CHARACTERISTICS

Simple formulae similar to those given in Section 6 are not available for predicting the operating characteristics of traffic at vehicle-actuated signals with due allowance for minimum and maximum green, and vehicle interval (gap time) settings. Various simple cases have been studied by Garwood (1940), Darroch *et al.* (1964), Dunne (1967), Potts (1967), Newell (1969*b*), Little (1971), Lehoczky (1972) and Cowan (1978). It has been shown by Newell and Osuna (1969) that, in realistic intersection cases, the formulae would be very long, involve many parameters, and would need more than simple algebra for their use. The formulae to represent the operation of modern vehicle-actuated controllers which involve a large number of control parameters are unlikely to be developed even for very simple intersection cases. However, some qualitative assessment and rule-of-thumb methods can be developed to facilitate an evaluation of vehicle-actuated signal operations.

Because the green times and cycle time are related to traffic demands, the average overflow queue is expected to be closer to zero at vehicle-actuated signals. Therefore, the delay, number of stops and queue length are, in general, expected to be smaller than those for fixed-time signals under the same flow conditions. Although this is true for light to moderate flow conditions, vehicle-actuated signals will, in effect, be operating as fixed-time signals under heavy flow conditions, with most green times running to their maximum values. In this case, the formulae given in Section 6 can be used for vehicle-actuated signals also, and this is expected to provide a satisfactory basis for evaluating the relative merits of alternative signal designs in terms of peak period operations.

However, the difficulty is in defining precisely when the vehicle-actuated signals start operating in a fixed-time fashion, because this depends not only on flow levels but also the vehicle interval, minimum and maximum green and other settings.

There appear to be two simple methods which are feasible. Webster and Cobbe (1966) suggested that the use of delay-minimising signal settings (see Section 7) in the delay formula for fixed-time signals would give delays roughly the same as the delays at vehicle-actuated signals with a vehicle interval setting of about 4 s. Therefore this method can be used as a first approximation to delays at vehicle-actuated signals. The second method has been proposed by Courage and Papapanou (1977). The adoption of this method to eqn (6.3) would be as follows.

(a) Calculate the uniform delay (first term of eqn (6.3)) using signal timings based on the practical cycle time formula (eqn (7.2)) of Section 7 with $x_p = 1.0$.

(b) Calculate the random delay (second term of eqn (6.3)) using signal timings based on maximum green and cycle times as determined by vehicle-actuated signal settings.

These methods could be extended to the use of the stop rate and queue length formulae given in Section 6. However, further research is needed to validate these methods and to define the conditions in which they are applicable.

APPENDIX I

CO-ORDINATED SIGNALS

I.1 SIGNAL TIMINGS

Signal control of a *network* of, i.e. two or more closely-spaced intersections, differs from *isolated* control because of the need for *co-ordination (linking)* of signals to allow for traffic interaction between successive intersections. Signal co-ordination helps to avoid frequent stopping and unnecessary delays to *platoons* (bunches) of vehicles formed at upstream signals. It also helps to prevent queues at a downstream intersection extending back and reducing the saturation flow (hence capacity) of an upstream intersection, which is of particular importance where there is limited queue storage space between intersections.

Signal co-ordination is accomplished essentially by operating all signals in the area with a *common* cycle time and by staggering the green periods in relation to each other according to the speed of vehicle platoons so as to obtain a *progression* of green periods along the road. The effectiveness of signal co-ordination depends on the degree to which vehicle platoons remain compact during travel between intersections and the amount of turning traffic from side roads hindering the progression of main road platoons. The *area* for signal co-ordination is therefore determined according to the distances between intersections, the relative proportions of turning and through flows and the amount of interference to free travel of vehicle platoons. For more efficient control, a large area can be divided into smaller *sub-areas* each with its own common cycle time according to the relative levels of congestion in, and hence the cycle time requirements of, these areas.

A basic form of co-ordinated control is fixed time, i.e. the cycle time and green times are predetermined and of fixed duration during the control period. Different *signal co-ordination plans* are prepared for different flow periods (typically morning peak, evening peak and off-peak periods). The co-ordination plans are implemented by means of a master controller which instructs local intersection controllers to change phases. In some cases, local vehicle-actuation is allowed within the constraints of the master plan, that is green times may vary although the cycle time is fixed but this is not necessarily more efficient than fixed-time co-ordination. The term *Area Traffic Control* is used for co-ordinated signal control by the use of a central computer (see Luk 1979; Middleton and Luk 1979; OECD Road Research Group 1972). A *dynamic* co-ordination system (SCAT: Sydney Co-ordinated Adaptive Traffic system) which is in use in Australia (Sims and Dobinson 1979) selects and adjusts basic co-ordination plans in each area, and connects and dis-connects various sub-areas according to measured traffic conditions. The SCAT system provides a sophisticated computer control system by which different control strategies can be implemented with relative ease.

Preparation of a signal co-ordination plan involves the calculation of a common area cycle time, the green times for each intersection in the area, and the *offsets*, i.e. the differences in starting times of the green periods at successive signals (see *Figs I.1* and *I.2*). In addition to normal movement and phase data which are used for calculating cycle time and green times for each intersection (see Section 4.2), the following data are required for calculating signal offsets for optimum progression.

(a) A *network diagram* in which movements and intersections are represented as *links* and *nodes* (see *Fig. I.3*).

(b) *Link distances*, i.e. stop-line to stop-line cruising distances, l_c (m).

