ROAD RESEARCH LABORATORY

Ministry of Transport

Road Research Technical Paper No. 56

Traffic Signals

BY

F. V. WEBSTER, B.Sc., Ph.D.

AND

B. M. COBBE, B.Sc., A.M.I.E.E.

EESC-USP
Biblioteca
CAMPUS DE S. CARLOS

LONDON

HER MAJESTY'S STATIONERY OFFICE

1966

Class. 388.312
Cutt. W379t
Tombo 674/75
st 231751

Published by
HER MAJESTY'S STATIONERY OFFICE
To be purchased from
49 High Holborn, London W.C.1
423 Oxford Street, London W.1
13a Castle Street, Edinburgh 2
109 St. Mary Street, Cardiff
Brazennose Street, Manchester 2
50 Fairfax Street, Bristol 1
35 Smallbrook, Ringway, Birmingham 5
80 Chichester Street, Belfast 1
or through any bookseller

Printed in England for Her Majesty's Stationery Office by McCorquodale, London

FOREWORD

TRAFFIC SIGNAL CONTROL is by far the most common type of control at heavily trafficked intersections in urban areas, and in the central areas of large towns delays at traffic signals can account for as much as a third of the total journey time. A great deal of research has been carried out to help the engineer in designing suitable traffic-signal schemes and in setting signals to minimize delays; much practical experience has also been gained by engineers working in the field over the last thirty years or so. Some of the results of research and some information based on experience have been published in a variety of journals, but none of the articles is really comprehensive.

This Technical Paper attempts to provide a more comprehensive treatment of the subject than has been possible previously; it contains much that has already been published, but in addition, records the results of further research carried out especially to fill in some of the gaps in the knowledge.

This Paper has been prepared jointly by an engineer from the Highways Division and a scientist from the Road Research Laboratory of the Ministry of Transport. The Paper is thus based both on research and practical experience of signal control.

D. J. LYONS,
Director of Road Research

ROAD RESEARCH LABORATORY,
April, 1966

CONTENTS

Page

Page

© Crown copyright 1966

Extracts from the text may be reproduced provided the source is acknowledged

Traffic Signals

SUMMARY

A DESCRIPTION OF fixed-time and vehicle-actuated traffic signals, including details of the various facilities available with present British vehicle-actuated equipment, is given in this Paper. Some information on pedestrian signals is also included. Methods of co-ordinating signals at two or more intersections are discussed.

Information is given on the various stages in the design of a signal scheme, e.g. warrants for signals; layout and siting of signals, islands and detectors; use of carriageway markings; widths of approaches and the use of special right-turning lanes and left filters; phasing and clearance periods.

The results of research into the capacity of intersections controlled by traffic signals are described, and it is shown how capacity is affected by the width of the carriageway, the gradient, the type of site, the presence of right-turning vehicles, heavy commercial vehicles and parked vehicles.

The results of research using an electronic computer to simulate traffic at traffic signals, previously published in *Road Research Technical Paper* No. 39, are repeated in this Paper for completeness. A formula for the average delay at fixed-time signals is given, and it is shown how this formula can be used to predict the delay at vehicle-actuated signals also. The effect on delay of a vehicle parked on the approach to a signal-controlled intersection is also investigated. From the delay formula expressions have been deduced for the cycle time and green times (fixed-time signals) which give the least overall delay to vehicles using the intersection, and the expressions can be used as an aid in setting vehicle-actuated signals. The procedure to be adopted in deducing optimum signal timings, when the phasing arrangement is complex or when saturation flow is not constant, is outlined. A comparison of vehicle-actuated and fixed-time working of a signal-controlled intersection is made and the advantages of vehicle-actuation are shown.

The use of signals at congested roundabouts to avoid the 'locking' condition which frequently occurs under heavy traffic is discussed. The problems associated with traffic signals on high-speed roads are mentioned together with a brief description of a new type of control equipment to overcome these difficulties. A formula is given for signal settings at road-works and bridges where shuttle working may be operated.

The Paper concludes with suggestions for the application of the information given, and a number of worked examples are appended to illustrate the methods described.

INTRODUCTION

Although an appreciable amount of published work is now available on traffic signals it deals mainly with isolated aspects of the subject. It was felt by the authors that something more comprehensive was required by those whose job it was to deal with the technical and design problems of signal-controlled intersections. This Paper was therefore written essentially as a practical guide, dealing, as comprehensively as the space allows, with the subject from the design point of view. The Paper does not cover the electrical or hardware aspects.

The techniques used in installing traffic signals, in arranging for the most efficient phasing and in co-ordinating groups of signals, etc., have evolved mainly from the experience of the engineers engaged on this type of work. It is difficult for newcomers to the subject to learn these techniques and an attempt is made in this Paper to remedy this situation.

As the volume of road traffic increases, more and more traffic signals are installed at intersections. Since it has been estimated that queueing at traffic signals accounts for about 100 million vehicle-hours each year it is clearly of the utmost importance to set the timings correctly so as to minimize this delay. This Paper repeats the rules given in an earlier Technical Paper(1)* for setting fixed-time signals and extends them to apply to vehicle-actuated signals. The previous Technical Paper on this subject was of a theoretical nature, and much difficult interpretation of the rules was required in order to deal with a number of not-uncommon situations. Consequently, guidance on the practical application of the results of research has been included here.

Capacity of signal-controlled intersections is a very important subject and is of considerable concern, not only at the design stage, but also when making economic assessments of different types of intersections and of the value of improvements at signal-controlled junctions. The latest results of the Road Research Laboratory's work are given in the Paper.

The Ministry of Transport has recently produced a Memorandum, 'Urban Traffic Engineering Techniques',(2) and is currently producing a Manual on 'Roads in Urban Areas'.(3) These papers will give additional practical information on traffic signals, although there will be a certain amount of overlap with this Paper.

A glossary of terms and symbols used in this Paper, together with some of the equations, is given in Appendix 1. It should be noted that the same units of time and distance must be used throughout any particular formula, unless otherwise specified.

HISTORICAL

The first traffic signal was installed in Westminster in 1868 and was of the semaphore-arm type with red and green gas lamps for night use. Unfortunately, however, an explosion occurred and no further experiments of this nature were tried for half a century. In 1918 the first manually operated three-colour light signals were installed in New York, and in 1925 manually operated coloured light signals were used by the police in Piccadilly, London. The following year the first automatically operated traffic signals in Great Britain were installed at Wolverhampton.

The natural development of traffic control methods led from manually operated to automatic fixed-time signals, where predetermined 'stop' and 'go' periods were alternately timed off. These signals helped to ease traffic conditions but were not efficient at junctions where the traffic volume varied considerably;

*As far as the practising engineer is concerned the present Paper supersedes the earlier one, though the latter contains information on the methods and analyses used in obtaining the results

'programme' controllers were therefore introduced to alter the length of the 'stop' and 'go' periods in steps throughout the day to fit in with a prearranged plan. Some streets having a large number of cross-roads were later equipped with fixed-time signals at each intersection along the route, and the signals were linked together in such a way that the green periods appeared to 'progress' along the road, allowing groups of vehicles more or less uninterrupted travel at a predetermined speed through the series of intersections. This was an improvement where a series of intersections could be dealt with together, but at isolated intersections traffic was still being delayed unnecessarily; this was due to the inflexibility of fixed-time signals and also to lack of knowledge of the settings which would give minimum overall delay.

At the beginning of the 1930s a first attempt at vehicular control of signals was made in the United States of America by placing microphones at the side of the road and requesting drivers to sound their horns. There were many objections to this scheme, and a method using electrical contacts placed in the paths of vehicles was subsequently tried. This method has survived in principle up to the present day although pneumatic tubes are now generally used in Great Britain. Air displaced in the tubes by a vehicle passing over them operates electrical contacts housed in a sealed compartment at the side of the road. Other types of detectors, e.g. inductive detectors, radar, magnetic and ultrasonic detectors, have been used abroad and some of these are being considered or are undergoing trials in this country.

The first vehicle-actuated signals in Great Britain were installed in 1932 at the junction of Gracechurch Street and Cornhill in the City of London. Unfortunately history repeated itself, and when the signals were switched into service an explosion occurred owing to seepage of gas into the controller cabinet. Despite this unhappy incident vehicle-actuated signals soon became established and three years later the first linked systems, consisting entirely of vehicle-actuated controllers, were installed in London and Glasgow.

There are now about 4000 signal installations in Great Britain, about 1000 of these being in the Greater London area.

PRESENT-DAY TRAFFIC SIGNALS

Signal aspects

The signal sequence of traffic signals in Great Britain is red, red/amber shown together, green and amber. The amber period is standardized by the Ministry of Transport at 3 seconds and the red/amber at 2 seconds. The 2-second red/amber is at present provided only by the latest types of signal controllers, the older types giving a 3-second red/amber.

Intergreen period. The time from the end of the green period of the phase* losing right-of-way to the beginning of the green period of the phase gaining right-of-way is called the 'intergreen' period. With the latest type of signal controller the minimum intergreen period is normally 4 seconds, the amber

*Phase is the sequence of conditions applied to one or more streams of traffic which, during the cycle, receive simultaneous identical signal indications period of one phase and the red/amber of the succeeding phase overlapping by 1 second. The intergreen period can be extended to 5 seconds when the amber and red/amber periods appear consecutively (sometimes called sequent ambers). Further extension of the intergreen period produces a period of 'all-red' between the amber and the red/amber. With the older controllers the minimum intergreen period is 3 seconds, the amber and red/amber being shown concurrently. Sequent ambers and all-red periods can be given, but the older controllers do not give partially overlapping ambers. The various intergreen periods are illustrated in Fig. 1.

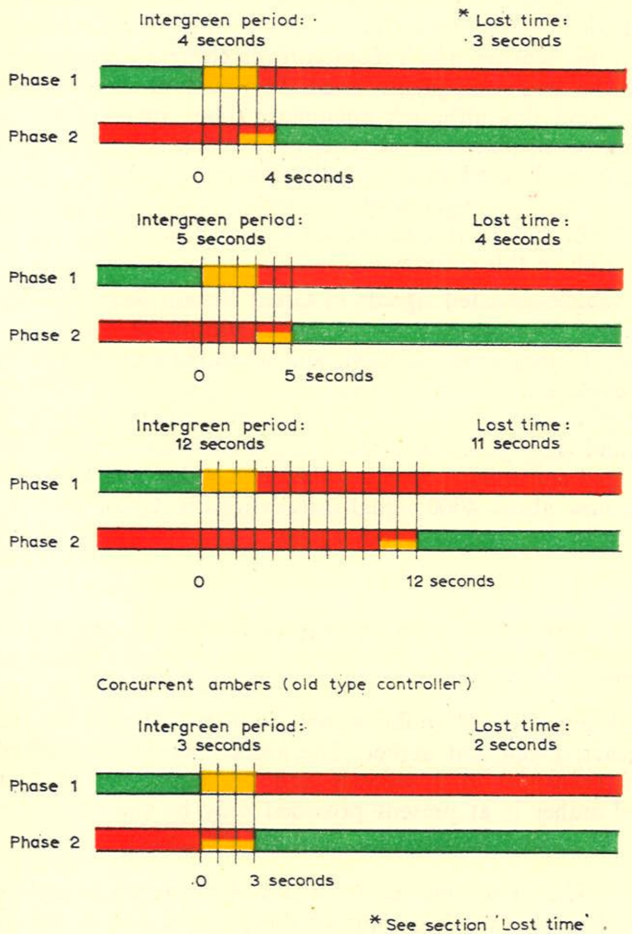


FIG. 1. Examples of intergreen periods at a 2-phase traffic signal

Types of signals available

There are essentially two types of signals in general use: fixed-time and vehicle-actuated. Although most fixed-time signals have been replaced by vehicle-actuated signals in Great Britain, in some countries, e.g. the U.S.A., fixed-time signals are far more numerous than the vehicle-actuated type. An intermediate type, semi-vehicle-actuated signals, with detectors on the side roads only, is discussed later.

Fixed-time signals

With fixed-time signals the green periods, and hence the cycle times, are predetermined and of fixed duration. The controllers are simple and relatively inexpensive but they are necessarily inflexible and require careful setting. They are most successfully used in linked systems. They can be equipped with time switches to alter the settings at certain periods of the day, to cover different traffic conditions.

Vehicle-actuated signals

With vehicle-actuated signals the green periods are related to the traffic demands, using detectors which are normally installed on all approaches. In the absence of demands the signals will rest indefinitely on the phase which was last served. With the latest British equipment the controller consists of several low-voltage electronic timers. The following facilities are available:

Minimum running period. This is the shortest period of right-of-way which is given to any phase and is long enough for vehicles waiting between the detector and the stop line to get into motion and clear the stop line. On the latest controllers the minimum running period is not fixed but varies between 7 and 13 seconds* according to the number of vehicles waiting. The minimum value and the rate of build-up can be varied to suit different site conditions. Under light flow conditions the minimum running period is automatically reduced; this permits a quicker change of right-of-way, thus reducing overall delay to vehicles, particularly when three or four phases are in use.

Vehicle-extension period. The green period may be extended beyond the minimum running period by vehicles passing over the detectors. As each vehicle crosses the detector the green period is extended by an amount called the vehicle-extension period. In many of the older controllers this extension was of a fixed duration, but in the latest controllers the extension is related to the speed of the vehicle as measured at the detectors. It is automatically varied to enable each vehicle to travel a pre-set distance (between 75 and 160 ft) from the detector to a point generally 10 or 20 ft beyond the stop line. The system is reasonably accurate for speeds between 15 and 30 mile/h but outside these limits there is greater variation in the extension distances; at the higher speeds there are inaccuracies in speed measurement, and at the lower speeds it is necessary to impose a cut-off in the extension time in order to avoid overlong extension periods. Extensions are individual and not cumulative and the associated timer is only reset to a new value if the next extension exceeds the unexpired time of the previous extension. When the interval between

*8 and 15 seconds on earlier models of the latest type of controller vehicles crossing the detector becomes greater than the vehicle-extension period, right-of-way is transferred to the next phase if so required (this is called a 'gap' change of right-of-way).

Pre-set maximum period. To prevent vehicles on a halted phase from waiting indefinitely because of a continuous stream of traffic on the running phase a maximum period is timed off, after which the signals change right-of-way irrespective of the state of the vehicle-extension period. The maximum period starts at the beginning of the green period if vehicles are waiting on any halted phase, or at the time the first vehicle passes over the detector on any halted phase, whichever is the later. Thus the period may be regarded as a maximum waiting period rather than a maximum green period. It can be set within the range 8 to 68 seconds.* When a change of right-of-way occurs owing to the expiry of the maximum period, provision is made for the right-of-way to return to the original road as soon as traffic conditions on the other roads permit (this is called maximum reversion), and the minimum running period is then normally set at 7 seconds but can be increased by movement of further vehicles over the detector.

If the traffic is fairly heavy on all phases the green periods may run successively to maximum, giving in effect fixed-time operation. Many signals in large cities operate in this manner during peak periods.

Variable maximum period. This is a facility which allows the maximum to be extended automatically beyond its pre-set value if the average rate of traffic flow at the end of the maximum period (on one or more of the running approaches as desired) exceeds a predetermined critical value. Right-of-way can then continue (provided there is no gap change) for so long as the average rate of flow exceeds the instantaneous value of a limiting rate of flow against which it is continuously compared—the value of the limiting rate of flow being the critical value at the beginning of the extended period, and increasing steadily thereafter.

Variable intergreen period. Extra clearance may be required to protect the passage of clearing traffic through an intersection before the next traffic stream is released. If a phase change occurs at the end of the minimum green or through a maximum termination or immediately at the end of a vehicle-extension the whole of the intergreen period is required. This is illustrated in diagram A of Fig. 2. It can be seen in diagram B that if the phase change takes place 2 seconds after the end of the vehicle-extension period the 'all-red' may be reduced by 2 seconds; this allows the same separation in time between the two moving streams. Similarly if 4 seconds elapse (diagram C) before the phase change then the extra intergreen period may be eliminated. This facility may be used in conjunction with additional detectors within the controlled area to extend the intergreen period, e.g. to control the all-red periods for shuttle working on bridges. Fixed intergreen periods may also be employed in suitable circumstances.

Sequence of phases. Normally the phases are served in cyclic order and if there is no traffic requiring a particular phase it is omitted. Provision is made where desired for reversion to a selected phase in the absence of demands from other approaches.

*Traffic signals on certain holiday routes can have the appropriate maximum period increased by a further 2 minutes by operation of a switch

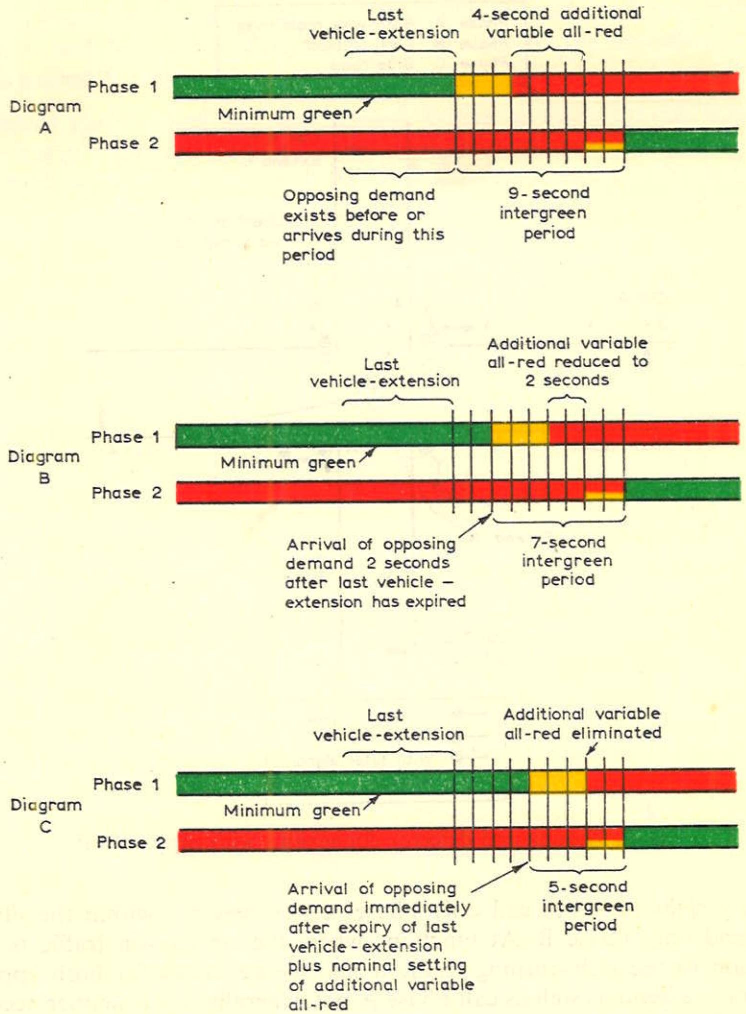


FIG 2. Variable intergreen period based on a 5-second minimum intergreen and 4-second additional variable all-red

Police facilities. A switch, accessible to the police, enables selection between normal vehicle-actuated working, fixed-time working to maximum settings and manual control by pushbutton.

Early cut-off. To facilitate a heavy right-turning movement from one approach, the green time of the opposing arm can be cut off a few seconds before the arm having the right-turn movement. The early cut-off may be either fixed-time or may be extended by a detector within the junction sited in the path of right-turning vehicles (see Fig. 3). If the 'both-way' running period for the main road is connected to Phase A of the controller, the early cut-off to Phase B and the side road to Phase C, then the detector in the opposing arm will normally call and extend Phase A; the detector in the approach with

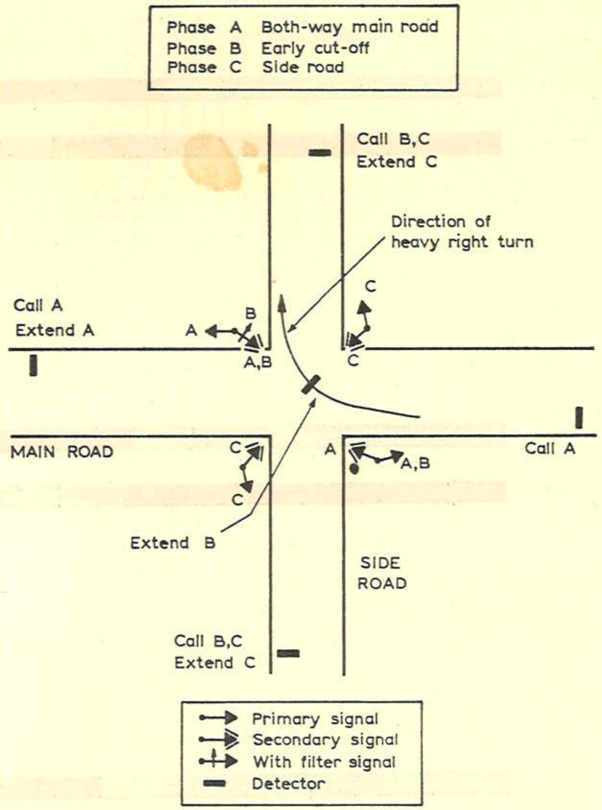


FIG. 3. Phasing and detector connexions with early cut-off

the heavy right turn will call only Phase A; the detector within the junction will extend only Phase B. At junctions where the straight-on traffic is heavy in relation to the right-turning traffic it may be necessary for both approach detectors to extend as well as call Phase A but generally this is neither necessary nor desirable. The side-road detectors call Phase B as well as calling and extending Phase C. (With this detector arrangement the early cut-off is brought in on every change to the side road and if this is not desired—e.g. under light traffic conditions—the early cut-off phase may be switched in and out under time-switch control. An alternative arrangement is for the detector within the junction to call Phase B, the call being stored—if there is no outstanding demand for Phase C—until a demand for Phase C is received. This arrangement will not be satisfactory unless the detector can be so sited that right-turning vehicles are bound to cross over it before stopping to make the turn; in addition, the detector must not be traversed by straight-on vehicles.)

Late release. An alternative way of dealing with right-turning traffic is to delay the start of the opposing traffic by a few seconds. It is difficult to arrange satisfactorily for the duration of a late release to be vehicle-controlled and hence it is usually a fixed-time period. Although less flexible than an early cut-off, a late release is sometimes preferable (see 'Phasing'). If the late-release period is connected to Phase A and the 'both-way' running period to Phase B of the controller, then the detector on the approach with the heavy right turn will call A and B and extend B, whilst the detector on the opposing arm will call and extend B only.

Combined early cut-off and late release. Where an early cut-off is used to serve a heavy right-turn demand it is important that the right-turning traffic should wait till the end of the 'both-way' running period and not try to establish a right-turning movement at the beginning of the period. This can be achieved by starting the opposing stream 3 to 4 seconds before the stream with the right turn (by a late release of the latter). A late release is therefore often provided on one approach in conjunction with an early cut-off on the opposing one. The phasing and detector connexions are shown in Fig. 4. (This arrangement is generally unnecessary on dual-carriageway roads.)

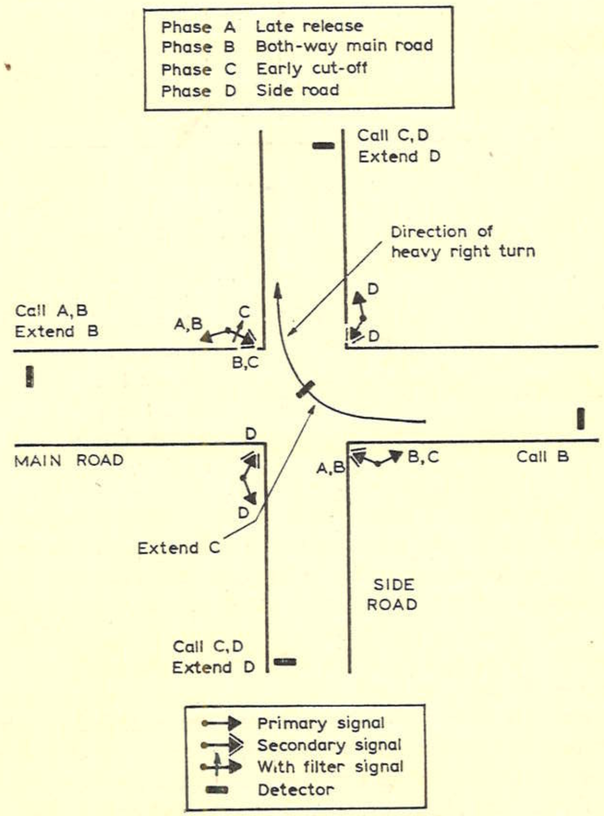


FIG. 4. Phasing and detector connexions with combined early cut-off and late release

Density control. This feature is available on certain older controllers but not on the latest type, for which the variable maximum facility has been developed. With density control the maximum period is automatically varied between a high and a low limit depending upon the traffic flow (see Fig. 5). After each vehicle passes over the detector the timing condenser in the maximum period circuit is charged at a low rate for a certain pre-set interval of time. At the

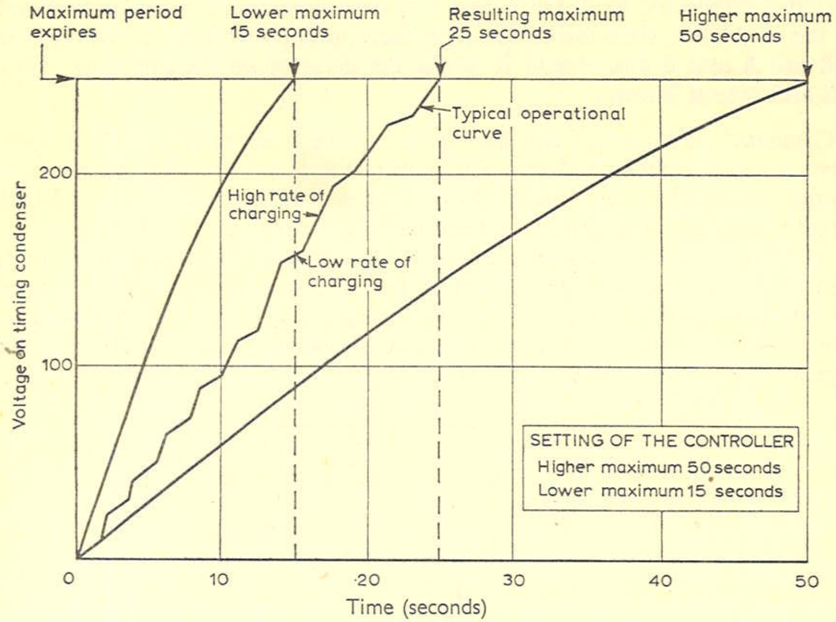


FIG. 5. Example of maximum timing using density control

end of this interval the charging reverts to a high rate. Thus very heavy traffic would prolong the low-rate charging interval so that the maximum period would be equal (or nearly so) to the high maximum setting. With very light traffic the maximum period would be little more than the low setting. Density control was designed primarily for junctions where the phases frequently run to maximum and where there is heavy traffic on one phase at one time of the day and on a different phase at another time of day. The maximum periods automatically adjust themselves to cater for such fluctuations in flow on each phase.

The variable maximum facility has the advantage over density control that the extension of the maximum is dependent on a heavy traffic flow at the end of an normal maximum period and not on traffic flow throughout the green period.

Semi-vehicle-actuated signals

With semi-vehicle-actuated signals detectors are installed on the side roads only and the right-of-way normally rests with the main road, being transferred immediately (or at the end of a pre-set period) to the side road when a vehicle passes over the side-road detector. The green period on the side road can be extended in the normal way by successive demands up to a pre-set maximum. After right-of-way has been returned to the main road, it cannot be taken away from the main road until the pre-set period has expired. Semi-vehicle-actuated signals at times of light traffic are believed to have a higher accident rate than fully vehicle-actuated ones; this is probably because of the immediate signal response which an approaching side-road vehicle gets on crossing the detector, thus causing the main-road stream to be interrupted quite arbitrarily. A dangerous situation could arise if a main-road vehicle could not stop in time

(owing to this arbitrary interruption of the green time) and a side-road vehicle was allowed to enter the intersection at speed. This danger can be lessened by introducing a 'delayed-change' facility so that the red/amber is delayed by 1 to 2 seconds after the vehicle crosses the detector. For this reason, and also because of delay considerations, only signals with full vehicle-actuation are installed in Great Britain.

Co-ordinated control systems

When two or more junctions are in close proximity on a main traffic route some form of linking is necessary to reduce delays and prevent continual stopping. The purpose of a linked system is to pass the maximum amount of traffic without enforced halts, while allowing for the claims of cross-street traffic. Sometimes minimum overall delay to all streams, including the side-road streams, is sought. Alternatively, or additionally, linking may be employed to prevent the queue of vehicles at one intersection from extending back and interfering with another. The several basic forms of linking are described below. Of these the simultaneous, alternate and flexible progressive systems require a master controller and may be used where several installations are linked together. Linking without a master controller, which is generally employed where only two installations are linked, but may be employed with more, is described under 'Tailor-made systems'.

Simultaneous system (*synchronized system*). All the signals along the controlled section display the same aspect to the same traffic stream at the same time. This type of system encourages speeding as some drivers try to pass as many intersections as possible before the signals change. Variations on this basic system can be used with both fixed-time and vehicle-actuated signals to give green times at each intersection corresponding to traffic needs. A common cycle is used throughout and a master controller keeps the local controllers in step.

Alternate system (*limited progressive system*). With this system consecutive signal installations along a given road show contrary indications. The aim is for vehicles to travel one block in half the cycle time. Drivers find that if they exceed the design speed of the system they are stopped at each signal. This system is not very suitable for streets where the distances between intersections vary appreciably. Variations on this basic scheme can be made with both fixed-time and vehicle-actuated signals under the control of a master controller. The alternate system can be extended to allow adjacent groups of synchronized signals to show opposite indications along the main route.

Flexible progressive system. The cycle time for each intersection in the system is common but the 'go' periods are staggered in relation to each other according to the desired road speed. This is intended to give a 'progression' of green periods along the road in both directions. To arrive at the best arrangement of the 'go' periods it is helpful to construct a time-and-distance diagram as shown in Fig. 6. The amount of the cross-street traffic must be taken into account in deciding the permissible length of the green periods on the main roads and, in general, a compromise is effected between the various requirements. With unevenly spaced junctions a compromise has also to be made between the two directions and the resulting system often is basically a mixture

designal

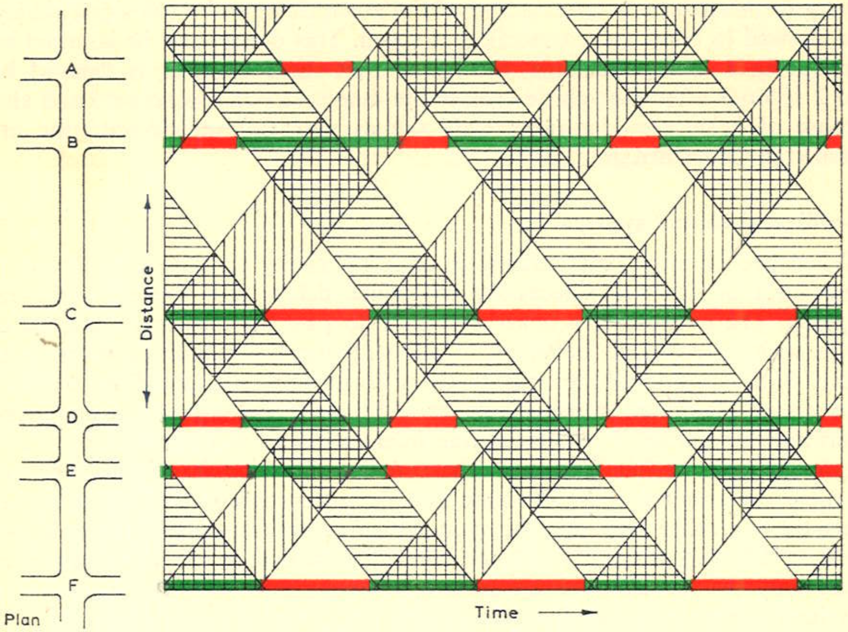


FIG. 6. Time-and-distance diagram for a hypothetical linked system

of 'alternate' and 'synchronized' working. This system can be used to give a 'preferential' movement, e.g. in the morning peak to favour the inbound flow at the expense of the fewer vehicles travelling in the opposite direction, and vice versa in the evening peak (see 'Delays and optimum settings of linked signals' for more details).

The flexible progressive system requires a master controller to keep the 'local' controllers at each intersection in step. Either fixed-time or vehicle-actuated signals can be used. If vehicle-actuated signals are used the local controllers at each intersection in the system operate according to the over-riding progressive plan whilst there are continuous demands from all detectors. If the flow falls and there is no longer a continuous demand on the detectors at a particular installation in the system it would be free to operate as an isolated vehicle-actuated installation, changing right-of-way when necessary to suit the traffic arriving at the intersection. However, there is the proviso that any change of right-of-way that takes place should not interfere with traffic passing through the system in accordance with the progressive plan. This means that there are certain periods in the cycle when a change of right-of-way cannot take place because there would be insufficient time in which to regain right-of-way for the road which, according to the progressive plan, is entitled to it. There are also periods when a forced change of right-of-way can be made to facilitate the working of the progressive plan.

One of the available flexible progressive systems incorporates three time pulses, designated 'Prevent', 'Privilege' and 'V.A.', with each traffic phase. The 'Prevent' pulse introduces a period during which the only change which

can be made is to the phase which will next require right-of-way. Some seconds later the 'Privilege' pulse permits a forced change to the required phase, if this is not already running, and then holds the controller on that phase. After the 'Privilege' period a 'V.A.' pulse either maintains the current phase or permits the controller to change to another phase according to traffic needs, i.e. to operate as an isolated vehicle-actuated controller until the 'Prevent' pulse for the next phase is received.

An alternative system uses two time pulses for each phase, designated 'Check' and 'Commence'. The 'Check' pulse is similar in effect to the 'Prevent' pulse of the other system. The 'Commence' pulse introduces a bias period during which signals are biased to the selected phase. Demands from other phases may be served if a gap occurs on the selected phase, but fresh demands on the selected phase will, during this period, bring about an immediate change to the selected phase (subject to expiry of the minimum green period on the phase losing right-of-way).

When traffic is very light the progressive system may be considerably less flexible than an unlinked system of vehicle-actuated signals because of its mode of operation. This limitation can be partly overcome by a time switch arranged to link and unlink all the signals in a system. A time switch can also change the progressive 'plan' in a predetermined manner. Alternatively, traffic integrators (devices for counting traffic) can be used to do this in accordance with the traffic flows at one or more locations. The traffic integrator can also vary the common cycle time. Since traffic integrators measure flow over a period, they cannot vary the common cycle time from cycle to cycle according to traffic needs. Where wide variations are required from cycle to cycle, either in cycle time or in the proportions of time given to each phase, the tailor-made systems described below are generally to be preferred.

Tailor-made systems. At the present time many schemes of linked signals in Great Britain are designed specifically, or 'tailor-made', for particular locations. These systems do not generally have a master controller to govern the operation of several local controllers as this often leads to less efficient control at particular key intersections in the system. In a 'tailor-made' scheme the signals at the key intersection are usually allowed to operate in a fully vehicle-actuated manner and to control the times at which the right-of-way is changed at the neighbouring intersections. This may be to favour traffic leaving the main intersection (which can be described as forward linking) or to prevent a queue back from the key intersection interfering with the previous intersection (which can be described as backward linking). The following are the most common forms of link and may be used either for forward or backward linking:

(a) Commencement of a particular phase at the key intersection gives a demand for a selected phase at the controlled intersection. Detector operations at the key intersection may be repeated at the controlled intersection.

(b) Commencement of a particular phase at the key intersection causes a forced change to a selected phase at the controlled intersection (subject to the minimum green on the running phase having expired).

(c) As (b) but in addition the maximum timer at the controlled intersection is disconnected for the duration of the phase at the key intersection.

(d) As (b) but in addition both the vehicle-extension timer and the maximum timer at the controlled intersection are disconnected for the duration of the phase at the key intersection. This facility can be used only when there is a permanent demand for some other phase at the key intersection.

The transmission of the pulses from the key intersection may be instantaneous or may be delayed to allow for the travel time between intersections. One disadvantage of this method of linking is that progression towards the key intersection is considerably less efficient than progression away from it, but under heavy traffic this can in practice be an advantage since it lessens the risk of congestion at the key junction by facilitating movement away from rather than towards that junction. Each particular location should be studied carefully to arrive at the most suitable system. More details of tailor-made systems are given in Appendix 2.

Delays and optimum settings of linked signals. Little research has been carried out on the optimum settings of linked signals. Delay, capacity, safety, stopping and starting are all important parameters which should be considered. Even if delay alone is considered there is little information available to guide the engineer in choosing a system which takes account of the instantaneous distribution of speeds, and variations in mean speed and traffic flow throughout the day.

With unequally spaced intersections on a two-way street, assuming constant speed throughout the system, the through bands of the time-and-distance diagram (see Fig. 6) do not coincide at every intersection. This means that at certain intersections (e.g. intersections A, B, D and E in Fig. 6) the platoons of vehicles from one direction arrive before those from the opposite direction. This causes inefficient operation of the system because more green time is required on the main road at such intersections for a given through-bandwidth and hence less green time can be allocated to the side roads. Time-and-distance diagrams are normally produced on a trial and error basis as outlined in Appendix 3. However, Morgan and Little(4) have described a method for synchronizing the signals on a route to produce bandwidths which are equal and as large as possible, given the green and red durations for each signal and the desired speeds of progression in each direction between adjacent signals. Their method can be used to adjust the synchronization to increase one bandwidth to some specified value (provided it is feasible) and to maximize the other. Even though their method takes account of different speeds in the various sections of the system it assumes that in any one section all vehicles travelling in the same direction have the same speed. They discuss the merits and limitations of other 'bandwidth' methods of synchronizing signals, namely, those proposed by Matson, Smith and Hurd,(5) Petterman,(6) Raus,(7) Bowers,(8) Davidson,(9) and the Traffic Engineering Handbook.(10)

Details of some interesting work on a mathematical treatment for linked signals have been given by Newell,(11)(12) who took the variation in speed of individual vehicles into account. He considered two cases: very light flow and almost saturated flow. For light flow conditions(11) the mathematical analysis indicated that with short distances between lights the optimum phasing depended on the proportion of green time to cycle time allocated to the progressive phase; with large distances between lights it appeared that the usual progressive timing was likely to cause greater delays than a random synchronization of the lights, but there are other schemes of synchronization which would be better than the random arrangement. For very heavy unidirectional traffic Newell(12) found, using certain assumptions, that the best synchronization for consecutive pairs of lights gave the best overall synchronization for unidirectional traffic. The optimum scheme is one for which the tail car of a platoon suffers no delay but the lead car is stopped. For heavy two-way traffic his analysis suggested that the optimum scheme was likely to be the one which was optimum for the direction with the lower flow. However, Newell concluded that much more research was required on this subject and the results should, of course, be tested under practical conditions.

A method of calculating the delay in a linked system, based on a graphical description of the traffic flow, has been given in an unpublished paper* by N. Forchhammer, a Danish signal engineer. The method can be used to estimate the delay for different levels of flow through the system, assuming all vehicles travel at the same speed. The effects of variation in mean speed can be determined as well as the effect of having a distribution of speeds at very low flows—this can give useful information on night-time settings. Using the method described, alternative signal schemes can be compared quickly and easily. A modification of Forchhammer's method has been described by Dick.(13)

The Laboratory has undertaken research on the simulation of a set of fixed-time linked signals using an electronic computer.(14) A method of linking fixed-time signals to give minimum delay over certain types of network has been proposed by Whiting and described by Hillier(15) in a paper which also gives proposals for unconventional forms of signal control.

Linking a network of streets. All the systems of linking mentioned previously can be applied, with varying degrees of success, to a network of streets. If the network consists entirely of one-way streets with alternate directions a useful basic system is the 'quarter-cycle offset' system in which the green periods at adjacent intersections are displaced by a quarter of a cycle. Figure 7 shows a grid network of streets with the time (in cycles) of the beginning of the green periods marked at each intersection. The arrows show the platoons of moving vehicles at the beginning of their green periods. With this type of system the platoons travel one block in a quarter-cycle compared with a half-cycle with the regular alternate system (for two-way streets). Since block lengths are usually fairly short in the centres of urban areas, the cycle time which gives a reasonable progressive speed is often much too short with the alternate systems, whereas a more reasonable cycle length can usually be found when the 'quarter-cycle offset' system is used.

Area traffic control. Consideration is now being given to the use of digital computers to provide systems of area traffic control.(15)(16)(17)(18) The purpose of such control is to reduce delays by

(a) better methods of linking signals according to the traffic situation at any given time;
(b) diversion of traffic away from congested routes to alternative routes where spare capacity is available; and

*'Calculation of delay in linked street signal systems based on a graphical description of traffic flow in the system'

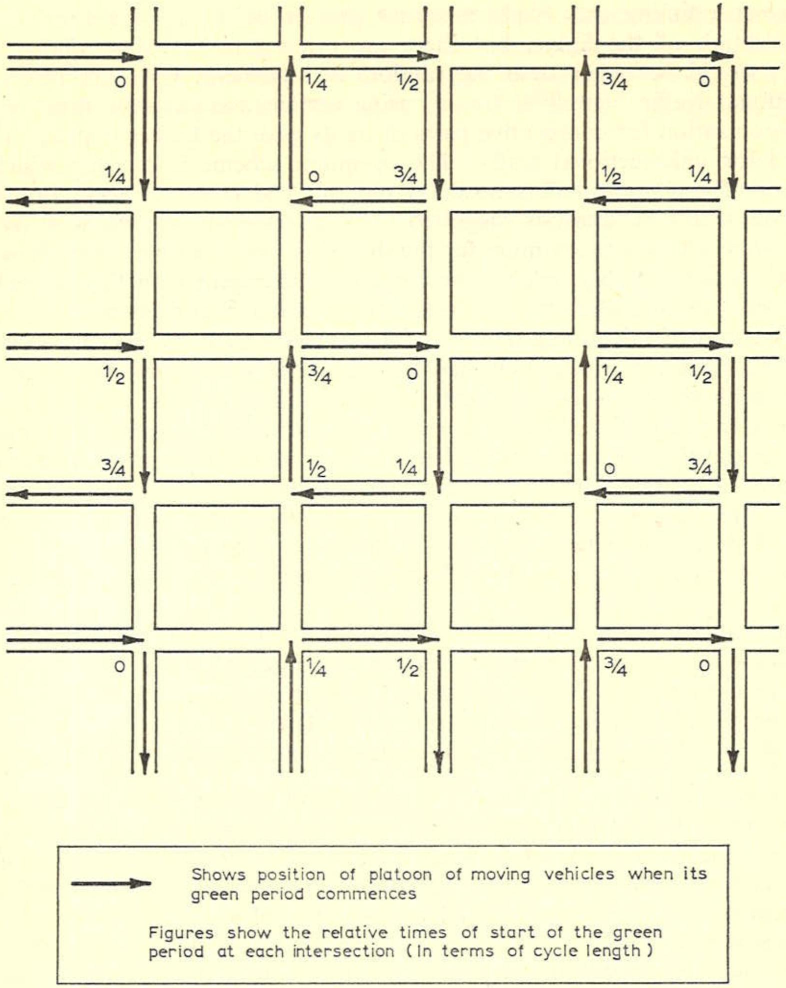


FIG. 7. 'Quarter-cycle offset' system for a grid network of one-way streets

(c) lane-switching, or switching of peak-period one-way systems, on tidal-flow routes.