(c) Average platoon *speeds*, V_c (km/h), under uninterrupted cruising conditions (i.e. not including signal delays), or average cruising *times*, $t_c = 3.6\, l_c / V_c$ (s).

(d) The sources and rates of *upstream flows* (i.e. platoons entering each link).

(e) *Platoon dispersion* characteristics to allow for different vehicle speeds in the platoon if this is to be (and can be) taken into account in the calculations (see *Fig. I.5*).

The calculation of signal co-ordination plans which yield optimum progressions is not an easy task especially for closed-loop network formations, e.g. two-way progressions on an arterial road, grid networks, etc. For this reason, various computer methods have been developed. At present, one of the most widely used computer programs is TRANSYT (Robertson 1969; Robertson and Gower 1977; Vincent, Mitchell and Robertson 1980) which determines optimal offsets and green times for a specified cycle time using a simulation-search technique. The latest version of TRANSYT (version 8 (Vincent *et al.* 1980)) has a cycle time selection facility which is based on the treatment of each intersection as an isolated one. The 6N version of TRANSYT (Akcelik 1979*b*) has a facility to search for an optimum cycle time allowing for co-ordination effects but the computing time may be prohibitive if a good initial cycle time is not specified. The TRANSYT program allows for different flows (platoons) entering each link from up to four upstream links and predicts dispersion of each platoon during its travel to the next intersection. It also allows for mid-block flow losses and gains. Although it limits the amount of flow which can enter from congested upstream links to their capacity values, it does not simulate backward spread of congestion from one upstream link to another. However, TRANSYT 8 has a 'maximum queue constraint' facility which partly takes account of this problem.

A manual method is described and an example given below for preparing signal co-ordination plans in simple network cases, e.g. two or three intersections. The method can also be used for preparing data for a program such as TRANSYT in complicated network cases:

(a) good signal timings used as initial values in the optimisation process can help reduce computing time; and

(b) signal timings can be specified as fixed values to act as constraints on the optimisation, e.g. to achieve offsets which minimise the interference from downstream queues as explained in Section I.1.2.

I.1.1 Common Cycle Time and Green Times

Since the approximate optimum cycle time formula given in Section 7 (eqn (7.1)) is not valid for co-ordinated signals, it is recommended that a practical cycle time, c_p, is calculated using eqn (7.2) for each intersection in the area. This is the cycle time which ensures that the degrees of saturation of all movements are below specified maximum acceptable values, $x < x_p$. A recommended general-purpose value which can be used for all movements is x_p = 0.90, although 0.95 can be used as an absolute limit under heavy demand conditions (Section 3.3). To achieve better progression, the main road traffic can be allocated lower x_p values and the side road traffic can be given higher x_p values (e.g. 0.85 and 0.95, respectively) provided longer delays and queue lengths are acceptable on side roads. This is an unequal-degree-of-saturation method which is implemented automatically using the critical movement identification and signal timing calculation method given in Section 4.

The intersection with the largest cycle time, c_p, is the *critical intersection* in the area. This cycle time can be used as the common area cycle time, i.e. all intersections in the area are to operate with this cycle time, provided that $c_p < c_{max}$, where c_{max} is a specified maximum cycle time (normally 120-150 s). It is recommended that the practical cycle time of the critical intersection is rounded up to the nearest multiple of 5 or 10 s before it is used as the common cycle time, c (if $c_p > c_{max}$, use $c = c_{max}$). It is possible to operate some minor intersections in the area with half the value of c (double cycling) if the practical cycle times calculated for these intersections do not exceed the value of $c/2$. This is useful, especially for decreasing delays to side road traffic and pedestrians, but vehicle stops are increased on the main road. Alternatively, the co-ordination areas can be re-arranged to achieve a better grouping of the intersections with similar cycle times (c_p) as far as the network geometry permits. Wherever possible, measures should be taken to decrease the cycle time requirement of the critical intersection (see Section 8.4) so as to improve the performance of traffic not only at the critical intersection but also in the control area as a whole.

The green times and phase change times are calculated for each intersection in the area using the common cycle time according to the method described in Section 7. The phase change times must then be adjusted to achieve co-ordination of successive signals as described below.

I.1.2 Offsets and Phase Change Times

The offset, i.e. the difference between the starting times of the upstream and downstream green periods, which provides a reasonable progression for a platoon of vehicles can be calculated from:

$$O = t_c + (g_u - g_d)/2 \qquad (I.1)$$

where t_c is the average cruising (uninterrupted travel) time and g_u, g_d are the upstream and downstream green times (all parameters are in seconds). The effect of this formula is to synchronise the mid-points of successive green periods using the average cruising time for the platoon under consideration.

An example is illustrated in *Fig. I.1* where t_c = 15 s and the green times at locations A and B are g_A = 40 s and g_B = 60 s (the same for opposite directions at each location). The offset for travel from A to B is found from eqn (I.1) using g_u = 40 and g_d = 60, i.e. O_{AB} = 15 + (40 − 60)/2 = 15 − 10 = 5 s. Similarly, the offset for travel from B to A is found using g_u = 60 and g_d = 40, i.e. O_{BA} = 15 + (60 − 40)/2 = 15 + 10 = 25 s. The two cases are illustrated in *Figs I.1a* and *b*, respectively. The example shows the difficulty of implementing desirable offsets for platoons in both directions not even considering the platoons from side roads (note that $O_{BA} \neq O_{AB}$ by definition in each case). It is therefore necessary to adopt either a compromise/optimum solution or a selective progression solution which favours chosen platoons.