Digital computers can provide additional facilities for

(a) preventing forward movement from one signal installation when the queue back from the next installation reaches a critical point;
(b) giving priority to forward movement at the next installation in order to clear the queue;
(c) banning of right turns (accompanied by re-routeing) when this manoeuvre causes disruption of through-traffic movement;
(d) switching in and out of special facilities for right-turning traffic in accordance with the general traffic requirements of the area; and
(e) emergency arrangements for traffic control when normal conditions are interrupted by accidents, roadworks, special events, weather and so on.

Pedestrian signals

At intersections controlled by signals the requirements of pedestrians are catered for in two ways in Great Britain. One method is to provide a crossing marked out in studs in front of the stop line (see Fig. 10) for use by pedestrians during normal signal timings, i.e. no special phases are given for them. This arrangement is normally used at intersections where turning traffic is not heavy. In the second method pedestrians' movements are controlled by separate signals during a special phase. This is a more positive method as all traffic is halted before the pedestrian phase is given, but it causes greater delay to vehicles.

Pedestrian signals have two aspects. The current Traffic Signs Regulations and General Directions[19] provide for the introduction of one aspect showing a red figure of a stationary man on a black background and the other showing a green figure of a walking man on a black background*, but these signals have not yet been introduced, and present signals show the word 'WAIT' in red on a black background, and the word 'CROSS' in white or green on a black background. With the present signals, the 'CROSS' indication is usually displayed for a pre-set period of 6 to 10 seconds according to the pedestrian flow, and is followed by a clearance period of 2 to 8 seconds during which all vehicle signals are at red and no signal is displayed to pedestrians. The 'WAIT' signal to pedestrians is then displayed coincident with the red/amber of the next vehicular phase, and continues until the green pedestrian signal is next given. The combined length of the pedestrian phase, the clearance period, and the following red/amber period is usually based on the time taken to cross the road at 4 ft/second. Where pedestrian flows are very heavy, longer times are given if the traffic situation permits. If the transit time to the pedestrian crossing for traffic starting up on the next vehicular phase is appreciable, appropriately shorter times may be given. When, however, the transit time permits of a reduced clearance period it is essential on multi-phase installations to ensure that adequate clearance is given to each possible following phase, e.g. if reduced clearance is given on phase change B to C (B being the pedestrian phase) a longer clearance may be needed in the phase change B to D on occasions when phase C is omitted.

A short all-red period is usually inserted before the green pedestrian signal is displayed to ensure that traffic is clear of the crossing before pedestrians are signalled to cross. The pedestrian phase may be introduced either (a) by operation of a pushbutton—this is the normal arrangement and avoids unnecessary delay to vehicles, or (b) automatically: this may be desirable particularly with linked signal systems to prevent signals with a pedestrian phase getting seriously out of step with adjacent signals.

Although pedestrians may normally be allowed to cross over any of the approaches to an intersection there will usually be one on which the pedestrian problem is most acute. The pedestrian phase should immediately follow the end of the vehicular phase on this approach.

These signal arrangements can also be used for pedestrian crossings sited between junctions. With one type of pedestrian-operated signal no vehicle detectors are installed and right-of-way normally rests with the traffic, but

*The symbolic pedestrian signals described above are being introduced in accordance with the recommendation of the Worboys Committee on traffic signs. Current experiments with pedestrian signals may lead to changes in the operation of signals as described above when the pushbutton is depressed the pedestrian receives right-of-way immediately (provided a pre-set minimum right-of-way period for vehicles has expired since the pedestrian phase was last called). It is often possible to omit vehicle detectors without difficulties arising, particularly in linked systems, but where vehicle approach speeds are high the installation of detectors enables the change to a pedestrian phase to be made wherever possible during a gap in the traffic, thus avoiding arbitrary changes which, with a 3-second amber, may give traffic insufficient warning to stop. In such cases the detectors, being for fast vehicles, should be sited some 250 to 300 ft from the crossing rather than at the standard distance—see 'Location of detectors'.

One difficulty with pedestrian signals as described above is that the pedestrian phases and clearance periods, being of fixed duration, have to be set to meet average conditions. This results in unnecessary delays to vehicles when only a few pedestrians wish to cross, and to inadequate time for pedestrians at their peak periods. Experiments are, therefore, in hand with a more flexible type of control for pedestrian crossings, where the signal sequence to vehicles includes a flashing amber period following the red signal: during this period vehicles must give way to any pedestrians wishing to cross, but may move over the crossing in the absence of pedestrians. Detectors would be required only where vehicle approach speeds were high.

Zebra crossings (uncontrolled) are a common type of pedestrian crossing in urban areas but the delay which they cause to vehicles increases greatly with increasing flow of pedestrians. With pedestrian flows across the road of more than about 1000 per hour the signal-controlled pedestrian crossing gives less delay to vehicles than the Zebra crossing.

The requirements of pedestrians are also discussed in 'Location of signals' and 'Pedestrian crossings'.

Signal equipment

Controller and signal assemblies. Signal equipment for use in Great Britain must conform to the appropriate British Standard concerning tolerances in timing and the components and materials used. The current Standard, at present being revised, is B.S.505:1939, 'Road traffic control (electric) light signals'.[20]

The functional requirements of the equipment must satisfy the current Ministry of Transport's 'Specification for Vehicle-Actuated Road Traffic Control (Electric) Light Signals'.

Detectors. Detectors used in Great Britain are at present of the pneumatic type, and consist of two tubes spaced a few inches apart. The time taken for a vehicle to travel between the first and second tube provides the means of speed-timing the vehicle-extensions. The two tubes also render the detector inoperative to vehicles passing over in the opposite direction. Single-tube detector pads, which are operated by vehicles crossing them in either direction, are simpler and cheaper than unidirectional detectors which are only operated by vehicles approaching an intersection. With single-tube detectors vehicles leaving an intersection, especially on a narrow road, are liable to cross the centre line (e.g. in passing a parked vehicle) and operate the detector pads for that approach either causing the signals to change unnecessarily or prolonging the green period unnecessarily and thereby increasing the delay to waiting

traffic. The difference in cost between single- and double-tube detectors is not very great and it is doubtful if the saving is worthwhile. Double-tube detectors which are arranged to give speed-timing and to be unidirectional are used in Great Britain, except for all-red extending detectors on bridges with shuttle working where 'both-way' operation is required.

Maintenance. Servicing of signals in Great Britain is normally carried out by the installing company on a 'beck-and-call' basis. In other countries where traffic signals are installed by British companies it is usual for the appointed agents of the companies to be responsible for this service, which includes inspection and overhaul every three months. Servicing normally includes controller and detector maintenance and may also cover group lamp replacement and maintenance of the optical systems. Premature lamp failures, cleaning and painting are usually dealt with locally and possibly also group lamp replacement and optical system maintenance.

In a balanced 2-phase installation lamps having a 1000-hour nominal life are normally replaced in the red and green signals every three months, and those in the amber signals every nine or twelve months. In unbalanced 2-phase installations, or where there are more than two phases, the replacement periods for the red and green lamps may need to be shorter. In extreme cases it may be worthwhile to interchange the red and green lamps after six weeks, and renew after three months. Internal cleaning of the optical system may be combined with lamp replacement: external cleaning may be needed more frequently at 'dirty' sites. A daily check for premature lamp failures and also, if possible, for detector faults is recommended.

Detector faults (other than intermittent troubles) normally result in failure to respond, or in a permanent demand. Assuming the signal installation appears generally to be operating satisfactorily, failure to respond can be checked by ensuring that each detector on a phase will, when depressed, initiate a demand, or give an extension. A permanent demand will not only cause the signals to switch to a phase in the absence of traffic but will hold the phase for the maximum running period. At installations arranged to revert to a selected phase in the absence of demands from other approaches, failure to respond can generally be readily recognized by failure to give extensions, and permanent demand can be recognized by holding to maximum. A permanent demand will of course mask any other fault on other detectors on the same phase. Detection of faults on linked systems is more difficult and requires a detailed knowledge of the mode of operation.

WARRANTS FOR SIGNALS

In Great Britain there are no standard requirements on which the necessity for signals is based, each situation being judged on its merits. Broadly speaking, the three primary aims of signal control are:

(a) to reduce traffic conflicts and delay;
(b) to reduce accidents;
(c) to economize in police time.

The first of these aims is the most common justification for signals nowadays. Delays at junctions of the priority type (major/minor) have been studied theoretically[21] and several comparisons have been made of the delays at junctions with and without signal control, one example being shown in Fig. 8. It can be seen that under light flows overall delay is greater with signals than

FLOW RATIO (major to minor road) = 4:1

PRIORITY-TYPE JUNCTION

(see Table 1 for definition of β_M, β_m and α

TYPE A (good visibility)	TYPE B (poor visibility)
β_M = 2 seconds	β_M = 2 seconds
β_m = 3 seconds	β_m = 5 seconds
α = 6 seconds	α = 8 seconds

TRAFFIC SIGNALS

Saturation flow (major) = 1800 vehicles per hour

Saturation flow (minor) = 1200 vehicles per hour

Intergreen times = 4 seconds each changeover

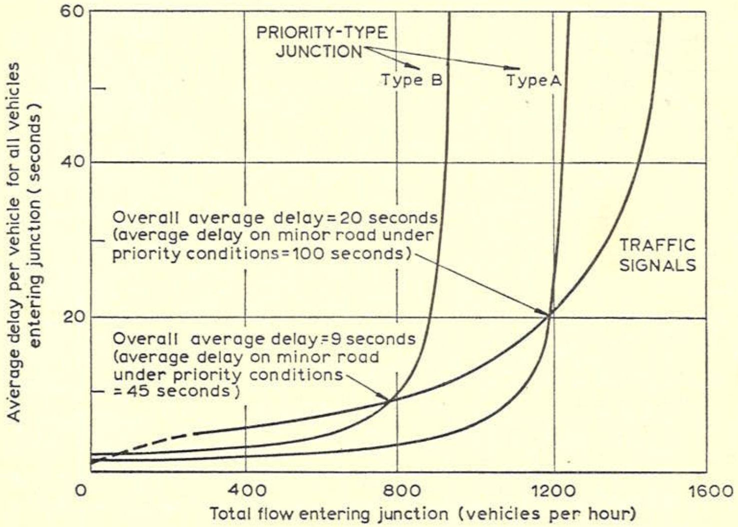


FIG. 8. Theoretical delay/flow curves for a hypothetical T-junction. Comparison is made between priority control and signal control

without but the reverse is true under heavy flows. The diagram supports the general finding that capacity is increased with signal control; this increase is often considerable. Figure 9 shows a typical example of the comparison of capacities. In order to give some idea of the magnitude of the maximum flows which can pass through uncontrolled junctions the results of some calculations for simple hypothetical T-junctions are given in Table 1. These are ultimate theoretical capacities (with correspondingly very high delays on the minor road). The capacities in these particular cases vary from 750 to 1650 vehicles per hour; it is to be expected that, when the ratio of major to minor flow is as high as 10:1, the maximum total flow entering the junction would be much higher than when there are equal flows on both roads.

One of the main advantages of signal control is that it generally eliminates much of the stress and difficulty of negotiating a heavily trafficked junction. The Ministry of Transport considers the minimum justification for signal control to be an average flow over 16 hours of the day of about 300 vehicles per hour,

For details of the priority-type junctions and traffic signals used in this example see Fig. 8.

To obtain the ultimate capacity read off the maximum minor-road flow for a given major-road flow and add these figures.

The dashed lines show that for a 4:1 flow ratio the respective capacities are

Traffic signals: 310 + 1240 = 1550 vehicles per hour

Priority junction: Type A: 252 + 1008 = 1260 vehicles per hour; Type B: 190 + 760 = 950 vehicles per hour

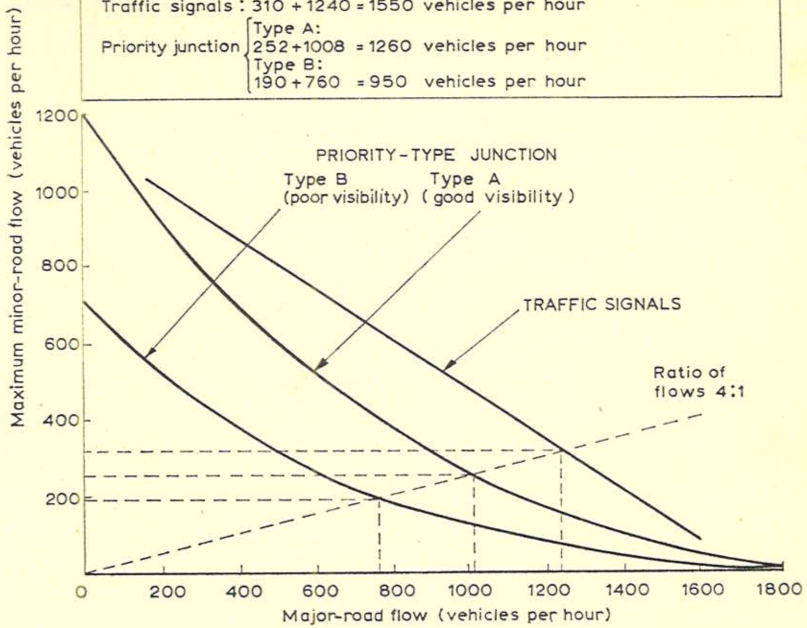


FIG. 9. Ultimate capacity of hypothetical T-junction with traffic signals or priority-type control

of which at least 100 vehicles per hour are on the minor roads. This would be equivalent to a peak-hour total flow entering the intersection (taken as about 10 per cent of the 16-hour total) of about 500 vehicles per hour. These figures are intended for guidance and not as absolute criteria, as many uncontrolled junctions are working perfectly satisfactorily with flows substantially in excess of these figures, as would be indicated by the figures in Table 1. The Ministry has suggested that in certain extreme cases of junctions controlled by 'Halt' signs it may be reasonable to install signals when the side-road flow is only 80 vehicles per hour if the main-road traffic is very heavy and long queues develop on the side road as a result. This again fits in reasonably well with theoretical calculations, as it can be seen from the bottom row of the right-hand column of Table 1 that the ultimate capacity of the minor road would be about 90 vehicles per hour when the ratio of major-road flow to minor-road flow is 10:1.

Studies of accidents at a sample of 21 sites revealed a 40 per cent reduction[22] when signal control replaced no control, or control by 'Halt' or 'Slow' signs. However, accidents were observed to increase when signals replaced roundabouts (two sites) and to decrease when roundabouts replaced signals (three sites).[22] Other information on the safety aspects of signal control is given in reference (23).

Signal control may be expected to reduce certain types of accident (e.g.

Table 1

Calculated ultimate capacities of some typical T-junctions

	α	β_m	β_M	ULTIMATE CAPACITY (vehicles per hour) Major/Minor flow ratio: 1	4	10
TYPE A* (good visibility)	6	3	1	1150	1400	1650
	6	3	2	1100	1250	1450
	6	3	3	1050	1100	1150
TYPE B* (poor visibility)	8	5	1	800	1000	1250
	8	5	2	800	950	1150
	8	5	3	750	900	1000

* α is the minimum gap (seconds) in the major stream which will be accepted by one vehicle from the minor stream

β_m is the minimum average time interval (seconds) between successive vehicles emerging from the MINOR road, i.e. when there is no flow on the major road

β_M is the minimum average time interval (seconds) between successive vehicles on the MAJOR road, i.e. when the flow is saturated

collisions between vehicles moving at right angles to each other) but is likely to increase some other types of accident (e.g. nose-to-tail collisions). A knowledge of the average number of accidents per annum at a particular site, and a study of movements before impact, may help in deciding whether signal control will be beneficial, and whether or not there is a *prima facie* case for considering signals on safety grounds. Records show that the average number of personal-injury accidents per annum at signalled junctions is about two for Great Britain and six for the Greater London area. It will be appreciated that the actual number of such accidents varies widely from site to site.

Signals are installed, even if the above warrants are not satisfied, if they are needed to form part of a linked system.

Elimination of police control, even if only part-time, would almost always be financially attractive, but the policeman has special advantages in dealing with turning vehicles (because he can see their direction indicators), in helping pedestrians, especially young children, and in dealing with jamming of the intersection.

In the U.S.A., where fixed-time signals are common, the traffic warrants for this type of signal are given in the *Manual on Uniform Traffic Control Devices for Streets and Highways.*[24] There are many times more signals per 100 miles of urban road in the U.S.A. than in Great Britain,[23] probably because of the grid-iron pattern of streets common to most American cities, which lends itself to linked signal systems. In the U.S.A. the minimum vehicular warrant for fixed-time signals in urban areas is a major-road flow (both directions combined) of 500 or 600* vehicles per hour for each of 8 hours of the day and

a flow on the busier minor road (approach direction only) of 150 or 200* vehicles per hour for the same 8 hours of the day. Where operating conditions on a major road are such that minor-road traffic suffers undue delay or hazard in crossing or entering the major road, the above warrants are adjusted to 750 or 900* vehicles per hour for the major road and 75 or 100* vehicles per hour for the busier minor road. The minimum warrant for pedestrian signals is 150 persons per hour crossing the major road on the busier crossing for each of 8 hours of the day, coupled with a major-road flow of 600 vehicles per hour for the same hours (1000 vehicles per hour if there is a median island). In rural areas and in isolated built-up areas the minimum warrants are 70 per cent of the requirements given above.

As in Great Britain, signals are installed even though the above warrants are not satisfied if they are needed to form part of a linked system. No firm minimum vehicular warrants are given in the U.S.A. for vehicle-actuated signals, which are installed if other conditions indicate the need for them and if they are likely to justify their cost.

LAYOUT OF INTERSECTION AND SITING OF SIGNAL EQUIPMENT

The locations of signals, stop lines, lane lines, traffic islands, pedestrian crossings, etc., recommended in Great Britain are shown in Fig. 10.

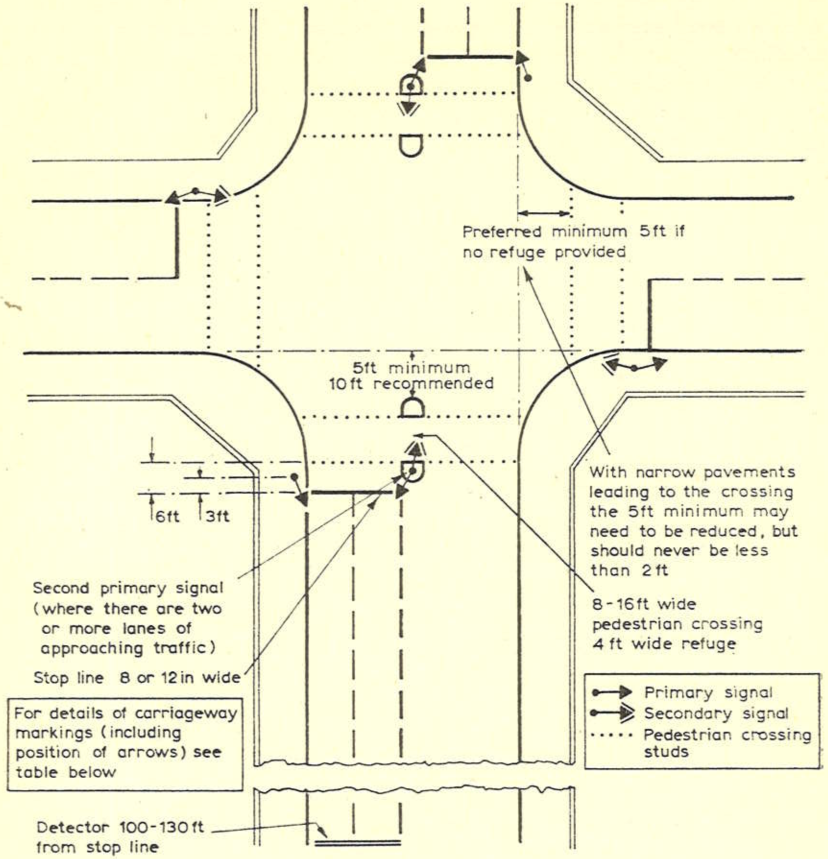
Location of signals

Normally one primary signal is installed 3 ft from the stop line, as shown in Fig. 10, and a second primary signal is usually installed if there is a central island. A secondary signal is commonly installed diagonally opposite the first primary signal (i.e. on the back of the primary signal which faces opposing traffic). The secondary may be opposite the outer approach lane or within an arc of 30° towards the offside of the centre line extended into the junction from the stop line, and should be as close as possible (see Fig. 11). If long distances of the order of 180 ft or more are unavoidable, then additional signals may be necessary.

If, at an intersection, the road opposite a particular approach is one-way into the intersection it is normal practice in Great Britain to exhibit two green arrows mounted side by side, as shown in Fig. 12, instead of the full-green aspect.

At sites where special pedestrian signals are not provided it is generally desirable that the vehicle signal indications should be visible to pedestrians as an aid in judging when to cross. At a simple four-way intersection having primary and secondary signals on each approach a pedestrian wishing to cross will usually have a view of at least two signals—one applicable to traffic he is about to cross and one applicable to traffic on the other phase. These indications, together with the pedestrian's assessment of the traffic situation, should be sufficient to ensure a safe crossing. If the intersection layout is not simple, then additional signals (see Fig. 13) for pedestrians may be desirable.[25] Additional signals may also be needed in one-way streets where there may otherwise be no signals in the pedestrian's view. Where such additional signals would show green towards any vehicle which happened to be travelling wrongly against the one-way system it is desirable to mask the green signal so that only a green cross (X) is visible.

*Depending on the number of lanes



CARRIAGEWAY MARKINGS

	WARNING LINES					
	30 mile/h limit		40 mile/h limit		over 40 mile/h limit and derestricted	
	Centre* lines	Lane lines	Centre* lines	Lane lines	Centre* lines	Lane lines
Length of mark (feet)	12	12	12	12	18	18
Gap (feet)	6	6	6	6	9	9
Width (inches)	6	4	6	4	6	4
Minimum number of marks	7	5	10	7	10	7
	ARROWS					
Distance from tip of arrows to stop line (feet)	50 150		80 240		120 360 (maxima)	

* Centre lines used together with lane lines for 4-and 6-lane roads; for 2-and 3-lane roads lane lines only used

FIG. 10. Typical layout used in Great Britain for traffic signal installation

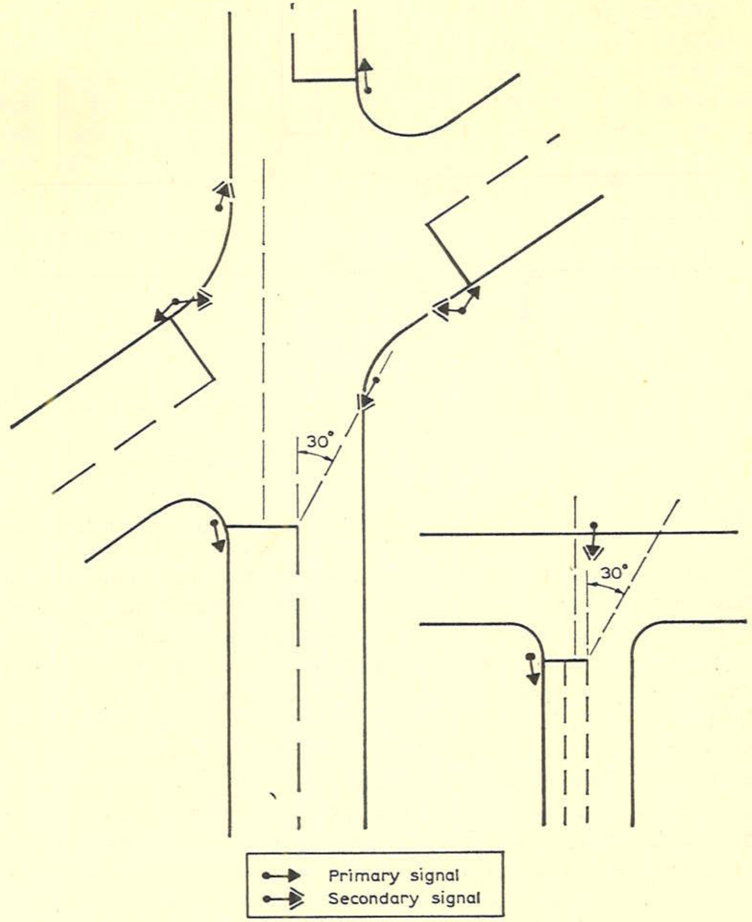


FIG. 11. Typical layouts showing positioning of secondary signals

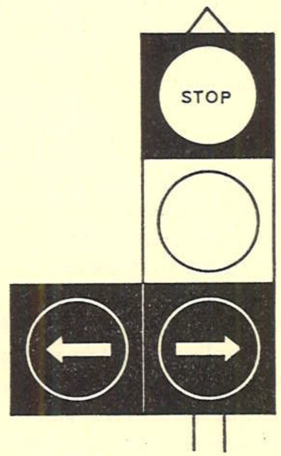


FIG. 12. Signal with two-green-arrow aspect

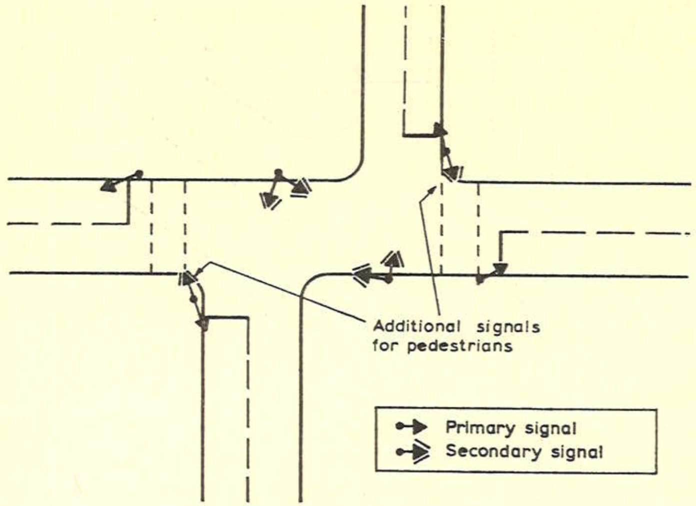


FIG. 13. Example of layout requiring additional signals for pedestrians

If more than two phases, or overlap phases, or filtration is employed, there is a risk that signal indications will confuse or even mislead the pedestrian. This arises since it is not practicable to present all indications being exhibited simultaneously which would be needed to determine what is happening—nor could a pedestrian readily interpret all indications even if given. For such situations it is generally preferable to leave the pedestrian to his own judgment of the traffic situation. It will often be worthwhile to re-arrange the vehicle signals so as to minimize the risk of pedestrians seeing vehicle signals which could be misleading. For example, with an early cut-off, the approach on the side which is cut off early may be arranged to have one primary signal on the nearside footpath and one secondary signal on the trailing edge of the central refuge of the approach concerned (instead of having a second primary on the leading edge of the refuge and a secondary on the far side of the junction). By having both primary and secondary on the approach to the junction there is less risk of pedestrians (and also right-turning drivers on the phase which is cut off early) seeing these signals and assuming, when they go to red, that all traffic movements in the junction have been stopped.

Alternatively, pedestrians may be helped by other means, e.g. a special pedestrian phase.

Carriageway markings

Drivers should be encouraged to queue in as many lanes as is practicable and lane lines on the approaches can help in this connexion (see 'Lane widths'). Details of lane lines and centre lines (thickness, length, gap, etc.) are given in the table in Fig. 10.

Carriageway markings within the intersection itself can assist drivers in taking the correct path through a complicated intersection. They can be of assistance also at junctions where there are opposing right-turners using the same phase and passing each other on their nearsides (see 'Layout for right-turning vehicles'). In some countries dotted stop lines are marked in the right-turning lanes if vehicles waiting to turn right are not prohibited from turning during the green

period for opposing straight-through traffic (see Plate 1). Where there are two or more lanes turning right from a single direction it is desirable to mark out the path they should take with paint or studs in such a way that cross-traffic is not confused by the markings. Plate 1 also shows an example of this in Germany.

Arrows together with abbreviated destination marks painted on the carriageway well before the intersection can often be useful in advising drivers of the position on the road which they should adopt. In the queueing lanes arrows should be used to denote the lanes allocated exclusively to a particular movement of traffic, e.g. right-turning traffic. The table in Fig. 10 indicates where on the approach arrows should be placed.

Studs are generally used to mark out any pedestrian crossing at the intersection (see 'Pedestrian crossings').

Traffic islands

Islands of 4-ft minimum width placed at or near the centre of the carriageway as shown in Fig. 10 are used mainly for the benefit of pedestrians. However, islands are also used for channelizing traffic: sometimes opposite islands are offset to give greater width for traffic entering the intersection; sometimes the islands are movable to facilitate reversible working of certain traffic lanes. When islands are used the width remaining to traffic on either side of them must be at least 14 ft for single-lane traffic and at least 18 ft on any side which normally carries two lanes of traffic. Islands should contain bollards (illuminated at night) with arrows indicating 'Keep left' or 'Pass either side' as appropriate.

Location of detectors

The current recommended distance (Great Britain) of the detectors from the stop line is 130 ft, but on roads in urban areas which are narrow, winding or which have appreciable gradients, or where speeds are not likely to exceed 30 mile/h, the detectors are located between 100 and 120 ft from the stop line. At T-junctions with awkward turns the detectors on the side road are only 60 ft away from the stop line.

For fast roads a new type of control equipment should shortly be in production which will have detectors at a considerable distance from the stop line (of the order of 500 ft) in addition to detectors in the normal position. Until this controller is available for use on this type of road a system of double detection is being used (see 'High-speed roads') with two detectors at approximately 130 ft and 240 ft from the stop line.

GEOMETRIC DESIGN

Some aspects of geometric design, namely visibility, kerb radii, channelization, etc., are dealt with in 'Urban Traffic Engineering Techniques'[2] and in 'Roads in Urban Areas'[3] and are not covered in this Paper. Some of these aspects, particularly channelization, are also considered in Special Report 74 of the U.S. Highway Research Board[26] and by Matson, Smith and Hurd,[5] Feuchtinger,[27] and Korte, Macke and Lapierre.[28]

The approach

Because signals permit traffic movement from any approach for only a proportion of the time, it is sometimes necessary for the intersection approaches,

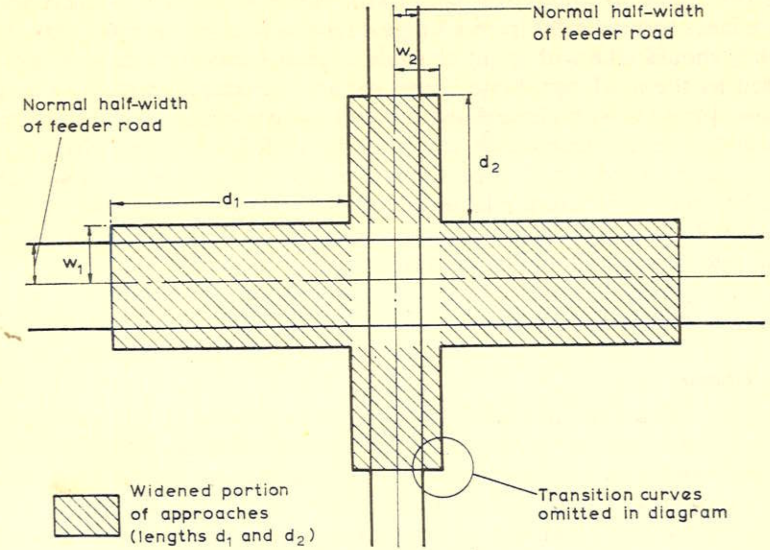


FIG. 14. Simplified diagram of widened approaches at signal-controlled intersection

where queueing takes place, to be wider than the roads which feed these approaches (see simplified layout in Fig. 14), in order to pass the required flow. If the intersection already exists, the timing of the signals can be adjusted for a given flow pattern to make the best use of the existing layout (see 'OPTIMUM SETTINGS: FIXED-TIME SIGNALS'). If the intersection is in its design stage, or if some changes can be made to the layout of an existing intersection, then a choice of approach widths may be available, after selection of which the green times can be adjusted to give the correct capacities for those approaches.

In selecting approach widths, one criterion which can be used is the minimization of the area occupied by the intersection. The selection of approach widths (w_1 and w_2 as in Fig. 14) was considered by Webster and Newby,[23] who assumed in their model that the maximum possible rate of flow past the stop line was proportional to the width of the approach (w_1, w_2) and also that the widened sections of the approaches (lengths d_1, d_2) were just long enough to accommodate the queues which could pass through the intersection during fully saturated green periods. They ignored the area which would be taken up, in practice, by tapers between the feeder roads and the widened approaches. With these assumptions they concluded that over the practical range of the ratio w_1/w_2 there was little difference in the required area of carriageway, and they suggested a rule for determining the width ratio which (a) gave greater widths as well as longer green times to the approaches carrying the higher flows, and (b) minimized the sum of the approach widths at the intersection.

By virtue of (a), extreme ratios of either approach width or green time are avoided and the intersection is more adaptable to changes in flow pattern in the surrounding area. The results of (b) are that it is more convenient (and perhaps safer) for pedestrians crossing the road, clearance distances are reduced and encroachment on pavements and frontage development is reduced.

Recently the Laboratory has constructed a more realistic theoretical model

which includes tapers of 1 in 10 between the feeder roads and the widened parts of the approaches, and the Laboratory's Pegasus II computer has been used in a wide variety of cases to find the signal settings and the ratios of approach widths which minimize the areas occupied by the intersections. The optimum ratios of approach widths were found to be materially the same as those given by the simple rule proposed by Webster and Newby.[23]*

The rule may be stated as follows:

For a normal two-phase cross-roads the approach widths should be proportional to the square roots of the flows. The green times and lengths widened should be in the same ratio as the widths, i.e.

$$\frac{w_1}{w_2} = \frac{g_1}{g_2} = \frac{d_1}{d_2} = \sqrt{\frac{q_1}{q_2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where q_1 and q_2 are the maximum flows on phases 1 and 2 respectively. Thus, a major road carrying four times as much traffic as its minor cross-road should have approaches which are twice as wide as the minor approaches and have green times which are twice as long.

Any approach width deduced from the application of this rule is subject to a minimum equal to that of the associated feeder road. Thus, where the rule suggests a width less than that of the feeder road the width of the latter is used and the green time made correspondingly less. The extra green time thus allocated to the other phase results in less widening being necessary on those approaches.

Generally, the maximum flows on the two (or more) arms of the same phase are approximately equal (though often occurring at different times of the day owing to tidal effects), but where this is not so the highest flow should be used with the above formula to determine the approach width and then, after the green time for the phase under consideration has been determined, the width of the arm with the lower flow can be determined.

The rule can be extended to cover forks and other intersections controlled by 3-phase signals by making the ratios of the widths

$$w_1 : w_2 : w_3 = \sqrt{q_1} : \sqrt{q_2} : \sqrt{q_3} \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

As before, the green times and lengths widened have the same ratios. The rule can in a similar way be extended to four or more phases.

For T-junctions with 2-phase control the ratios of widths, green times and lengths widened should be

$$\frac{w_1}{w_2} = \sqrt{\frac{q_1}{2q_2}} \text{ and } \frac{g_1}{g_2} = \frac{d_1}{d_2} = \sqrt{\frac{2q_1}{q_2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where the suffix 2 refers to the stem of the T-junction. Thus, a major road through a T-junction carrying four times as much traffic as the stem should have a width 1·4 times that of the stem, a length of widening 2·8 times and a green period 2·8 times as long as that of the stem. (See worked example No. 1 in Appendix 7.)

*A different approach to the minimization of the area needed at signals has been given by Smeed and Hillier[29]

For a cross-roads where one of the arms is of insignificant traffic importance and need not have widened approaches the T-junction rule would be more applicable than the cross-roads rule.

It is suggested that these results should be used initially to obtain a preliminary assessment of the approach widths; if necessary, the widths obtained should be modified according to site conditions. In some cases the presence of existing buildings near to the intersection may prevent uniform widening of the approach from being carried out, and it may only be possible to flare the approach. Where the approach is a dual carriageway extra space can be gained by reducing the width of the central reserve (if more than 4 ft) to 4 ft.

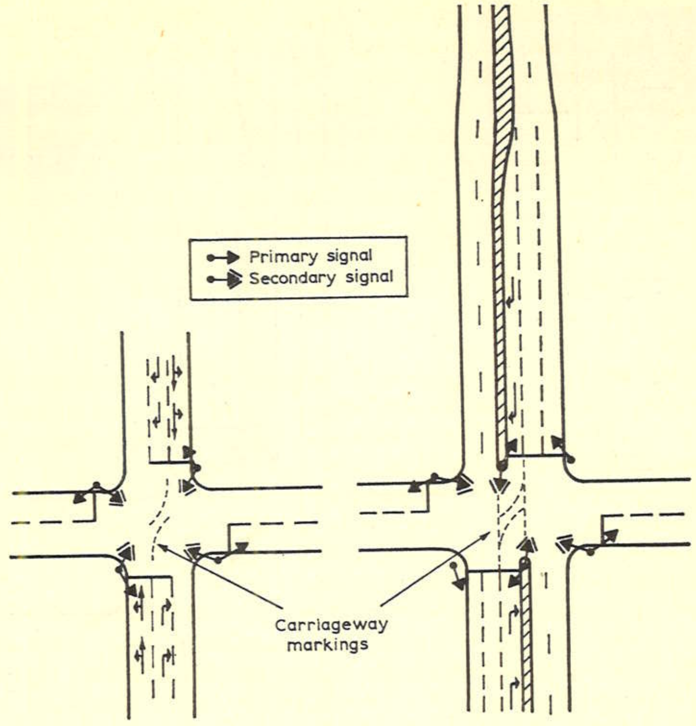
Lane widths

It is normal practice in this country for lanes to be 10 ft wide at an intersection, though occasionally 9-ft lanes have to be accepted at some existing intersections. Some countries have found that in certain cases capacity is increased by having very narrow lanes[30] (down to 7 ft) even though drivers of wider vehicles are unable to keep within them. With queueing lanes wider than 10 ft it is likely that capacity would be wasted, though this depends on traffic composition; for example, where there is a high proportion of bicycles, or of wide vehicles, it may be beneficial to have a wider nearside lane.

Some traffic engineers recommend having the same number of lanes on the exit side of the intersection as there are straight-through lanes (partly or exclusively used by straight-through traffic) on the approach side.[31][32] If, however, site conditions make it necessary to have fewer lanes on the exit side of the intersection, a distance of about 300 ft on that side should be allowed for merging to take place, though this could be reduced if there are many turning vehicles at the intersection, i.e. fewer vehicles going straight ahead. It is most desirable that vehicles travelling in through-lanes should not be obstructed by either parked vehicles or waiting right-turners, and the latter should wherever possible have their own lane or lanes.

Layout for right-turning vehicles

Opposing right-turners can turn on either the offside or the nearside of each other. In the former case they have good visibility and can see an approaching gap in the opposing stream in which to complete their turn. On the other hand, if there are too many turners from the two directions for the storage space within the intersection, the two streams may interlock causing congestion in the intersection. With the nearside method locking cannot occur but visibility is often restricted, and drivers usually have to wait until the end of the green period before turning in order to be sure that there is no opposing straight-through traffic. If the nearside method of turning right is used there may be advantages in offsetting the centre line (or the central reserve) so that more space is available to traffic approaching the intersection than to traffic leaving it. In some cases, it may be desirable to place the opposing right-turners opposite each other as in Fig. 15 (see also Plate 2). The layout suggested in Fig. 16 improves visibility for right-turners by allowing the leading vehicles in the right-turning queue to see round the opposing right-turners without encroaching on the straight-through lanes. This figure also gives suggestions for appropriate carriageway markings and details of phasing. Where the flow of right-turning vehicles is exceptionally high and it is necessary to provide more than one lane in each direction for them,



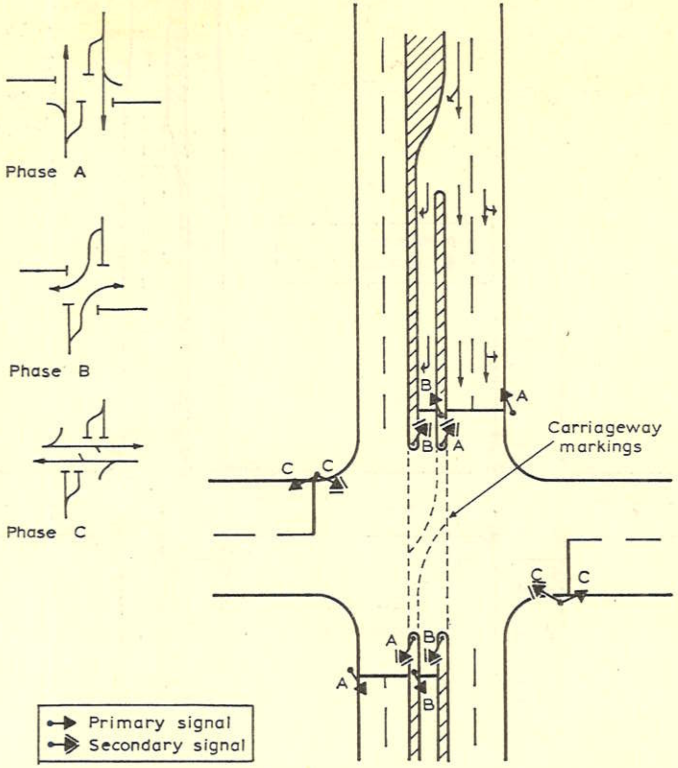
(a) Single carriageway (b) Dual carriageway

FIG. 15. Suggested layouts with right-turning lanes opposite each other

these lanes should be widened to at least 14 ft at the mid-point of the turn, especially if the turn has a radius of less than 50 ft (see also 'Carriageway markings').

Pedestrian crossings

The layout shown in Fig. 10 includes pedestrian crossings marked out in studs; this is a usual feature of intersections where there are appreciable numbers of pedestrians. A pedestrian refuge is usually placed at or near the centre of a single carriageway if the widths remaining to traffic in the two directions are sufficient (see 'Traffic islands'). Where pedestrians have to cross a very wide approach it is desirable to place the stop line well back from the crossing (about 20 ft) so that, when the vehicular phase begins, drivers can easily see if any pedestrians have not completed their crossing and can delay their start accordingly. It is desirable in some cases to restrict the crossing of pedestrians to certain approaches at an intersection and guard rails can be used to prevent pedestrians crossing at unmarked places (e.g. where filter streams may be moving at times unexpected by the pedestrian). On one-way streets pedestrians can be signalled to cross without any interference from turning traffic and without reducing the green times to traffic, but on two-way streets it is sometimes necessary to allocate a special phase to pedestrians if they are very numerous. More detail is given in 'Pedestrian signals' and 'Location of signals'. Experiments on other methods of signalling pedestrians are currently in hand.