When preparing signal co-ordination plans for light traffic conditions (off-peak periods) it may be preferable to use an offset equal to the average

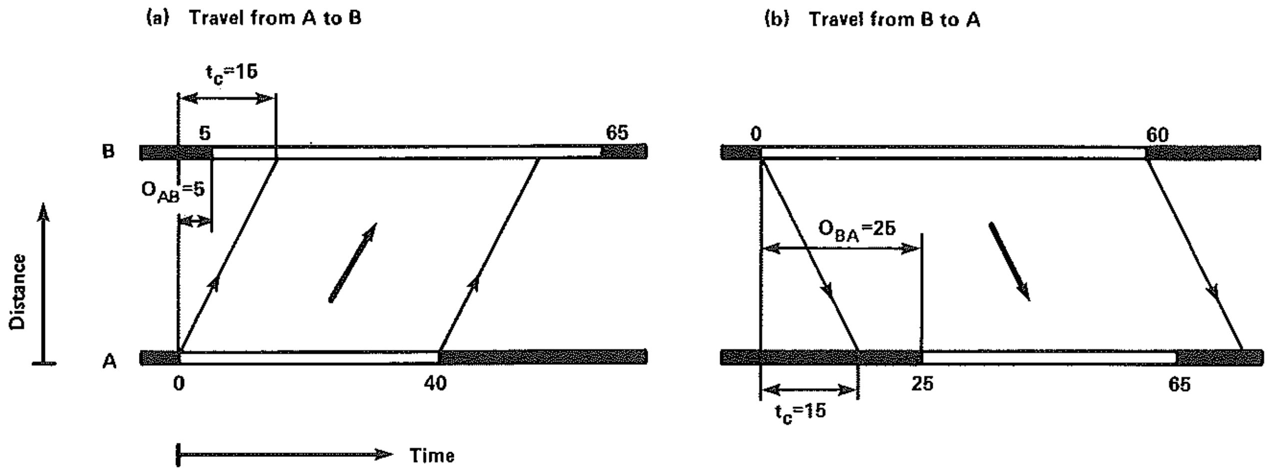

Fig. I.1 — Offsets for co-ordinated signals (eqn (I.1)) cruising time, $O = t_c$, so as to provide for uninterrupted travel of the front of a platoon (note that t_c used for an off-peak plan may be smaller than the value used for a peak period plan). For heavy traffic conditions, the queue in front of an approaching platoon must be taken into account. The downstream queue may be the result of a high degree of saturation (overflow queues as explained in Section I.2) and/or the flows turning from side roads at the upstream intersection. The starting time of the upstream green period must be delayed by an amount which is a function of the length of the queue in front. In other words, the offset must be calculated to allow the front of the platoon leaving the upstream intersection to reach the end of the queue in front when it is moving (Bacon 1977; Gazis 1974; Gadd and Lay 1979; McShane and Pignataro 1980).

It is therefore recommended that, in cases where the downstream queue is likely to interfere, the offsets are calculated from

$$O = t_c - (N_d / s) \qquad (I.2)$$

where t_c is the average cruising time in seconds, N_d is the average number of vehicles in the downstream queue (before the arrival of the platoon under consideration), s is the saturation flow (veh/s) at the downstream signal, and hence (N_d/s) is the time for the downstream queue to clear the stop line. The basis of eqn (I.2) is illustrated in *Fig. I.2* for a single lane queue. It is seen that, if the queue length (N_d) is sufficiently large, eqn (I.2) can give a negative offset value (a 'reverse progression' case). For example, t_c = 15 s, N_d = 10 veh and s = 1500 veh/h = 0.417 veh/s gives O = 15 − (10/0.417) = 15 − 24 = −9 s.

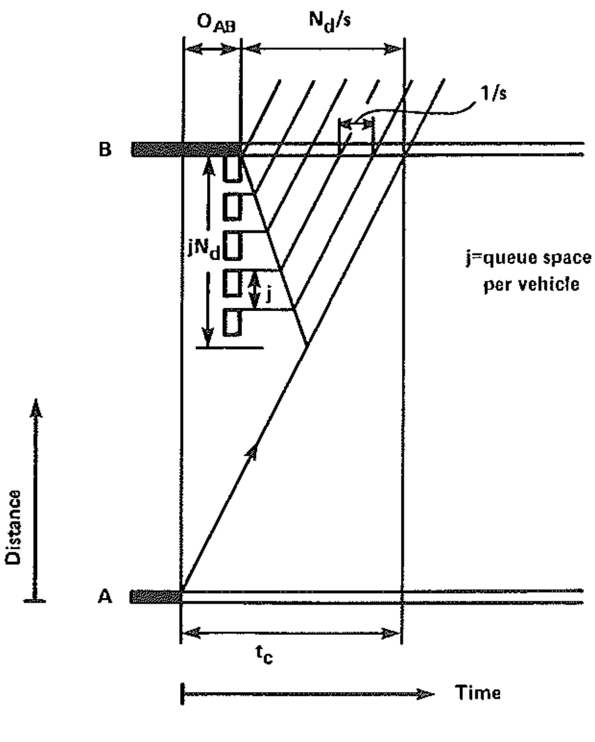

Fig. I.2 — Offset allowing for downstream queue interference (eqn (I.2))

The difficulty in using eqn (I.2) is to determine the value of N_d. This can be calculated from $N_d = N_o + N_t$ where N_o is the average overflow queue and N_t is the total number of vehicles per cycle which enter from side roads (turning flows) and queue in front of the main platoon (excluding those which arrive and depart during the green period). N_o can be calculated using eqn (I.5) given below (note that N_o is negligible for degrees of saturation below about 0.7, i.e. for moderate to low flows). N_t can be determined according to the arrival times of the front and end of side-road platoons in relation to the green and red periods at the downstream signal. Alternatively, N_d or (N_d/s) can be determined by field observation.