Note: Phase B signals show right-turn green arrow in place of full green

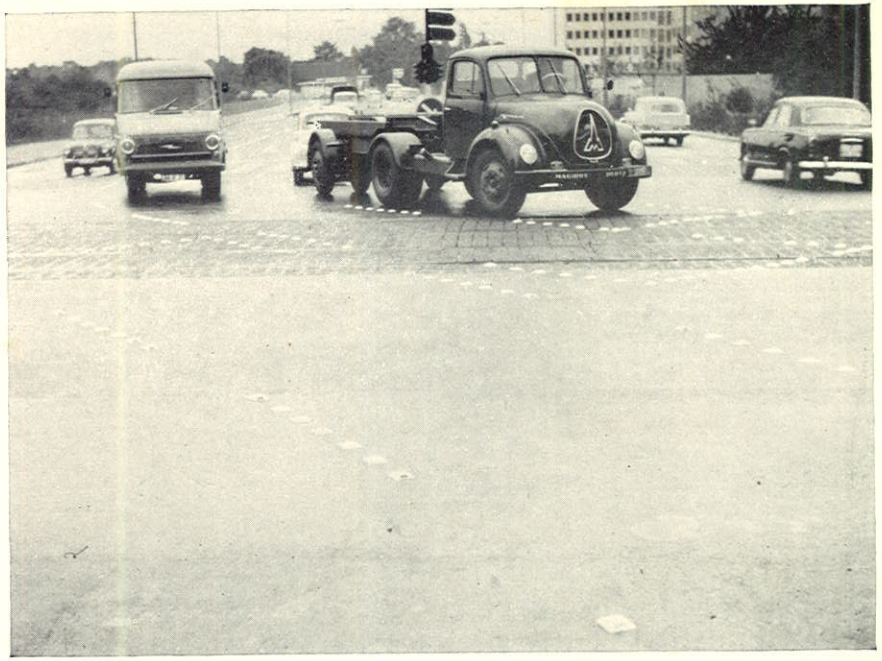
FIG. 16. Arrangements for dealing with heavy right-turning movements on opposing arms (see also Fig. 18)

Subways and footbridges provide a safer method of crossing the road, but pedestrians do not always use them unless the alternative surface-level path is such that it takes more time to cross. Guard rails are often used to make the surface path less convenient.(33) It should be noted, however, that the effort involved in using subways and footbridges is often not inconsiderable and further that many people, particularly women and old people, have a reluctance to use subways at night-time.

DESIGN OF SIGNAL SCHEMES

Phasing

The phasing depends mainly on the number of roads entering the junction and the amount of right-turning traffic. It is desirable to reduce the number of phases required to the least number which will work satisfactorily. Normally 2-phase control is satisfactory for straight cross-roads when there is not too much right-turning traffic. Special provision for right-turners is rarely needed for volumes of 60 per hour or less. This is because a small number of vehicles can usually turn right without difficulty during the intergreen period following right-of-way, and cycle times are on average 1 to 2 minutes, thus giving adequate

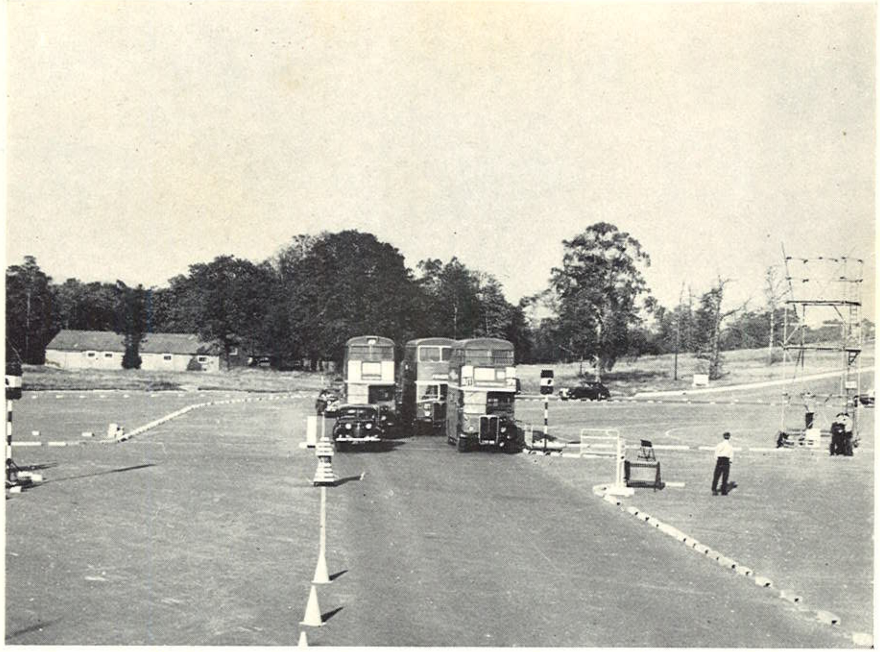


Carriageway markings for double left-turning lanes at a signal-controlled intersection in Germany (right-hand rule of the road)

PLATE 1

Double right-turning lanes at an intersection in Slough, Bucks

PLATE 2

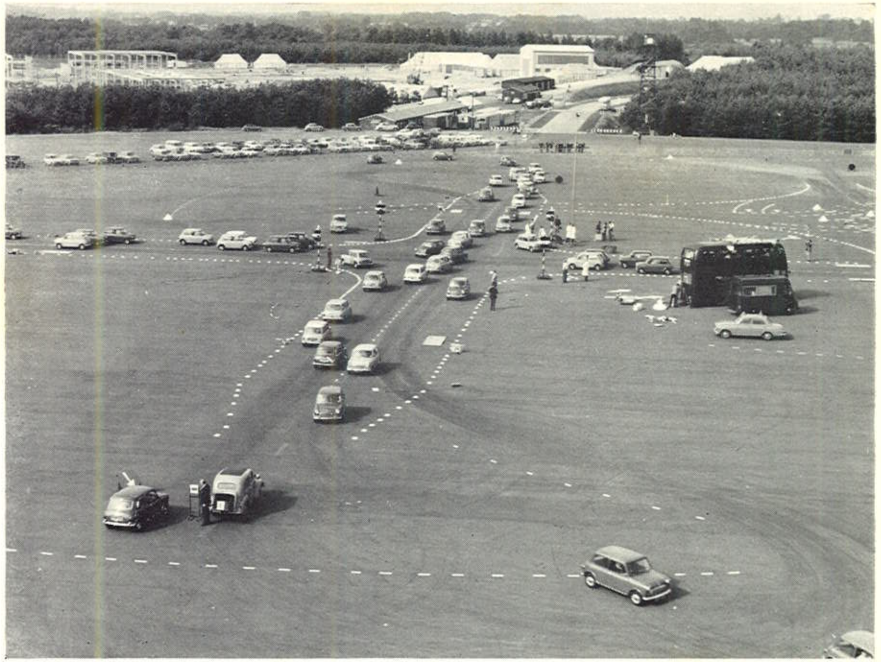


One of the trial intersections in a controlled experiment on traffic capacity

PLATE 3

A 60-ft wide approach used in a controlled experiment on traffic capacity

PLATE 4



Controlled experiment on the research track comparing signals with other forms of control

PLATE 5

A typical 'good' site

PLATE 6

A typical 'average' site

PLATE 7

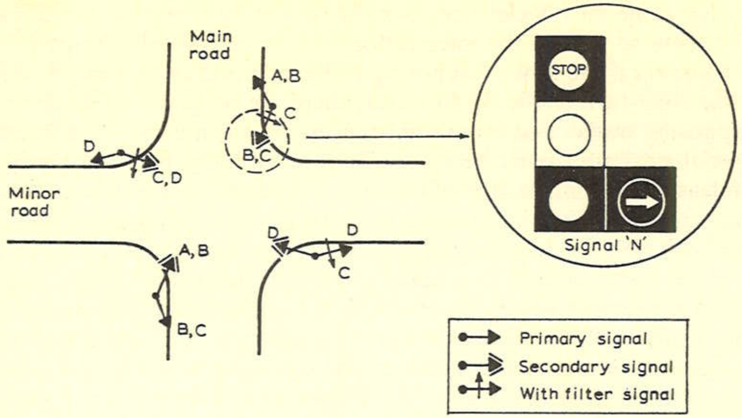
A typical 'poor' site

PLATE 8

facilities for some 60 vehicles per hour. More can be accommodated if there is room for them to wait in the intersection, and there may also be opportunities for right-turners if the flow of opposing traffic is less than that on the approach having the right-turn traffic. If, however, there are no gaps, or insufficient gaps in the opposing traffic, and more right-turners than can wait in the intersection, then special provision must be made. This will usually be by an early cut-off or late release, or a combination of the two. Each arrangement has its advantages and disadvantages. With late release it is not possible satisfactorily to vary the duration with vehicle detectors according to the number of turning vehicles. Furthermore, traffic behaviour is uncertain at the change to overlap—sometimes the turning movement which has established itself will continue; sometimes the opposing traffic will start immediately. It is difficult to overcome this by any special signal indications. If a green arrow signal were to be used in addition to the usual green signal during the late-release period there is a risk that drivers would not notice when the green arrow was extinguished. These disabilities lead to inefficiency in control and possible danger. On the other hand, the arrangement may be desirable where there is only a single lane on the approach with the heavy right turn (so that vehicles have a chance to turn both at the beginning and end of the green period and so lessen the risk of blocking the lane) or where the opposing arm is on a down gradient (so that vehicles turn only when opposing traffic on the down gradient is, and has been for some time, at rest).

The early cut-off period can be closely controlled with vehicle detectors and is generally preferable with a multi-lane approach. It has a particular advantage in that any complementary left-turning movement from the cross road can be cleared by filter signal at the same time and without such traffic needing to merge with any other traffic stream. The filter is automatically followed by a green signal. Figure 17 illustrates the phasing arrangement and includes also a short late release, so that the turning movement cannot establish itself at the expense of opposing traffic at the beginning of the 'both-way' running period. The early cut-off period can readily be indicated to turning traffic by displaying a filter arrow beside the green signal during the early cut-off period (see Fig. 17). Thus, where one turning movement is appreciably heavier than the other, an early cut-off to facilitate the heavier turn is likely to provide the most satisfactory control; this may be combined as in Fig. 17 with a late release to assist the lesser turning movement. Where, however, two opposing arms of an intersection both have substantial right-turning movements the situation is considerably more complicated, especially if such vehicles tend to turn on the offside of each other (i.e. go round each other) rather than to turn on their nearsides. Although junction layout tends to determine which movement will take place, the simpler nearside movement can be encouraged and made safer where the lanes containing turning vehicles can be directly opposite each other, as shown in Fig. 15. As was mentioned earlier (in 'Layout for right-turning vehicles'), the arrangement shown in Fig. 16 is preferable, where the opposing movements of turning traffic are separated completely from through movements. The phasing for this situation is given in Fig. 16. An adaptation of this for single carriageways is shown in Fig. 18: this layout requires the use of double signal heads so that drivers are fully advised of the permitted traffic movements.

A simple, but less efficient, means of dealing with opposing turning movements is to provide a short all-red period between phases during which vehicles held within the junction may clear. With all these solutions it may be necessary



PHASE	PHASE DIAGRAM	SIGNAL 'N' ASPECT
A (Late release)		Red
B (Overlap)		Green
C (Early cut-off)		Green and green arrow
D (Minor road)		Red

FIG. 17. Phasing arrangements for late release, early cut-off and filter

to limit the cycle time either to avoid undue interference by right-turning traffic to the through-traffic movement in the junction or to avoid locking of right-turners. In extreme cases it may be necessary to provide separate phases for opposing approaches of a junction.

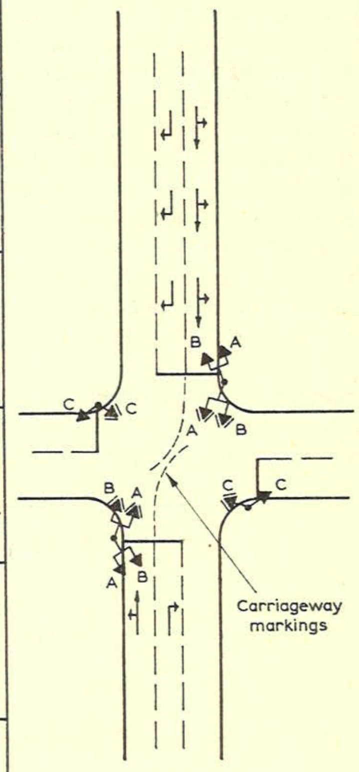
A 3- or 4-phase controller may be necessary at more complicated junctions with five or more roads, and at ordinary cross-roads where a pedestrian phase is required. Sometimes a staggered junction with a major road requires a 3-phase, 4-part signal installation where the major road is given two green periods in each cycle and the less important roads just one each.

Standard controllers give up to six phases, of which not more than four can be vehicle-actuated. An early cut-off or late release counts as a phase but an extra clearance does not. Phases can be switched in and out under time-switch

control so that where, for example, an early cut-off is needed to deal with turning traffic at peak periods, but not at other times, the controller will switch in this facility only at the required times. The possibility of prohibiting certain traffic movements to enable the number of phases to be reduced should be explored.

With vehicle-actuated signals it is usually arranged that if no demand for a particular phase is received then that phase is omitted from the cycle and delay to vehicles is consequently reduced.

Phase	A B Signals	Phasing	Layout
Intergreen		—	
A			
Intergreen		—	
B			
Intergreen		—	
C			



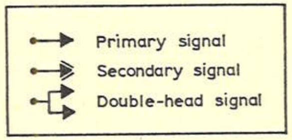


FIG. 18. Alternative to arrangement shown in Fig. 16 where space is limited

Filter signals

Filter signal lights mounted alongside the main signals are sometimes used to permit movement of vehicles in the direction shown by the green arrow even though the main signal is showing red. These signals can create problems for pedestrians, particularly those crossing the road from which the vehicle emerges. The vehicles in the filter stream also incur the risk of collision with vehicles in the traffic stream with which they merge. For these reasons it is desirable to restrict filtration against a red signal to sites where a substantial advantage in handling traffic is thereby achieved and pedestrian needs can satisfactorily be met despite filtration. Generally it is desirable also to limit filtration against a red signal to periods in the cycle when such traffic will not be required to merge with other traffic. Each situation should be judged on its merits to decide whether a filter light would be safe, and it may be desirable after installation to keep careful watch on the site for some time. If the 'filtered' traffic cannot be separated from the 'through' traffic by an island then it may be advantageous to set the stop line well back from the pedestrian studs (say 20 ft) so that drivers and pedestrians crossing the road get a better view of each other, and so avoid the risk that a pedestrian may cross a line of waiting vehicles and walk right into the path of vehicles moving in accordance with the filter signal. A better solution is to provide guard rails where possible so that pedestrians cannot cross the approach with the filter signal.

Because of a tendency for vehicles to continue filtering after a filter signal is extinguished and no other change in signal indications occurs, it is a requirement in Great Britain that filter signals shall always be followed either by a green signal (when the tendency to continue running is of no consequence) or by an amber signal. Usually control can be so arranged that filtration precedes a green signal, but where this is not possible double signal heads are provided so that the filter signal has its own amber and red. The full sequence with double signal heads is:

Left-hand signal	Right-hand signal
Red	Red
Red/Amber	Red/Amber
Green arrow	Green
Green arrow	Amber
Green arrow	Red
Amber	Red
Red	Red

An alternative sometimes preferred to a left filter is a slip-road which allows traffic to turn left continuously without coming under the control of the signals. A dotted white line across the exit of the slip-road (where it joins the cross road) is normally required to indicate that caution is required by drivers using the slip-road.

As mentioned earlier, filter signals are also used to indicate an early cut-off period. A right-hand filter signal lights up alongside the full green at the beginning of the early cut-off. This reduces delays by indicating when the right-of-way to the opposing traffic stream ceases.

Clearance periods

The intergreen period is arranged on the latest controllers to have a minimum of 4 seconds (on the older-type controllers concurrent ambers gave a minimum intergreen of 3 seconds). Intergreen periods may need to be extended from the minimum of 4 seconds to some suitable value in the following circumstances:

(a) to allow vehicles to clear the intersection when the distance across the junction is greater than normal (see below);

(b) to improve safety when the road carries fast traffic (where the special equipment described later is installed the extra clearance is needed only on a maximum change);

(c) on roads where there are appreciable numbers of right-turning vehicles (although an early cut-off or late release will usually be more satisfactory);

(d) to improve safety for pedestrians and to assist them in crossing the road at intersections where there is a high pedestrian flow but where provision of a separate pedestrian phase is not practicable.

With regard to (a), a rough guide to the extra clearance time required can be obtained from a consideration of the relative transit times to the probable collision points, it being assumed that vehicles enter the junction at a constant speed and that the probable collision points are at the intersections of the (imaginary) centre lines of the traffic lanes extended into the junction. (In practice, of course, there will be collision areas rather than collision points, since vehicles have width and length, and in addition can swerve; also drivers will tend to brake or accelerate to avoid a collision. To take account of all these factors in determining extra clearance time requirements would be impracticable; it is also unnecessary since the manoeuvrability of a vehicle tends to offset its size, and the calculation on the assumptions quoted have been found to give reasonable results.)

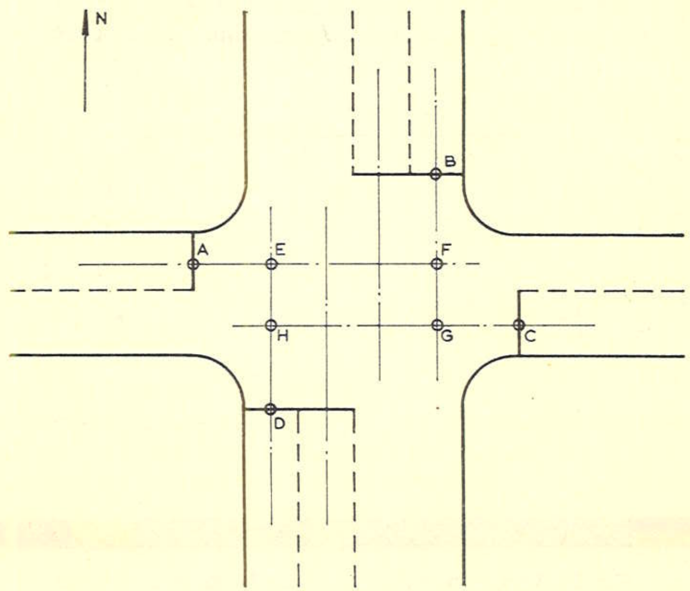


FIG. 19. Potential collision points

The probable collision points for a typical junction are shown in Fig. 19. Following the east-west phase the collision points of concern are F and H (since for G and E the clearing vehicles will arrive earlier and the starting vehicles will arrive later). G and E are the collision points of concern following the north-south phase. The suggested procedure for calculating clearance periods is as follows:

Measure the extra distance travelled to the probable collision points by vehicles losing right-of-way compared with those gaining right-of-way and call the larger one x ft. (For example if AF − BF = 30 ft and CH − DH = 35 ft then x=35 ft.) If x is less than 30 ft then the minimum intergreen period following the east-west phase should be satisfactory, but for every 30 ft or part thereof that x exceeds 30 ft add 1 second to the intergreen. Repeat for every possible phase change. If vehicle speeds on the clearing phase are substantially less than on the starting phase the calculated extra clearance time may need to be increased. If the junction is on a steep up-grade, or if there is a high proportion of slow-moving vehicles, 1 second should be added to the minimum intergreen time for every extra 20 ft of clearance distance, instead of 30 ft as given above.

CAPACITY

The amount of traffic that can pass through a signal-controlled intersection from a given approach depends on the green time available to the traffic and on the maximum flow of vehicles past the stop line during the green period.

Saturation flow

When the green period commences vehicles take some time to start and accelerate to normal running speed, but after a few seconds the queue discharges at a more or less constant rate called the saturation flow (see Fig. 20). The saturation flow is the flow which would be obtained if there was a continuous queue of vehicles and they were given a 100 per cent green time. It is generally expressed in vehicles per hour of green time. It can be seen from Fig. 20

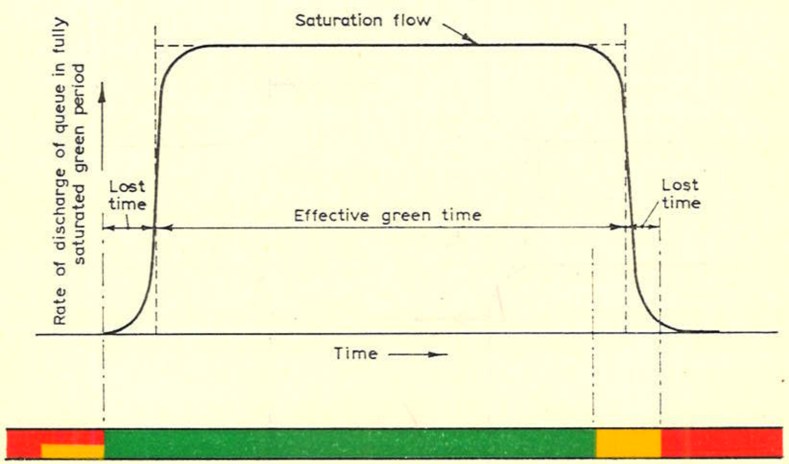


FIG 20. Variation with time of discharge rate of queue in a fully saturated green period

that the average rate of flow is lower during the first few seconds (whilst vehicles are accelerating to normal running speed) and during the amber period (as some vehicles decide to stop and others to carry on). It is convenient to replace the green and amber periods by an 'effective green' period, throughout which flow is assumed to take place at the saturation rate, and a 'lost' time during which no flow takes place. This is a useful concept because capacity is then directly proportional to effective green time. In graphical terms this means replacing the curve in Fig. 20 by a rectangle of equal area where the height of the rectangle is equal to the average saturation flow. The base of the rectangle is called the effective green time and the difference between this and the combined green and amber periods is lost time.

If G = combined green and amber periods (seconds)
g = effective green time (seconds)
c = cycle time (seconds)
l = lost time (seconds)
and s = saturation flow (vehicles per hour)

$$\text{capacity} = \frac{gs}{c} \text{ vehicles per hour} \quad \text{......................(4)}$$

$$\text{where } g = G - l \text{ seconds} \quad \text{............................(5)}$$

Saturation flow and lost time can be measured on the road directly, and a method for doing this is described in *Road Note* No. 34.[34]

Estimation of saturation flow

Though a direct measurement of saturation flow and lost time is obviously desirable in order to obtain reliable results this is not always practicable or indeed possible, e.g. when designing new intersections, and rules based on measurements of saturation flow carried out by the Laboratory at a large number of intersections can be used. If estimates of changes in saturation flow resulting from proposed changes in the numbers of right-turners, parked vehicles, or from a widening scheme are required, then the only method is to use the mean results obtained from a number of sites.

Saturation flow depends on the layout of the intersection (especially the width of the approach), the numbers of right-turning vehicles and goods vehicles, the presence of a parked vehicle and many minor factors. Investigations were carried out at about 100 signal-controlled junctions mainly in the London area but including some in other large cities.

To supplement the observations of actual traffic on the road and to extend the range of some of the variables, traffic experiments under 'controlled' conditions were carried out off the highway on a test track (see Plates 3 and 4) and at the Laboratory's research track (see Plate 5).

Effect of approach width. The saturation flow (s), expressed in terms of passenger car units* per hour with no turning traffic and with no parked vehicles present, is given by

$$s = 160w \text{ p.c.u./h} \quad \text{..............................(6)}$$

*Each type of vehicle is equivalent to a number of private cars in respect of its road-capacity requirements. This is called the 'passenger-car-unit' (p.c.u.) equivalent and may vary between different types of roads and intersections where w is the approach road width in feet (measured from kerb to inside of pedestrian refuge or centre line, whichever is the nearer, or to inside of central reserve in the case of a dual carriageway). This result is applicable to approach widths of from 18 ft to at least 60 ft (the maximum width tested). For widths between 10 ft and 17 ft the saturation flow shows a slight step effect (see Fig. 21):

$w =$	10	11	12	13	14	15	16	17	ft
$s =$	1850	1875	1900	1950	2075	2250	2475	2700	p.c.u./h

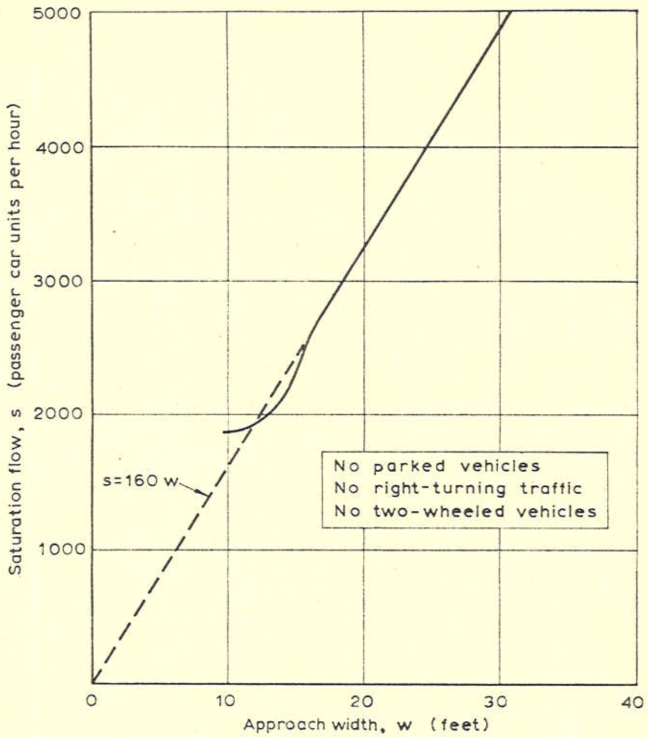


FIG. 21. Effect of approach width on saturation flow

The width is assumed to be constant for at least the length of the *approach* (defined as the length which will accommodate the queue which can just pass through the intersection during a fully saturated green period). If the approach is flared, the method given in Appendix 4 can be used to estimate saturation flow and effective green times. For approaches of non-constant width see *Road Note* No. 34[34] for effective approach widths.

Saturation flow was found to be lower than the values given above by about 6 per cent in the off-peak period; this may have been because drivers were then in less of a hurry. The rules given above are considered sufficiently accurate for most practical purposes for both peak and off-peak periods.

Effect of gradients. For each 1 per cent of uphill gradient the saturation flow was found to decrease by 3 per cent, and for each 1 per cent of downhill gradient the saturation flow increased by 3 per cent.[35] The gradient was defined as the

average slope between the stop line and a point on the approach 200 ft before it. The results were based on gradients not exceeding 10 per cent uphill and 5 per cent downhill and referred to sites where the slope continued through the intersection.

Effect of composition. The effect of different types of vehicle on the saturation flow at traffic signals is given by the following p.c.u. equivalents:

1 heavy or medium goods vehicle	$= 1\frac{3}{4}$ p.c.u.
1 bus	$= 2\frac{1}{4}$ p.c.u.
1 tram	$= 2\frac{1}{2}$ p.c.u.
1 light goods vehicle	$= 1$ p.c.u.
1 motorcycle, moped or scooter[36]	$= \frac{1}{3}$ p.c.u.
1 pedal cycle[36]	$= \frac{1}{5}$ p.c.u.

The p.c.u. equivalent of heavy vehicles was found to be the same on gradients (within the limits given in the previous section) as on level approaches.

Effect of right-turning traffic. If the right-turning movements from opposite directions cause the intersection to lock (see 'Layout for right-turning vehicles' and 'Phasing') then the capacity of the intersection cannot easily be assessed. In practice, locking should be prevented by using one of the methods described earlier. Under non-locking conditions the effects of right-turning traffic depend on whether or not conflicting traffic moves on the same phase and on whether or not the right-turning traffic is given exclusive lanes. There are four possibilities:

(i) *No opposing flow, no exclusive right-turning lanes.* An overall figure for saturation flow for the approach (irrespective of turning movements) can be obtained using the rules given above.

(ii) *No opposing flow, exclusive right-turning lanes.* The saturation flow of the right-turning stream should be obtained separately. It has been found [37] that the saturation flow (s) of a stream turning through a right angle depends on the radius of curvature (r) and is given by

$$s = \frac{1800}{1+5/r} \text{ p.c.u./h for single-file streams,* } \qquad \text{(7)}$$

$$\text{and } s = \frac{3000}{1+5/r} \text{ p.c.u./h for double-file streams,*} \qquad \text{(8)}$$

where r is measured in feet. (See part 1 of worked example No. 2 in Appendix 7.)

(iii) *Opposing flow, no exclusive right-turning lanes.* The effect of right-turners under these circumstances is three-fold. Firstly, because of the opposing traffic, they are delayed themselves and consequently delay other (non-right-turning) vehicles in the same stream; secondly, their presence tends to inhibit the use of the offside lane by straight-ahead

*For $r = \infty$, i.e. for the straight-ahead path, the values obtained from these expressions are 1800 and 3000 p.c.u./h for single- and double-file respectively. These compare closely with observed straight-ahead values of 1850 and 3200 p.c.u./h for 10 ft and 20 ft lanes respectively vehicles owing to the risk of being delayed and, thirdly, those right-turning vehicles which remain in the intersection at the end of the green period take a certain time to discharge and may delay the start of the cross-phase.

The first two of these effects can be allowed for by assuming that on the average each right-turning vehicle is equivalent to $1\frac{3}{4}$ straight-ahead vehicles.[38]

The third effect is more complicated. Right-turners may discharge through suitable gaps in the opposing stream. Observations indicate that a gap (α) of 5 or 6 seconds is typical. Figure 22 has been constructed from theoretical results[21] for two situations, (1) when the opposing flow is in a single lane and α is assumed to be 5 seconds and (2) when the opposing flow is in two or more lanes and α is assumed to be 6 seconds. The graph shows, for a given opposing flow, the effective

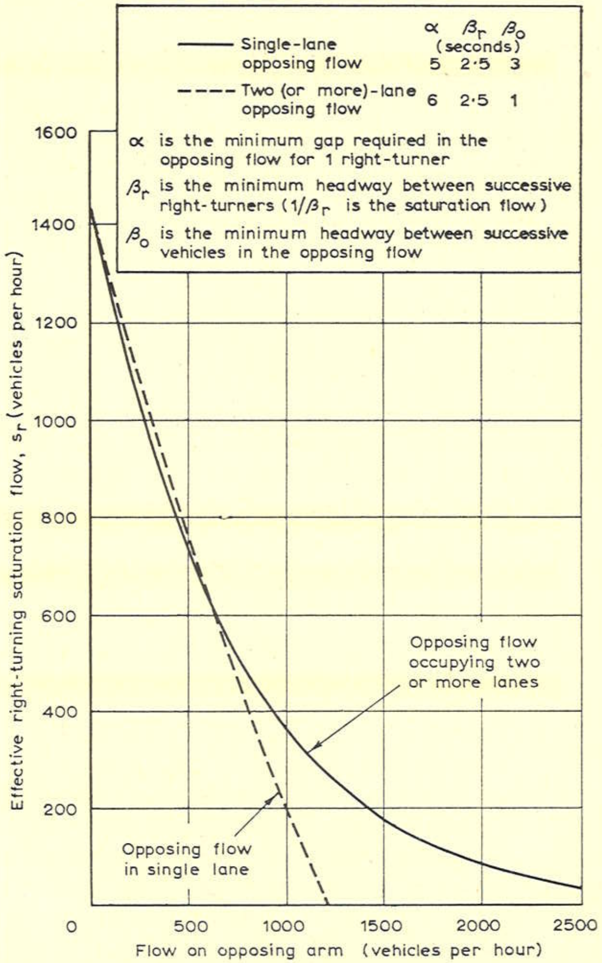


FIG. 22. Estimation of the effective right-turning saturation flow (s_r) for use with equation (9) (see also Appendix 5)

right-turning saturation flow, i.e. the maximum theoretical flow of right-turning vehicles (s_r) passing through gaps in the opposing flow, assuming this latter flow were running continuously. To convert s_r to the maximum number of right-turning vehicles per cycle (n_r) that can take advantage of gaps in the opposing stream, the following equation can be used (see Appendix 5 for derivation):

$$n_r = s_r\left(\frac{gs - qc}{s - q}\right) \qquad \text{.............................(9)}$$

where q and s are the flow and saturation flow values for the opposing arm and g and c are the green time and cycle time respectively. If g and c

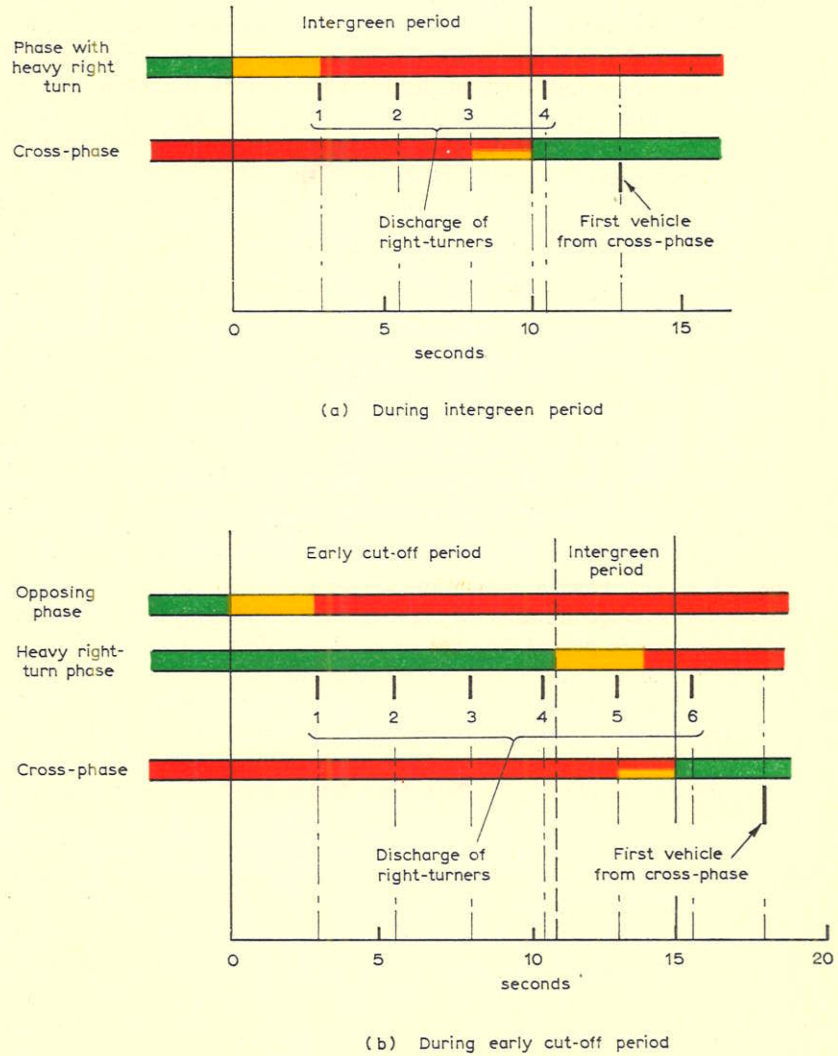


FIG. 23. Examples of times of discharge of right-turning vehicles during (a) intergreen period and (b) early cut-off period

are in seconds s_r should be expressed in vehicles per second before substitution in equation (9).

The difference between the average number of right-turners per cycle and n_r gives the average number waiting at the end of the green period (n_w). The rate of discharge of these vehicles is approximately one every $2\frac{1}{2}$ seconds. If it is assumed that the first of the waiting right-turners crosses a point on the centre line of the junction just as the signal turns to red (i.e. 3 seconds after the beginning of the intergreen period), then the second reaches this point $2\frac{1}{2}$ seconds later, the third 5 seconds later, and so on (see Fig. 23). For there to be no wastage of time between the right-turners clearing and the cross-traffic starting the first vehicle on the cross-phase should arrive at this point just $2\frac{1}{2}$ seconds after the last right-turner. If it assumed that the first vehicle on the cross-phase takes about 3 seconds from the start of its own green period to accelerate from rest and reach this point (i.e. arriving 3 seconds after the end of the intergreen period) then, neglecting variations in the number of right-turners waiting, there would be no wastage of time if the intergreen period were $2\frac{1}{2}\,n_w$ seconds. If I, the intergreen time, is less than $2\frac{1}{2}\,n_w$ the difference gives a rough estimate of the extra delay to the start of the cross-traffic. With *random* numbers of right-turners arriving per cycle the total delaying effect on the start of the cross-traffic will be greater than if the *average* number were assumed to arrive each cycle, but for most purposes it is probably unnecessary to take this into account.

In estimating whether a particular sequence of signal timings will give the required capacity it is necessary to make calculations of this type. It is also desirable to carry out this procedure when determining the length of a fixed early cut-off period (see (b) in Fig. 23), or, in certain cases. the length of an intergreen period, where an early cut-off period is not practicable. Figure 23 (b) shows that a fixed early cut-off period (as defined in the diagram) should be equal to $2\frac{1}{2}\,n_w$ less the length of the intergreen period following the early cut-off. A variable early cut-off period should have a maximum length sufficient to cater for at least twice the average number of right-turners remaining at the end of the both-way green period because of random effects. (For small values of n_w the maximum should be longer, e.g. for $n_w=1$, four times, and for $n_w=2$, three times the value $2\frac{1}{2}\,n_w$.) Even longer times may be justified at sites where right-turning vehicles which become trapped would be unduly obstructive.

Worked example No. 2 (Part 2) in Appendix 7 illustrates the method outlined above.

(iv) *Opposing flow, exclusive right-turning lanes.* There should be no delay to the straight-ahead traffic using the same approach as the right-turners, but there will be an effect on the cross-phase and this should be calculated as outlined in (iii) above.

Effect of left-turning traffic. The effect of left-turners on saturation flow depends on the sharpness of turn and on the pedestrian flow. The rules regarding the effect of curvature given in the previous section for right-turning traffic can

equally well be applied to well-defined left-turning streams. Where, however, left-turners in small numbers are intermixed with straight-ahead vehicles it is unnecessary to make a correction for them as the general saturation-flow relations given earlier include the effects of the left-turning traffic (forming about 10 per cent of the whole traffic) present when the studies were made. If left-turners form appreciably more than 10 per cent of the traffic a correction could be made for the excess over 10 per cent by assuming each left-turner is equivalent to $1\frac{1}{4}$ straight-ahead vehicles.[39]

Effect of pedestrians. The effect of the pedestrian flow has not been determined accurately and probably depends to a large extent on the particular conditions of the site. It is suggested that for average pedestrian flows no corrections are needed since pedestrians were present when the original studies were made, but for abnormally high pedestrian flows the effect should be taken into consideration when classifying the site according to the rules given below in 'Effect of site characteristics'.

Effect of a parked vehicle. It has been found that the reduction in saturation flow caused by the parked car nearest to the stop line on the particular approach is equivalent to a loss of carriageway width at the stop line, and can be expressed approximately as follows:

Effective loss of carriageway width

$$= 5{\cdot}5 - \frac{0{\cdot}9(z-25)}{k} \text{ ft (if positive)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

where z ($\geqslant$ 25 ft) is the clear distance of the nearest parked car from the stop line (ft)* and k is the green time (seconds). If the whole expression becomes negative the effective loss should be taken to be zero. The effective loss should be increased by 50 per cent for a parked lorry or wide van.

Effect of site characteristics. Many other factors affect the saturation flow, but to a much smaller extent than those already discussed. These factors have been grouped together to give an assessment of the type of site. Sites are classified as good, average, or poor according to the description given in Table 2 (see also Plates 6, 7 and 8).

For the three classes of site the percentages above and below the standard values of saturation flow given previously are shown in the table. Interpolation between these categories would seem reasonable. It may be useful to note that out of about 100 sites the lowest percentage of the standard saturation flow was 70 and the highest recorded was 135.

Worked example No. 3 in Appendix 7 illustrates the way the results in this section can be used to estimate saturation flow.

Lost time

Experiments in London have shown that, in the average signal cycle, lost time caused by starting delays and reduced flow during the amber period amounts to about 2 seconds per phase but is very variable, values from 0 to 7 seconds having been observed. The same average value has been found for intersections

*For z < 25 ft the distance should be taken as 25 ft

Table 2

Effect of site characteristics on saturation flow

Site designation	Description	Percentage of standard saturation flow
Good	Dual carriageway. No noticeable interference from pedestrians, parked vehicles, right-turning traffic (either owing to their absence or because special provision is made for them). Good visibility and adequate turning radii. Exit of adequate width and alignment. (See Plate 6.)	120*
Average	Average sites. Some characteristics of 'Good' and 'Poor'. (See Plate 7.)	100
Poor	Average speed low. Some interference from standing vehicles, pedestrians, right-turning traffic. Poor visibility and/or poor alignment of intersection. Busy shopping street. (See Plate 8.)	85†

* Highest value recorded 135 } out of 100 observation sites
† Lowest value recorded 70 }

with gradients (within the ranges previously specified). In addition to the lost time for a particular approach a certain amount of time in the cycle is lost owing to all-red periods. Figure 1 shows the relation between the intergreen period and lost time. Thus, if the intergreen period is I seconds and the starting delays plus unused amber time are l seconds of each combined green and amber period, then the lost time corresponding to each change of right-of-way is $(I-a)+l$ seconds where a is the amber period. Generally $l=2$ seconds, which, with a 3-second amber period, gives a lost time of $(I-1)$ seconds at each phase change.

Capacity of the whole intersection

The capacity of the whole intersection is dependent on the total amount of lost time (L) in the cycle. Lost time* is $\Sigma(I-a)+\Sigma l$ which becomes $\Sigma(I-1)$ seconds when $l=2$ seconds and $a=3$ seconds. The rest of the cycle is 'useful' time and is shared among the phases. If the signals are correctly set it will be seen later than the 'predominant' approaches of the phases (i.e. those with the highest ratios of flow to saturation flow) all reach capacity simultaneously. The other approaches will therefore have spare capacity. The ultimate capacity of the intersection can be defined as the maximum flow which can pass through the intersection with the same relative flows on the various approaches and with the existing proportions of turning traffic. Assuming that right-turners do not cause the saturation flow to fall after the first few seconds of green time, then the capacity will increase as the cycle time increases, since the ratio of lost time to useful time decreases (the effect becomes negligible when the cycle is very long).

* Σ (sigma) denotes the summation over all phases

In practice, it is usual to set an upper limit of 120 seconds for the cycle time, although in special cases this is increased. If the capacity were taken as the flow which could just be accommodated by such a cycle the delays would generally be excessively high. A practical capacity of 90 per cent of this maximum possible flow, which produces generally acceptable delays, is recommended.

In general, if the ratio of the flow to the saturation flow of the predominant arm of each phase is denoted by y, then the cycle time, c_m, which is just long enough to pass all the traffic, is given(1) by

$$c_m = \frac{L}{1-Y} \qquad \text{.................................(11)}$$

where Y is the sum of the y values over the phases and L is the total lost time $\{\Sigma(I-a)+\Sigma l\}$. The maximum value of Y which can be accommodated is therefore

$$Y = 1 - \frac{L}{c_m} \qquad \text{.................................(12)}$$

If, for practical purposes, c_m is taken as 120 seconds and $Y_{\text{pract.}}$ is 90 per cent of the maximum possible Y value then

$$Y_{\text{pract.}} = 0{\cdot}9 - 0{\cdot}0075\,L \qquad \text{..........................(13)}$$

where L is in seconds. The percentage reserve capacity is then given by

$$\frac{100(Y_{\text{pract.}} - Y)}{Y} \qquad \text{...............................(14)}$$

(See worked examples Nos. 4 and 7 (part 2) in Appendix 7.)

DELAY

It is estimated that in Great Britain delays at traffic signals amount to about 100 million vehicle-hours each year. If a saving of only a few per cent could be effected by using improved methods to set signals the financial saving would be considerable.

Although delays at signals have been studied on the road most of the Laboratory's work was done initially by simulating traffic behaviour at signals on a special purpose computer[40] and later on a general purpose computer (the pilot ACE which was at the time housed at the National Physical Laboratory).[1] In the simulation technique, artificially generated traffic of a random nature was used to feed the computer. It was assumed that once the signals turn green traffic discharges during the green period at a constant rate (the saturation flow) as long as the queue exists.

Some later theoretical work which takes account of bunching in the arrival pattern has been carried out by Miller.[41]

Delay with fixed-time signals

Computations of delay were carried out for a variety of flows, saturation flows and signal settings, and from the results a formula was deduced for the average delay on any single approach to an intersection controlled by fixed-time signals (or, of course, vehicle-actuated signals operating on a fixed cycle because of heavy traffic demands, i.e. during peak periods). The details of the simulation method are given in reference (1) and only the results are given here. It was found that

$$d=\frac{c(1-\lambda)^2}{2(1-\lambda x)}+\frac{x^2}{2q(1-x)}-0{\cdot}65\left(\frac{c}{q^2}\right)^{\frac{1}{3}}x^{(2+5\lambda)} \quad \text{.........(15)}$$

where d = average delay per vehicle on the particular arm

λ = proportion of the cycle which is effectively green for the phase under consideration (i.e. g/c)

x = the degree of saturation. This is the ratio of the flow to the maximum possible flow under the given settings of the signals and equals $q/\lambda s$

c, g, q and s are as defined earlier (see Glossary in Appendix 1 for definitions)

If c is in seconds, q should be in vehicles per second to give the delay in seconds.