The desirable offsets are expressed in terms of corresponding phase change times in the signal coordination plan as an input to the master controller. Because the offset calculations are based on the starting times of effective green periods, the following offset-phase change time relationship can be used for this purpose:

$$O_{ud} = (F_d + a_d) - (F_u + a_u) \qquad (I.3)$$

where O_{ud} is the offset for travel from u to d, F is the phase change time, and a is the start lag (intergreen time, I, plus starting loss as explained in Section 2). Hence $(F + a)$ is the starting time of the effective green period, and u and d denote upstream and downstream signals.

For manual signal plan preparation purposes, the same start lag can be assumed for upstream and downstream signals, and hence, $O_{ud} = (F_d + I_d) - (F_u + I_u)$ can be used. Therefore, the phase change time at the downstream signal, F_d, to achieve a desirable offset O_{ud} is

$$F_d = F_u + I_u + O_{ud} - I_d \qquad (I.4)$$

where F_u is the known upstream phase change time and I_u, I_d are the upstream and downstream intergreen times. The difference between F_d from eqn (I.4) and the phase change time calculated assuming isolated operations (as referred to in Section I.1.1) must be added to all phase change times at the downstream intersections so that the green times remain unchanged. If, in any summation, a negative phase change time is obtained this must be corrected by adding c (cycle time). If a value larger than c is obtained, this must be corrected by subtracting c (see the example in Section I.1.3). With these corrections, all offsets are expressed in the range 0 to c.

For more than two intersections, the offset calculation and phase change time adjustment process described above must be repeated for each intersection in turn. In a closed-loop network, if an intersection has been treated once, its phase change times can be modified only by altering the offset relationships established previously. On the other hand, if each intersection is dealt with only once, the desirable offsets cannot be allocated to some platoons. This corresponds to a selective (preferential) progression plan which can be achieved by allocating the desirable offsets to the platoons with higher flows first. For an overall minimum delay, or minimum delay-and-stops strategy, it is necessary to use a computer program such as TRANSYT.

I.1.3 Example

A signal co-ordination plan is to be prepared for the intersections in Examples 1 and 2 in Appendix D. The movement and network diagrams are shown in *Fig. I.3.* Nodes 1 and 2 represent the intersections in Examples 1 and 2 of Appendix D respectively. The links have the corresponding movement numbers with the node numbers used as prefix. Pedestrian movements at Node 1 are not shown so as to keep the diagram simple. Stop line arrival flow and saturation flow is shown for each link. Flows entering links 22, 23 and 13 are shown, e.g. for Link 22, 740 veh/h enter from Link 11 and 130 veh/h enter from Link 15. The sum of upstream flows differ from the stop line arrival flow as it would be found in practice. This may be due to counting on different days or due to mid-block flow losses or additions (e.g. for Link 13, the sum of upstream flows is 990 veh/h compared with the stop line flow of 650 veh/h, the difference may be considered to be due to the traffic turning into the mid-block side road shown in *Fig. I.3a*).

The same cruising distance, l_c = 450 m, and speed, V_c = 60 km/h, and hence the same cruising time, $t_c = 3.6 \times 450/60 = 27$ s, is assumed for all platoons.

The practical cycle times calculated in Appendix D are $c_{p,1}$ = 86 s and $c_{p,2}$ = 61 s for Intersections 1 and 2 respectively. Therefore, Intersection 1 is the critical one. Let us choose c = 90 s as the common cycle time. The green times for this cycle time can be found as explained in Appendix D. The effective movement and displayed phase green times are as follows.

Intersection 1:

$g_{11} = 62$, $g_{12} = 39$, $g_{13} = 20$, $g_{14} = 58$, $g_{15} = 19$, $G_A = 38$, $G_B = 19$, $G_C = 17$.

Intersection 2:

$g_{21} = 22$, $g_{22} = 49$, $g_{23} = 19$, $g_{24} = 43$, $g_{25} = 19$, $g_{26} = 18$, $g_{27} = 11$, $G_A = 21$, $G_B = 18$, $G_C = 17$, $G_D = 10$.

The phase change times for these green times are found as follows (refer to *Tables D.1* and *D.2* for intergreen times).

Intersection 1:

$F_A = 0$
$F_B = F_A + I_A + G_A = 0 + 6 + 38 = 44$
$F_C = F_B + I_B + G_B = 44 + 5 + 19 = 68$
Check: $F_A = F_C + I_C + G_C = 68 + 5 + 17 = 90 = c$.

Intersection 2:

$F_A = 0$
$F_B = F_A + I_A + G_A = 0 + 4 + 21 = 25$
$F_C = F_B + I_B + G_B = 25 + 9 + 18 = 52$
$F_D = F_C + I_C + G_C = 52 + 7 + 17 = 76$
Check: $F_A = F_D + I_D + G_D = 76 + 4 + 10 = 90 = c$.