The last term of the equation has a value in the range 5 to 15 per cent of d in most cases. A rough approximation to the delay may therefore be given by

$$d=\frac{9}{10}\left\{\frac{c(1-\lambda)^2}{2(1-\lambda x)}+\frac{x^2}{2q(1-x)}\right\} \quad \text{.......................(16)}$$

To enable the delay to be estimated more easily, equation (15) has been written as

$$d=cA+\frac{B}{q}-C \quad \text{...............................(17)}$$

where $A=\frac{(1-\lambda)^2}{2(1-\lambda x)}$, $B=\frac{x^2}{2(1-x)}$, and C is the third term. A and B have been tabulated (see Tables 3 and 4) and C has been calculated as a percentage of the first two terms of equation (17) and is given in Table 5 in terms of x, λ and M where M (=qc) is the average number of vehicles arriving per cycle. (See worked example No. 5 in Appendix 7.) A nomogram for the determination of delay based on equation (15) has been produced by Dick.(42)

The delay formula has been tested under actual road conditions at several fixed-time and vehicle-actuated intersections,(1) (43) and the variation between observed and calculated values was no greater than would be expected owing to random fluctuations (see Fig. 24).

The way the delay varies with traffic flow is illustrated in Fig. 25 in a typical case (g=30 seconds, c=60 seconds, s=1800 vehicles per hour).

It will be seen that when the flow reaches about 90 per cent of the ultimate capacity (900 vehicles per hour in this case) the delay rises steeply. Computations of delay were made for values of x up to 97·5 per cent of saturation. Theoretically, the delay increases to infinity as the flow tends to the ultimate capacity, but in practice the level of flow rarely remains at a high value for a long period; for example, the flow falls off at the end of the peak period, and the queues generally

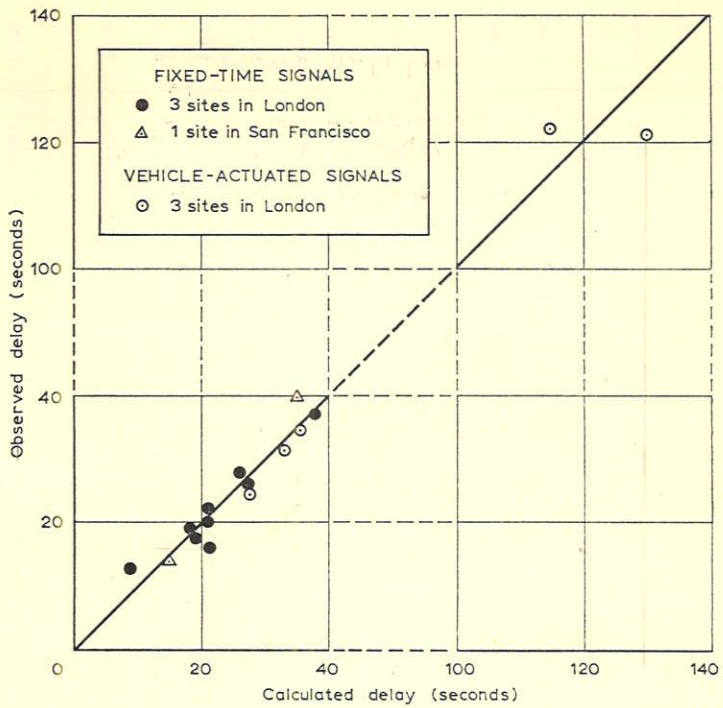


FIG. 24. Observed and calculated average delays at traffic signals

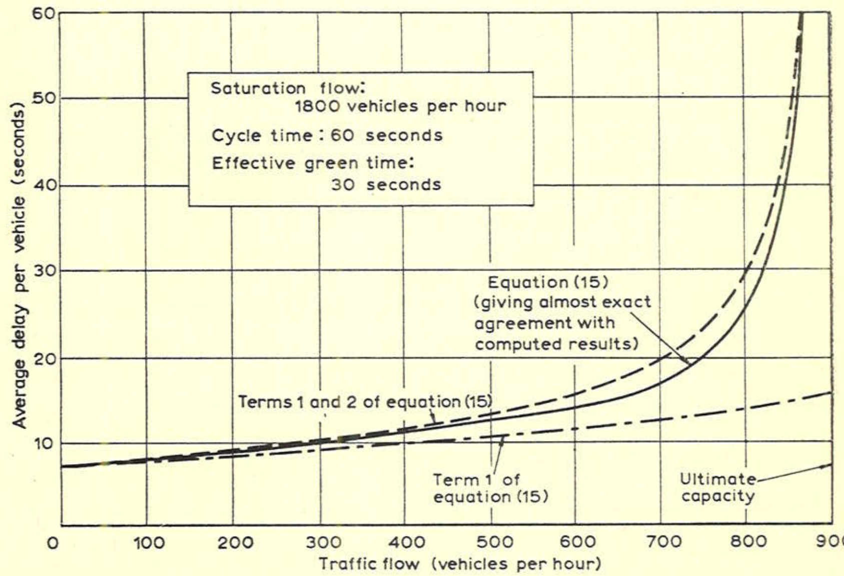


FIG. 25. Typical delay/flow curve obtained from fixed-time traffic signal computations

Table 3

Tabulation of $A = \dfrac{(1-\lambda)^2}{2(1-\lambda x)}$

x \ λ	0·1	0·2	0·3	0·35	0·40	0·45	0·50	0·55	0·60	0·65	0·70	0·80	0·90
0·1	0·409	0·327	0·253	0·219	0·188	0·158	0·132	0·107	0·085	0·066	0·048	0·022	0·005
0·2	0·413	0·333	0·261	0·227	0·196	0·166	0·139	0·114	0·091	0·070	0·052	0·024	0·006
0·3	0·418	0·340	0·269	0·236	0·205	0·175	0·147	0·121	0·098	0·076	0·057	0·026	0·007
0·4	0·422	0·348	0·278	0·246	0·214	0·184	0·156	0·130	0·105	0·083	0·063	0·029	0·008
0·5	0·426	0·356	0·288	0·256	0·225	0·195	0·167	0·140	0·114	0·091	0·069	0·033	0·009
0·55	0·429	0·360	0·293	0·262	0·231	0·201	0·172	0·145	0·119	0·095	0·073	0·036	0·010
0·60	0·431	0·364	0·299	0·267	0·237	0·207	0·179	0·151	0·125	0·100	0·078	0·038	0·011
0·65	0·433	0·368	0·304	0·273	0·243	0·214	0·185	0·158	0·131	0·106	0·083	0·042	0·012
0·70	0·435	0·372	0·310	0·280	0·250	0·221	0·192	0·165	0·138	0·112	0·088	0·045	0·014
0·75	0·438	0·376	0·316	0·286	0·257	0·228	0·200	0·172	0·145	0·120	0·095	0·050	0·015
0·80	0·440	0·381	0·322	0·293	0·265	0·236	0·208	0·181	0·154	0·128	0·102	0·056	0·018
0·85	0·443	0·386	0·329	0·301	0·273	0·245	0·217	0·190	0·163	0·137	0·111	0·063	0·021
0·90	0·445	0·390	0·336	0·308	0·281	0·254	0·227	0·200	0·174	0·148	0·122	0·071	0·026
0·92	0·446	0·392	0·338	0·312	0·285	0·258	0·231	0·205	0·179	0·152	0·127	0·076	0·029
0·94	0·447	0·394	0·341	0·315	0·288	0·262	0·236	0·210	0·183	0·157	0·132	0·081	0·032
0·96	0·448	0·396	0·344	0·318	0·292	0·266	0·240	0·215	0·189	0·163	0·137	0·086	0·037
0·98	0·449	0·398	0·347	0·322	0·296	0·271	0·245	0·220	0·194	0·169	0·143	0·093	0·042

Table 4

Tabulation of $B = \dfrac{x^2}{2(1-x)}$

x	0·00	0·01	0·02	0·03	0·04	0·05	0·06	0·07	0·08	0·09
0·1	0·006	0·007	0·008	0·010	0·011	0·013	0·015	0·017	0·020	0·022
0·2	0·025	0·028	0·031	0·034	0·033	0·042	0·046	0·050	0·054	0·059
0·3	0·064	0·070	0·075	0·081	0·088	0·094	0·101	0·109	0·116	0·125
0·4	0·133	0·142	0·152	0·162	0·173	0·184	0·196	0·208	0·222	0·235
0·5	0·250	0·265	0·282	0·299	0·317	0·336	0·356	0·378	0·400	0·425
0·6	0·450	0·477	0·506	0·536	0·569	0·604	0·641	0·680	0·723	0·768
0·7	0·817	0·869	0·926	0·987	1·05	1·13	1·20	1·29	1·38	1·49
0·8	1·60	1·73	1·87	2·03	2·21	2·41	2·64	2·91	3·23	3·60
0·9	4·05	4·60	5·28	6·18	7·36	9·03	11·5	15·7	24·0	49·0

have insufficient time to build back to the lengths required to give excessively long delays. Furthermore, drivers, on seeing the end of the queue a long distance away from the intersection, may well turn off and find some alternative route under these conditions and this has, in fact, been observed at a particular junction in central London which was running to capacity for a large proportion of the day.(44) The average delay at this junction was several minutes.

The *average* value of the delay is usually the one which is quoted and used in calculations, but it is often useful to know the variability of delays. Fluctuations in delay are primarily due to random fluctuations in the numbers of vehicles arriving at an intersection. Results obtained from observations of delays at several sites in London have shown the standard deviation to be about three-quarters of the mean delay; computations carried out using simulation methods with random traffic gave a standard deviation greater than this.

Delay with vehicle-actuated signals

When the green periods are running to maximum because of heavy traffic demands (i.e. during peak periods) the average delay per vehicle may be estimated from the fixed-time signal delay formula (equation (15) above). This has been tested in practice at several vehicle-actuated intersections in the London area and good agreement was obtained between observed and theoretical values (see Fig. 24).

When the green periods are not running to maximum it is much more difficult to derive a simple method for predicting the delay to vehicles. However, a few computations of delay using the computer mentioned earlier have shown that the delay obtained with a vehicle-extension period of about 4 seconds (i.e. the minimum practical value for older controllers employing fixed-time extensions) is roughly the same as the value of delay obtained from the fixed-time formula

Table 5

Correction term of equation (17) *as a percentage of the first two terms*

x	λ \ M	2·5	5	10	20	40
0·3	0·2	2	2	1	1	0
	0·4	2	1	1	0	0
	0·6	0	0	0	0	0
	0·8	0	0	0	0	0
0·4	0·2	6	4	3	2	1
	0·4	3	2	2	1	1
	0·6	2	2	1	1	0
	0·8	2	1	1	1	1
0·5	0·2	10	7	5	3	2
	0·4	6	5	4	2	1
	0·6	6	4	3	2	2
	0·8	3	4	3	3	2
0·6	0·2	14	11	8	5	3
	0·4	11	9	7	4	3
	0·6	9	8	6	5	3
	0·8	7	8	8	7	5
0·7	0·2	18	14	11	7	5
	0·4	15	13	10	7	5
	0·6	13	12	10	8	6
	0·8	11	12	13	12	10
0·8	0·2	18	17	13	10	7
	0·4	16	15	13	10	8
	0·6	15	15	14	12	9
	0·8	14	15	17	17	15
0·9	0·2	13	14	13	11	8
	0·4	12	13	13	11	9
	0·6	12	13	14	14	12
	0·8	13	13	16	17	17
0·95	0·2	8	9	9	9	8
	0·4	7	9	9	10	9
	0·6	7	9	10	11	10
	0·8	7	9	10	12	13
0·975	0·2	8	9	10	9	8
	0·4	8	9	10	10	9
	0·6	8	9	11	12	11
	0·8	8	10	12	13	14

by substituting in this formula calculated values of optimum cycle time and green time (not necessarily the values observed in practice). The results given later in 'COMPARISON OF FIXED-TIME AND VEHICLE-ACTUATED OPERATION' confirm this finding, and in 'OPTIMUM SETTINGS: FIXED-TIME SIGNALS' it is shown how the optimum settings are obtained. It is recommended that the following method be used:

(1) Calculate the optimum cycle time and optimum green times for the flows at the particular times of day. (See 'OPTIMUM SETTINGS: FIXED-TIME SIGNALS'.)

(2) Even if these values are outside the 'practical' range, substitute them in the formula for delay (equation (15)) to find the average delay per vehicle. Alternatively, the tables may be used. The result obtained is the delay which would be expected with an efficient vehicle-actuated installation (i.e. one with speed timing or with a low fixed-extension period).

(3) With a fixed vehicle-extension period of 10 seconds, fixed-time operation would virtually be obtained with all except light flows, whilst for a fixed vehicle-extension period of 6 or 7 seconds the delay would probably be about halfway between the calculation based on optimum timings (steps (1) and (2) above) and on maximum periods. (See worked example No. 8 in Appendix 7.)

Effect of a parked vehicle on delay

A vehicle which parks on the approach to a signal-controlled intersection causes the saturation flow to be reduced (see 'Estimation of saturation flow').

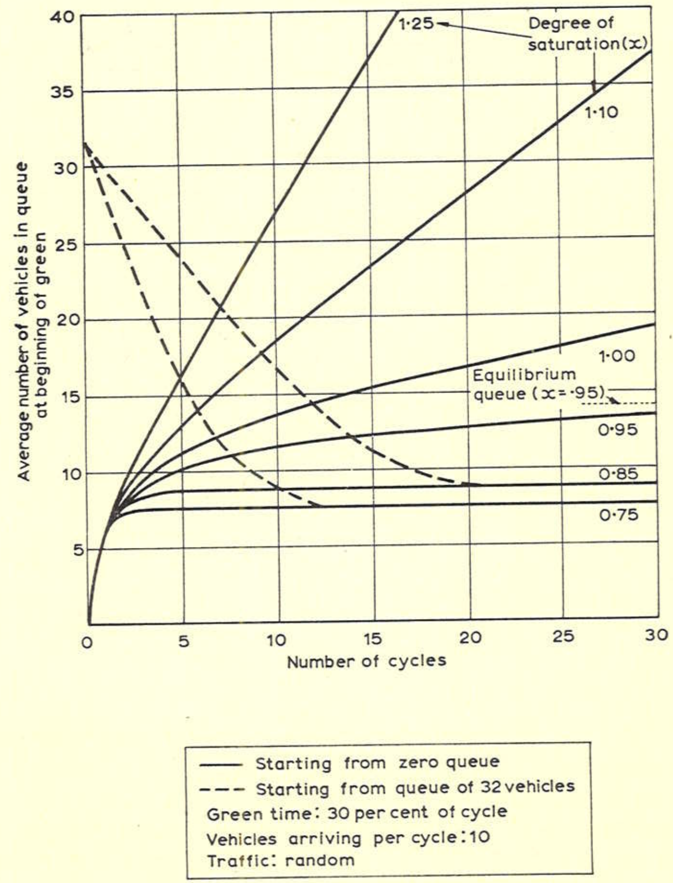


FIG. 26. Mean queue lengths as equilibrium is approached

This causes a sudden change in the degree of saturation, i.e. in the ratio of flow to maximum possible flow. Similarly, when the vehicle leaves there is again a sudden change in the degree of saturation. In the research into delays outlined above it was assumed that conditions were in a steady state, i.e. that the flow was varying randomly about a constant mean. Simulations have, however, been carried out to investigate variations in the queue, and hence in the delay, when there is a sudden change in the degree of saturation, and some of the results are shown in Fig. 26. It can be seen that, even with a zero queue initially, equilibrium is reached within a few cycles for degrees of saturation less than 0·9, but for degrees of saturation between 0·9 and 1·0 the queue takes a long time to settle down (theoretically an infinite time for $x=1$). Above $x=1$ the queue increases rapidly at first and then continues to increase more slowly; for $x>1{\cdot}1$ the queue increases more or less linearly with time.

Thus, it is approximately correct, and sufficient for practical purposes, to say that equilibrium is attained immediately following an increase in the degree of saturation which results in a final degree of saturation of less than 0·9, and it can be shown[45] that in these situations the total extra delay to other vehicles owing to the presence of a vehicle parked for a time T is

$$\frac{c\lambda q(1-\lambda)^2\,(X-x)}{2(1-\lambda x)\,(1-\lambda X)}T+\frac{(X-x)\,(X+x-Xx)}{2(1-x)\,(1-X)}T \quad \text{...........(18)}$$

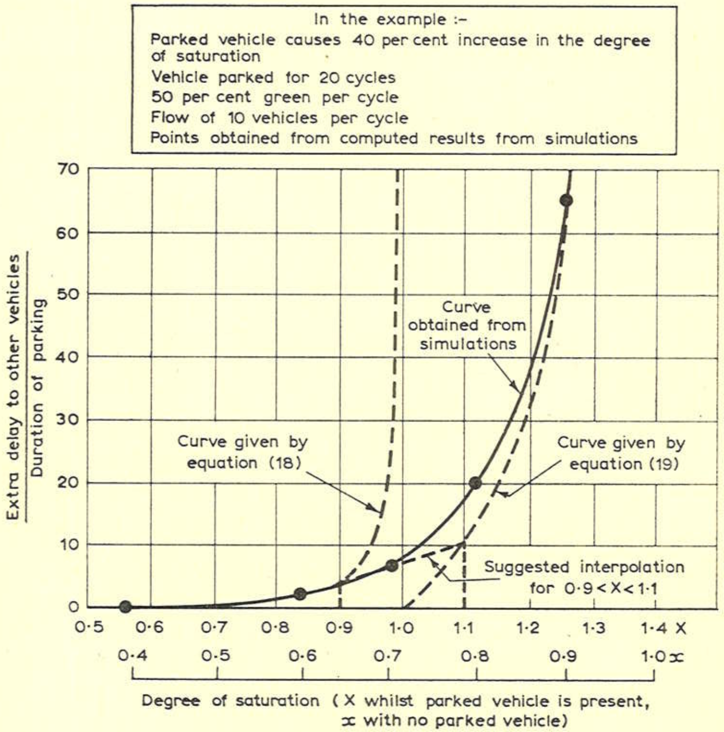


FIG. 27. Example of extra delay to other vehicles caused by a parked vehicle

where X and x are the degrees of saturation respectively with and without the parked vehicle present, and λ is the ratio g/c. If c is in seconds, q should be in vehicles per second.

It is also approximately correct to say that for $X > 1{\cdot}1$ the queue increases linearly with time and on this assumption the extra delay experienced by vehicles passing through the intersection can be shown to be[45]

$$\frac{q(X-1)\,(X-x)\,T^2}{2X^2(1-x)} \quad \text{.............................(19)}$$

For $0{\cdot}9 < X < 1{\cdot}1$ the extra delay has not been determined mathematically. Simulations to determine the extra delay have been made in a number of cases and the solid curve in Fig. 27 shows some typical results. The dotted curves are derived from the two expressions given above. If the two dotted curves are joined by a straight line between $X=0{\cdot}9$ and $X=1{\cdot}1$ a very rough approximation to the true curve, as given by the simulated results, is obtained.

Some results of calculations based on these formulae are given in Table 6. It can be seen from the table that the total extra minutes of delay caused by a vehicle parking close to a signal-controlled intersection can be very large, and is sometimes out of all proportion to the gain to the driver or owner of the vehicle parking; for example, on a 20-ft approach with a vehicle parked for 30 minutes 75 ft from the stop line when the flow is 1200 p.c.u./h the aggregate extra delay to the other vehicles is 1000 minutes.

An expression has also been derived for the duration of the effect of the parked vehicle, i.e. the time the vehicle is parked plus the time required to disperse the queue, for those situations where the parked vehicle raises the degree of saturation to greater than unity. The duration of the effect is given by

$$\frac{(q-\lambda S)\,T}{\lambda s-q} \quad \text{..................................(20)}$$

where S and s are the saturation flows with and without the parked vehicle present.

The maximum individual delay is imposed on the vehicle which arrives just as the parked vehicle is about to leave (the queue is at its maximum at this time). The extra delay over and above that which this vehicle would have had in the absence of the parked vehicle is given by

$$x\left(1-\frac{1}{X}\right)T \quad \text{................................(21)}$$

for cases where $X > 1$. (See worked example No. 6 in Appendix 7.)

Table 6

Total extra delay (minutes) to other vehicles due to a parked vehicle

(cycle time 1 minute; proportion of green time per cycle 50 per cent)

Approach width (feet)	Distance of parked vehicle from stop line (feet)	Duration of parking (minutes)	Flow (passenger car units per hour)			
			800	1000	1200	1400
16	25	10	55*	290		
		30	310	2600		
	75	10	50*	240	†	†
		30	290	2200		
	150	10	16	150*		
		30	47	1250*		
20	25	10	21	80*	358	3400
		30	62	650*	3320	31 000
	75	10	8	33	125*	1500
		30	25	99	1000*	14 000
	150	10	2	6	27	170
		30	6	17	81	1500
24	25	10	5	11	38	125*
		30	14	33	114	1100*
	75	10	3	6	17	100*
		30	9	19	51	700*
	150	10	1	2	5	15
		30	3	7	15	44

* Estimated by interpolation

† These flows were not attained with the available green time, even in the absence of a parked vehicle

Cycle time

In deducing an expression for the cycle time which gives the least delay to all traffic, it has been found sufficiently accurate to select one arm only from each phase to represent that phase. Provided (as is usually the case) the lost times are more or less the same for different arms of the same phase, the arm with the highest ratio of flow to saturation flow is selected as the predominant arm and this ratio is denoted by the symbol y. Cases where the lost times are different for approaches belonging to the same phase are considered later.