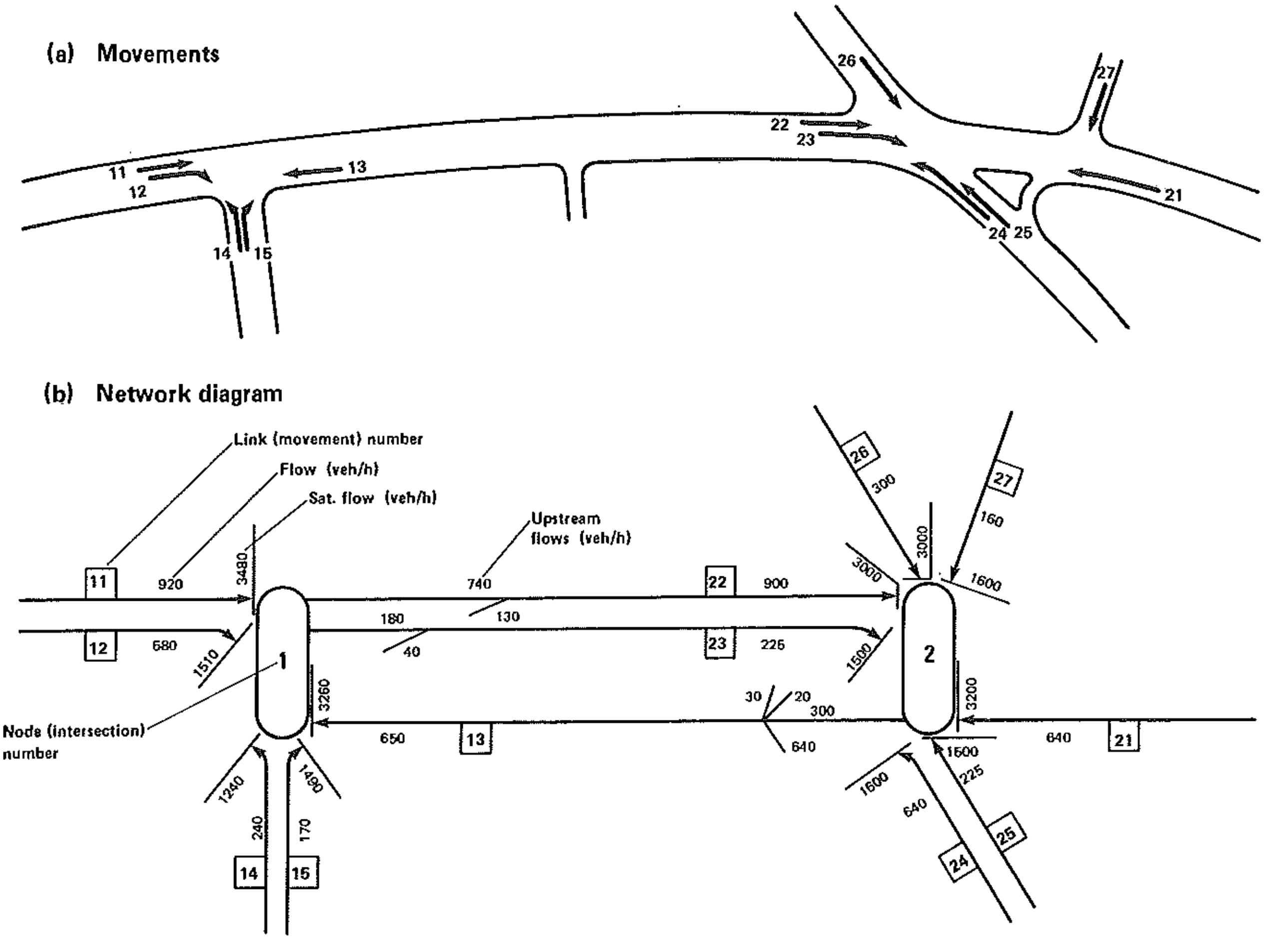

Fig. I.3 — Movement and network diagrams for co-ordinated signals (an example)

Let us now calculate the offsets and adjust the phase change times as required. The platoon which travels from Link 11 to Link 22 has the heaviest flow (740 veh/h). The platoon from Link 24 to Link 13 appears to have a high flow rate (640 veh/h) but a significant part of this flow is lost mid-block as discussed above. Let us therefore allocate the ideal offset from eqn (I.1) to the platoon from Link 11 to Link 22. Using $g_u = g_{11} = 62$, $g_d = g_{22} = 49$ and $t_c = 27$, the desirable offset is $O_{11\to12} = 27 + (62 - 49)/2 = 34$ s. Because both Link 11 and Link 22 have Phase A as their starting phases at the respective intersections, the phase change time required at the downstream signal to achieve the desired offset is calculated from eqn (I.4) as

$$F_d = F_{A,2} = F_{A,1} + I_{A,1} + O_{11\to12} - I_{A,2} = 0 + 6 + 34 - 4 = 36.$$

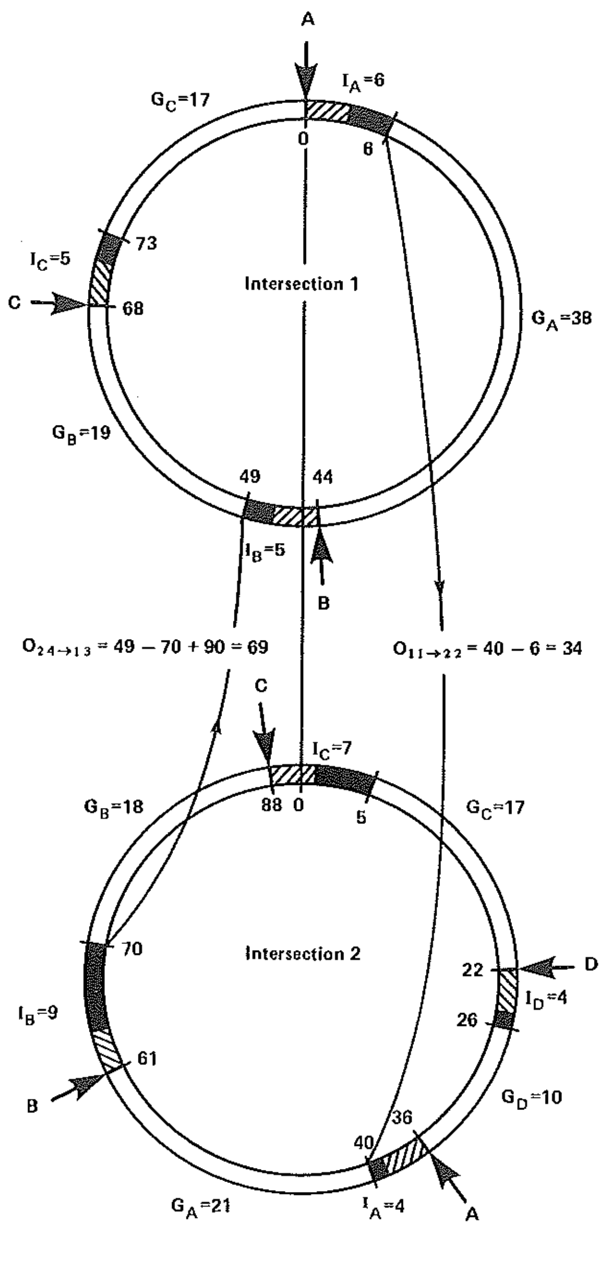

Fig. I.4 — Cycle diagrams for the example in Fig. I.3 (c = 90)

The other phase change times at Intersection 2 must be adjusted accordingly in order to keep the green times unchanged:

$F_A = 36$
$F_B = 25 + 36 = 61$
$F_C = 52 + 36 = 88$
$F_D = 76 + 36 = 112 > c$, therefore
$F_D = 112 - c = 112 - 90 = 22$
Check: $F_A = F_D + I_D + G_D = 22 + 4 + 10 = 36$.

The cycle diagrams for Intersections 1 and 2 representing the above solution are given in *Fig. I.4*. In this selective progression solution, the offset for the platoon from Link 24 to Link 13 (left-turning traffic) is $O_{24\to13} = (F + I)_{13} - (F + I)_{24} = (F + I)_{B,1} - (F + I)_{B,2} = (44 + 5) - (61 + 9) = -21$ s (or $-21 + 90 = 69$ s). The offset for the platoon from Link 21 to Link 13 (through traffic) is $O_{21\to13} = (F + I)_{B,1} - (F + I)_{A,2} = (44 + 5) - (36 + 4) = 9$ s.

An overall optimum solution obtained for this example using the 6N version of TRANSYT program (Akcelik 1979*b*) with a stop penalty of $K = 20$ (see Section 6.7) gave the same cycle time and green times as calculated manually, but offsets differed significantly. The phase change times describing this solution are:

Intersection 1: $F_A = 0, F_B = 44, F_C = 68$
Intersection 2: $F_A = 18, F_B = 44, F_C = 70, F_D = 4$

The corresponding offsets are $O_{11\to22} = 16$, $O_{24\to13} = 86$ and $O_{21\to13} = 27$. The average delays and number of stops on Links 13, 22 and 23 as predicted by TRANSYT program (version 6N) for manually calculated timings and TRANSYT optimised timings are given in *Table I.1*. It was found that TRANSYT timings improved the operating efficiency (delay and stops combined) by 7 per cent for Links 13, 22 and 23, or by 2 per cent for the whole network (i.e. including side roads whose conditions were the same in both solutions). *Table I.1* shows that improvements in the number of stops were more significant (13 per cent). The example indicates the value of using a program such as TRANSYT for preparing co-ordinated signal plans, in particular for determining offsets, even in the case of a simple network.

TABLE I.1

TRANSYT RESULTS FOR THE EXAMPLE IN FIG. I.3

Link	*Manual timings*		*TRANSYT timings*	
	Ave delay (s)	*No. of stops (veh/h)*	*Ave delay (s)*	*No. of stops (veh/h)*
13	45	624	45	629
22	11	590	10	406
23	41	208	39	207

I.2 OPERATING CHARACTERISTICS

The prediction of delay, number of stops and queue length in the case of co-ordinated signals is complex because of flow interactions between intersections. Arrival and departure patterns depend on the way vehicles are formed into regular platoons at upstream signals, the way the platoons disperse as they travel along the road, and the time they arrive at a downstream signal. The signal offset is therefore an important control parameter.

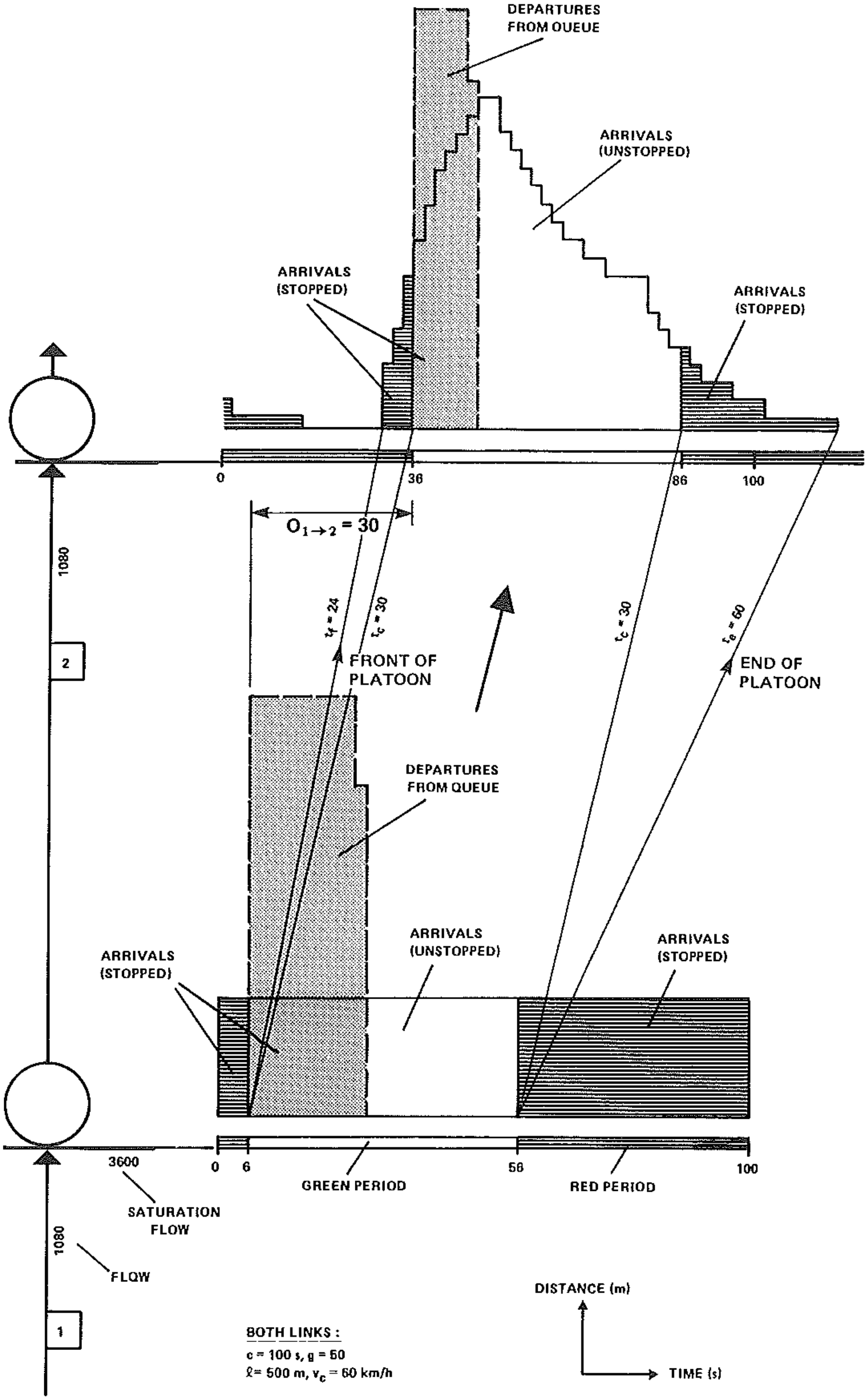

Fig. I.5 — Vehicle platoons and signal offsets

A simple case of a pair of links (movements) with the same arrival flow, saturation flow and signal timing parameters (hence the same degree of saturation) is shown in *Fig. I.5.* The arrival and departure patterns (i.e. the number of vehicle arrivals in, say, 2 s intervals) shown for Link 1 assume regular arrivals and departures, and hence correspond to the uniform components (first terms) of the delay, stop rate and queue length formulae in Section 6 (eqns (6.3), (6.5) and 6.7)). In *Fig. I.5*, it is assumed for the sake of simplicity that the only platoon entering Link 2 is the platoon which is discharged from Link 1. The arrival pattern at the stop line of Link 2 is determined by the departure pattern of Link 1 modified according to a platoon dispersion process which allows for different vehicle speeds in the platoon. The pattern shown in *Fig. I.5* has been obtained using the TRANSYT computer program. The platoon arrival pattern of Link 2 shown in *Fig. I.5* is of a cyclic character as it is generated from the regular arrival/departure pattern of Link 1. Similarly, all platoon arrival and departure patterns can be predicted and the corresponding operating characteristics can be calculated for an average signal cycle. As in the case of isolated signals, these can be called the 'uniform' components of delay, stop rate and queue length. The calculations can be carried out using a traffic model such as that used in TRANSYT program or using simplified analytical expressions (e.g. see Allsop 1969; Buckley, Leong and Ong 1969; Dick 1965; Gartner, Little and Gabbay 1975; Newell 1969*a*).

In order to allow for the effects of temporary or persistent oversaturation, the 'overflow' components of delay, stop rate and queue length must be added to the uniform components of these performance measures as in the formulae given in Section 6 for isolated signals. Based on the evidence given in Robertson (1969, Appendix 2) that the effect of random variations in flow levels from cycle to cycle is smaller at co-ordinated signals than at isolated signals, probably because of the structured nature of arrival and departure patterns, a simple approximation can be made by assuming that the average overflow queue at undersaturated co-ordinated signals is half the value of the average overflow queue at undersaturated isolated signals. The following expression, which is based on this assumption, can be used to calculate the average overflow queue for both undersaturated and oversaturated co-ordinated signal cases:

$$N_o = \begin{cases} \dfrac{QT_f}{4}\left(z + \sqrt{z^2 + \dfrac{6(x - x_o)}{QT_f}}\right) & \text{for } x > x_o \\ \text{zero} & \text{otherwise} \end{cases} \qquad (I.5)$$

The variables in this expression are the same as in eqn (6.1) given in Section 6, the only difference being in constant 6 under the square root instead of 12 in eqn (6.1).

Using the average overflow queue, N_o, given by eqn (I.5), the overflow terms of delay, stop rate and queue length at co-ordinated signals can be calculated as ($N_o x$), (N_o/qc) and (N_o), respectively, the same as in eqns (6.3), (6.5) and (6.7) of Section 6. The derivation of these expressions is explained in Akcelik (1980*b*). TRANSYT version 8 makes similar corrections for 'random plus oversaturation' effects but there are differences in the expressions used (especially because the calculations are based on a formula which corresponds to that used for isolated signals in previous versions of TRANSYT).

In *Fig. I.6* the delay and stop rate as a function of the signal offset are shown for Link 2 of *Fig. I.5.* These are calculated using the 6N version of TRANSYT program. In this example, the overflow components are negligible because of the low degree of saturation and high capacity per cycle ($x = 0.6$, $sg = 50$ veh/cycle).

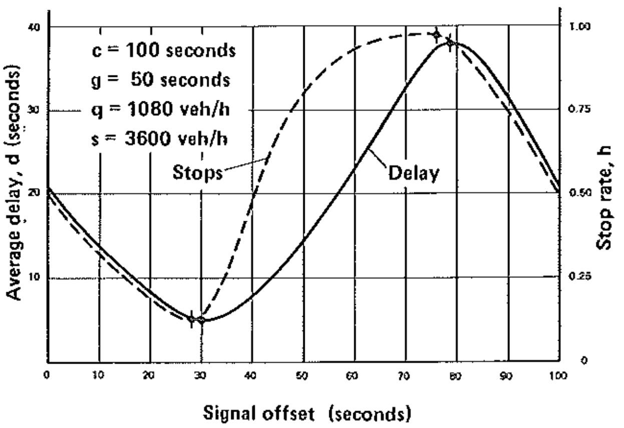

Fig. I.6 — An example of average delay and stop rate as a function of the signal offset

APPENDIX J

BLANK FORMS

Blank forms to aid the calculations for:

(a) critical movement identification,

(b) saturation flow estimation, and

(c) saturation flow and lost time measurements are given in the following pages.

CRITICAL MOVEMENT SEARCH TABLE

(a) DATA

Movement	Starting Phase	Termin. Phase	Intergreen Time (I)	Min.Disp. Green (G_m)	Arrival Flow (q)	Satur. Flow (s)	Lost Time (ℓ)	Min.Eff. Green (g_m)	Prac. Deg.Sat. (x_p)
1									
2									
3									
4									
5									
6									
7									
8									
9									
10									
11									
12									

(b) CALCULATIONS

Movement	y = q/s	u = y/x_p	100 u+ℓ	t_m = G_m + I	t	Check for c =			
						uc + ℓ	t′	g	x = (c/g) y
1									
2									
3									
4									
5									
6									
7									
8									
9									
10									
11									
12									

* Critical movements

SATURATION FLOW ESTIMATION

LANE WIDTH AND GRADIENT ADJUSTMENT

Movement Number =

Lane Number	Environment and Lane Type	Sat. Flow (Table 5.1) tcu/h	Lane Width m	Factor, f_w (eqn 5.2)	Sat. Flow tcu/h
					TOTAL =

Gradient, G_r = per cent

Factor, f_g (eqn 5.3) =

s_{tcu} =

TRAFFIC COMPOSITION ADJUSTMENT

Movement Number =

	Left		Through		Right		Total
	CAR	HV	CAR	HV	CAR	HV	
Flow count, q_i (veh)							$= q$
Equivalent, e_i (tcu/veh)			1	2			$f_c =$
Weighted flow, $e_i q_i$ (tcu)							$= \Sigma e_i q_i$

Through car equivalents (e_i) from Table 5.2
(eqn 5.5 for opposed turn equivalent, e_o)

$f_c = (\Sigma e_i q_i)/q =$

$s_{veh} = s_{tcu}/f_c =$

SATURATION FLOW AND LOST TIME MEASUREMENT

(Site information)

CYCLE NUMBER	DEPARTURES FROM QUEUE (vehs)			SATURATION TIME * (s)	GREEN TIME (s)
	First Interval	Middle Interval	Last Interval		
	1	2	3	4	5
1 2 3 4 5					
6 7 8 9 10					
11 12 13 14 15					
16 17 18 19 20					
21 22 23 24 25					
26 27 28 29 30					
TOTAL	X_1 =	X_2 =	X_3 =	X_4 =	X_5 =
SAMPLES	n_1 =	n_2 =	n_3 =	n_4 =	n_5 =

* Not to include the amber time, i.e. the maximum value to be the green time in col. 5.

INTERGREEN TIME (s) =

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