By differentiating the equation for the overall delay at an intersection with respect to the cycle time it was found[1] that the cycle time with the minimum delay could be represented by

$$c_o = \frac{1{\cdot}5L+5}{1-y_1-y_2-\ldots.y_n} = \frac{1{\cdot}5L+5}{1-Y} \text{ seconds} \qquad \ldots\ldots\ldots\ldots(22)$$

where $y_1, y_2 \ldots. y_n$ are the maximum ratios of flow to saturation flow* for phases 1, 2 .... n, $Y = \Sigma y$ and L is the total lost time per cycle (in seconds). This cycle time will be referred to as the 'optimum cycle time'.

Under light traffic conditions the optimum cycle time as deduced from this formula may be very short. From a practical point of view, including safety considerations, it may be desirable to regard a cycle time of about 25 seconds as the lower limit. It may also be desirable to regard a cycle time of about 2 minutes as the upper limit, since the gain in capacity with very long cycles is often insignificant. This limit may be exceeded only in exceptional circumstances (e.g. at multi-phase junctions or under conditions where at times the primary function of the signals is to facilitate traffic movement on one route at the expense of another, for example at weekends on holiday routes).

Some examples of the variation of delay with cycle time are shown in Fig. 28. It has been found that for cycle times within the range three-quarters to one-and-a-half times the optimum value the delay is never more than 10 to 20 per cent above that given by the optimum cycle. For most practical purposes this result may be used in deducing a compromise cycle time when the level of flow varies considerably throughout the day. It would, of course, be better either to change the cycle time to take account of this or, as is more common, to use vehicle-actuated signals. However, if it is desired to use a single setting of fixed-time signals, the simple approximate method outlined below may be used.

(1) Calculate the optimum cycle time for each hour of the day when the traffic flow is medium or heavy, e.g. between the hours of 7 a.m. and 7 p.m., and average the results over the day.

(2) Evaluate three-quarters of the optimum cycle time calculated for the heaviest peak hour.

(3) Select whichever is the greater as the cycle time.

*When using this formula for setting signals, measured, rather than estimated, saturation flows should be used wherever possible. This is because small changes in the assumed saturation flow result in relatively large changes in calculated cycle time (for minimum delay) as capacity is approached

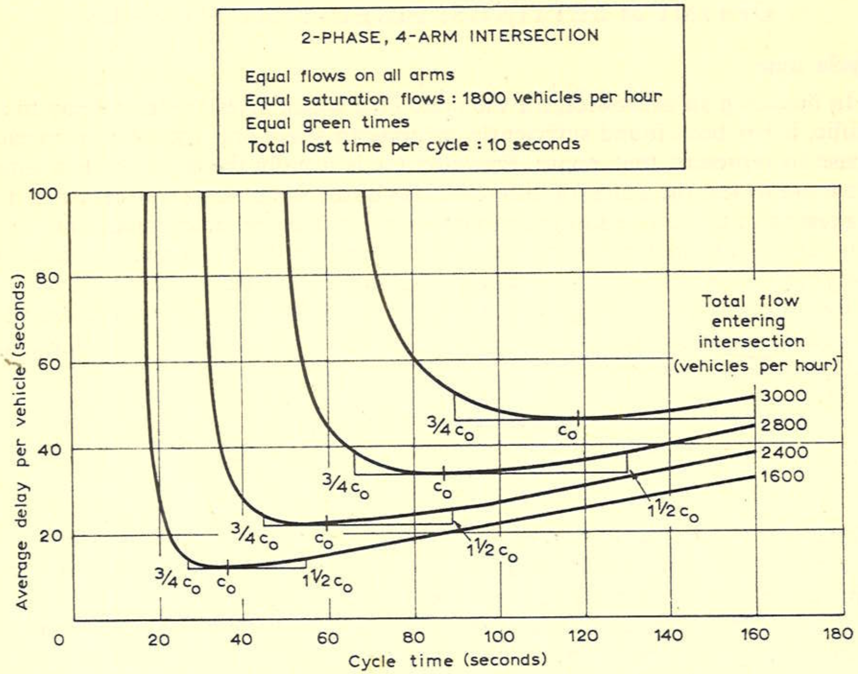


FIG. 28. Effect on delay of variation of cycle length

Green times

A simple rule for setting the green times to give the least overall delay to all traffic using the intersection was derived from the delay equation.[(1)] It was found that the ratio of the effective green times should equal the ratio of the y values, i.e.

$$\frac{g_1}{g_2}=\frac{y_1}{y_2} \qquad \text{.....................................(23)}$$

where g_1 and g_2 are the effective green times of phases 1 and 2 respectively. This rule can be extended to 3- or more phase operation. Strictly, the green-time ratio should be made a few per cent nearer to unity if the y-value ratio differs appreciably from unity, but for most practical purposes the effect is too small to be of significance (see page 11 of reference (1)). Also, it has been shown that where the two arms of a single phase have different values of the ratio q/s, approximately minimum overall delay is still obtained by dividing the cycle according to the y values as given above, even though the q/s ratio of the other arm(s) of the phase may vary between zero and y.

If $c_o - L$ is the total effective green time in the cycle the above rule gives

$$\left.\begin{aligned} g_1 &= \frac{y_1}{Y}(c_o - L) \\ g_2 &= \frac{y_2}{Y}(c_o - L) \end{aligned}\right\} \qquad \text{................................(24)}$$

etc.

The procedure when dealing with early cut-off periods, etc., is illustrated in 'Treatment for special cases'.

The way in which the delay varies with the ratio of the green periods is shown in Fig. 29 for four different cycle times. If the delay should be kept within the limits of 10 to 20 per cent above the minimum possible, the green times (assuming the cycle time is the optimum value) should not differ from the optimum values by more than one-fifth of the total lost time per cycle. This means that the proportional latitude for green-time setting is very much less than that for the

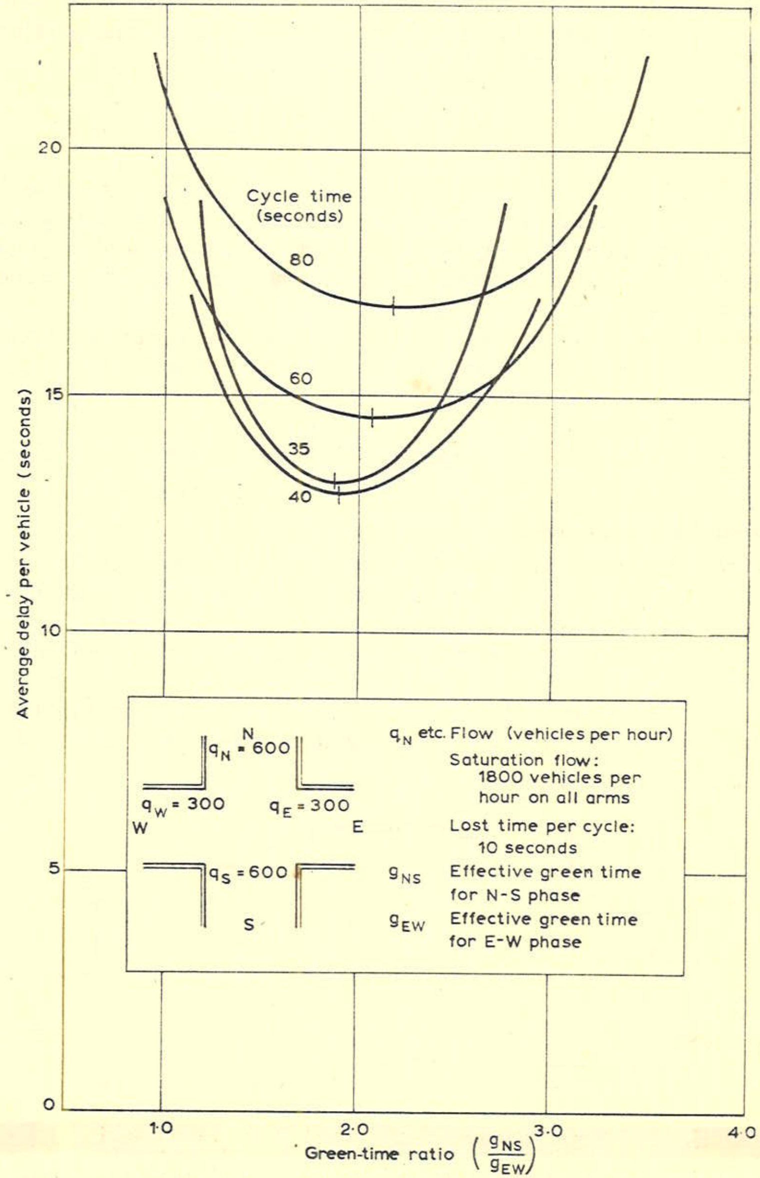


FIG. 29. Variation of delay with ratio of the green times

cycle-time setting. If the cycle time is shorter than the optimum value the latitude is reduced even more, but if it is longer than the optimum value the latitude is increased, e.g. in the example illustrated in Fig. 29 the change in g_{NS} or g_{EW} necessary to shift from one asymptote to the other is 10 seconds when $c=40$ seconds (optimum), but when $c=80$ seconds the corresponding change is 30 seconds. Thus it is better to have a cycle time longer than optimum if there is the likelihood of appreciable error in setting the green-time ratio.

Where the level of traffic flow is varying throughout the day and a single value of green time is required for each phase, it is suggested, in view of the findings given above regarding latitude, that the total effective green time in the cycle ($c_o - L$) should be divided in proportion to the average y values (for each phase) for peak periods only, i.e.

$$\frac{g_1}{g_2} = \frac{(y_1)_{\text{peak}}}{(y_2)_{\text{peak}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(25)$$

where $(y_1)_{\text{peak}}$ is the average y value during peak periods for phase 1, and $(y_2)_{\text{peak}}$ that for phase 2. If these values are likely to be appreciably different from the optimum ones for any peak period the cycle time should be increased to a value greater than that which would otherwise be obtained from the rules given under 'Cycle time' above.

Just as it was necessary to set limits on the cycle time for practical purposes, so it is necessary to avoid having green times too short and this is catered for automatically with existing controllers which have a minimum green period of 7 seconds.

Worked example No. 7 in Appendix 7 illustrates the method of calculating the optimum settings of fixed-time signals and the reserve capacity.

Treatment for special cases

Saturation flow which falls off during the green period. It should be noted that constant saturation flow has been assumed in the derivation of the cycle-time formula. If the saturation flow falls off throughout the green period (e.g. owing to heavy right-turning movements or to a flared approach), this formula does not hold.

If, however, the shape of the saturation-flow histogram is similar to that shown by solid lines in Fig. 30 and can reasonably be replaced by the quadrilateral

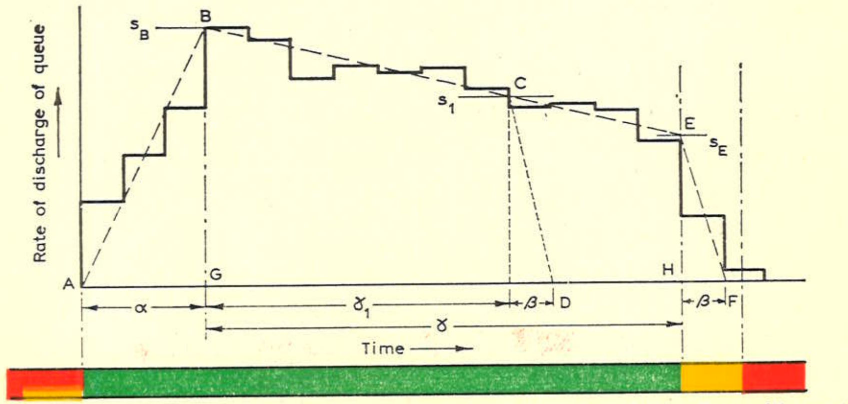


FIG. 30. Histogram of saturation flow where rate of discharge falls off steadily with increasing green time

shown by the dashed lines and having the same area, then the procedure described in Appendix 6 and illustrated in worked example No. 9 in Appendix 7 can be used to determine the optimum cycle time and green times.

In some cases the saturation flow falls off in a fairly definite step (see Fig. 31) after several seconds of green time to a lower and constant value, e.g. when right-turners block one lane. In this case there are two possible saturation flow values and two lost times depending on when the green time ends. Usually the drop in saturation flow takes place in the first 10 seconds of green time and it is unlikely that a practical optimum green time would end prior to this; thus only one combination of saturation flow and lost-time values need be considered, and this combination is shown in Fig. 31. If the step occurs later in the green period it is unlikely that, on averaging the results for a number of cycles, the step would remain sharp; it is far more likely that the mean saturation flow curve would approximate to the type given in Fig. 30.

Lost times and y *values different for arms belonging to the same phase.* Since it is not obvious which arm of a particular phase is the predominant one when the lost times and y values are different, each arm in turn should be taken as the predominant one and the optimum cycle time calculated using the appropriate combination of y and l values. The predominant arm is the one which requires the longer cycle time and it is this cycle time which should be selected as the optimum one.

Early cut-off or late-release features incorporated in the cycle. The phase which benefits from the early cut-off can be said to consist of two stages, (1) flows A and B together (see Fig. 32) and (2) flows A and R together. Considering arm A, the q/s value for flow A should be based on a saturation flow for the straight-ahead flow only, or if the latter quantity has to be estimated it should be made appropriate to the width of the approach less the width of the right-turning lane or lanes. If no lanes are marked and the right-turning traffic flows in single file, 9 ft of width could be allowed for it. The q/s value for the right-turning traffic should preferably be based on a measured saturation flow. In the absence of this a saturation flow of 1600 p.c.u./h for one lane and 2700 p.c.u./h for two lanes turning right may be assumed if the turn is shallow. For sharp turns the rule given in 'Effect of right-turning traffic' should be used.

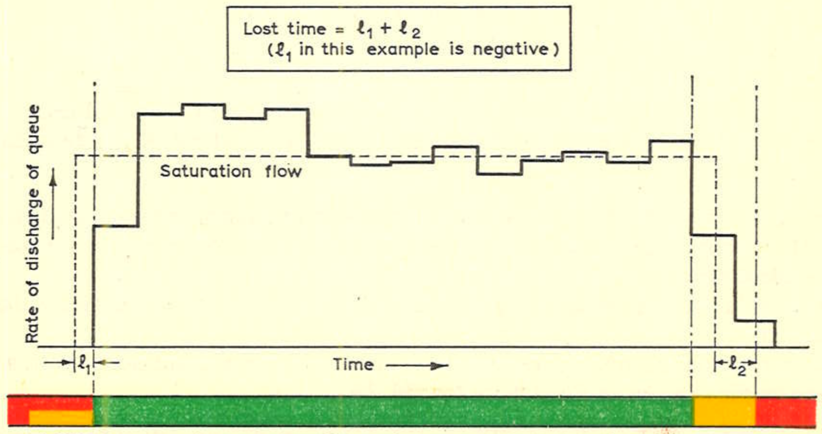


FIG. 31. Histogram of saturation flow where rate of discharge falls off sharply during the green time

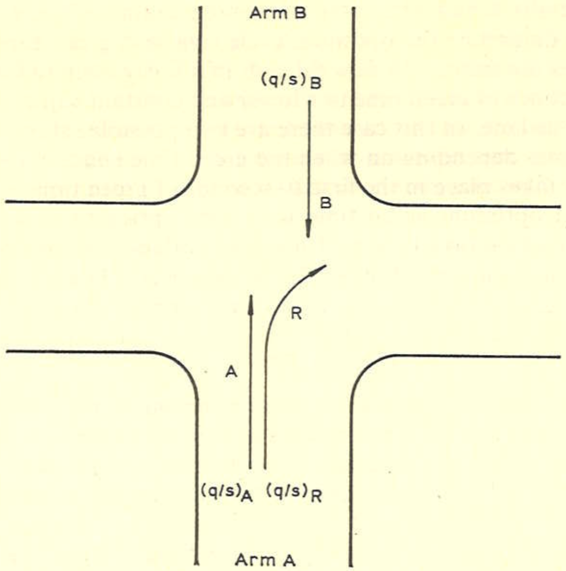


FIG. 32. Designation of flows and q/s values when early cut-off used

The y value for the two stages taken together will be $(q/s)_B+(q/s)_R$ or $(q/s)_A$ whichever is the greater. If $(q/s)_B+(q/s)_R>(q/s)_A$, then the y value for the first stage will be $(q/s)_B$ and that for the second stage $(q/s)_R$. If, however, $(q/s)_A$ is the greater, then the green time which flow A requires should be divided in proportion to the q/s values of streams B and R. Thus, the y value for the first stage would be

$$\frac{(q/s)_B\,(q/s)_A}{(q/s)_B+(q/s)_R}$$

and that for the second stage

$$\frac{(q/s)_R\,(q/s)_A}{(q/s)_B+(q/s)_R}$$

The normal rules for signal settings can then be used with these values.

The procedure is similar to that given above when there is a late-release feature.

Separate right-turning lanes. When the right-turning lanes are separated from the other traffic, calculations of the q/s values should be made for each independent stream.

The y value for the phase as a whole will be the larger of the two values calculated.

Further guidance on saturation flow under different conditions of right-turning traffic is given in *Road Note* No. 34.[34]

Separate left-turning lanes. When there are exclusive left-turning lanes separate calculations of the q/s values should be made for the left-turning streams as well as for the other traffic, and the larger q/s value taken to represent the phase as a whole.

If there is a slip-road so that left-turning traffic can flow at any time, without coming under the control of the signals, the y value for the phase should relate to the flow and saturation flow of the other traffic, i.e. that excluding left-turners. Some guidance on saturation-flow measurements under these conditions is given in *Road Note* No. 34.[34]

When there is a left filter and there are exclusive left-turning lanes the procedure to be adopted is similar to, though a little more complicated than, that given above for use when there is an early cut-off feature. Suppose the left filter coincides with the green period for arm C (see Fig. 33), then there are two

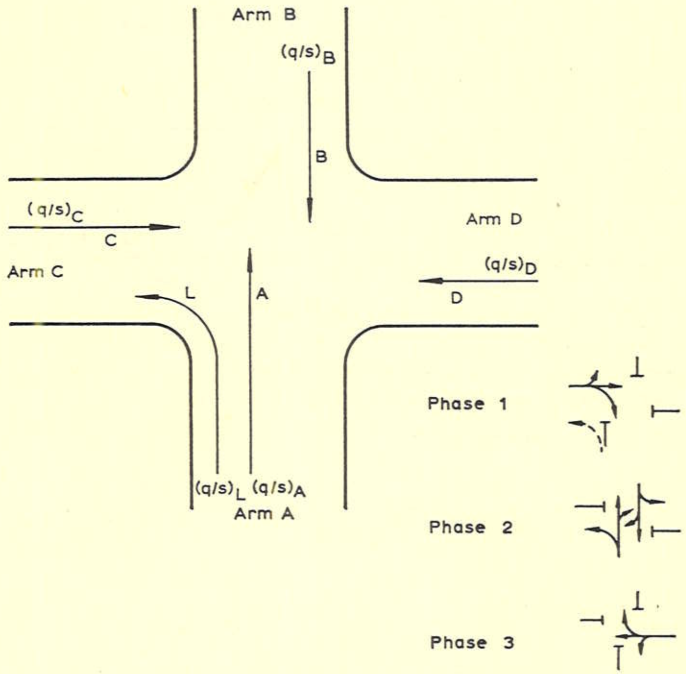
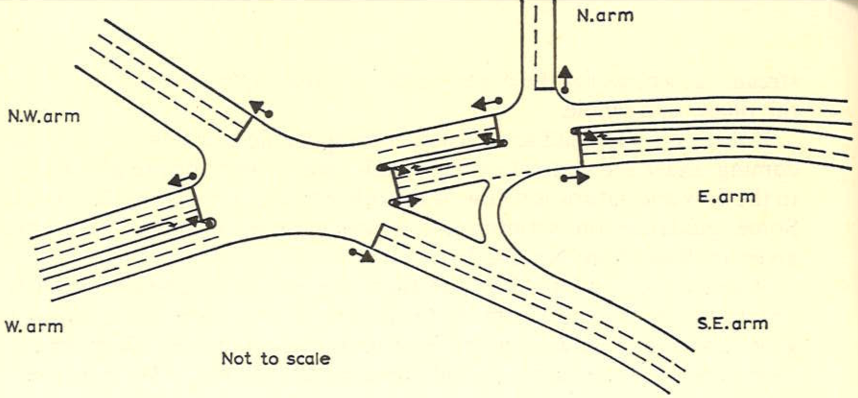


FIG. 33. Designation of flows and q/s values when left filter used (assuming left filter running during the green period for Arm C)

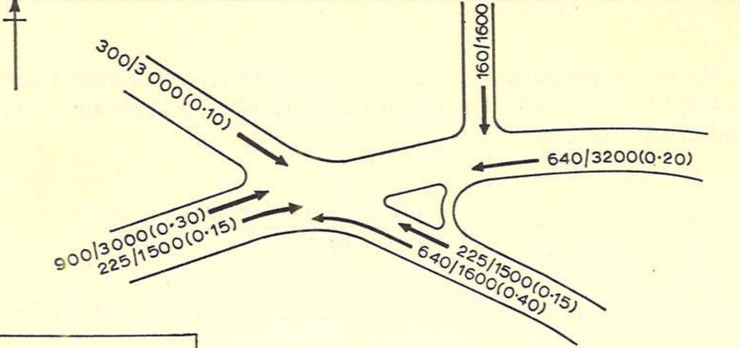
possibilities to consider: (1) when the left-turning flow is so great that it requires more green time than phase 1 and phase 2 traffic combined, and (2) when the left-turning traffic can quite adequately be catered for during these two phases. If the intergreen time between phases 1 and 2 could be ignored it would only be necessary to compare the magnitudes of $(q/s)_L$ and $(q/s)_{A,B}$* $+(q/s)_C$, but in most cases this is not so and a derivation of the optimum cycle time has to be made in order to determine which flows predominate. In the first of these two cases, the cycle time (call it c_1) would be

$$c_1=\frac{1{\cdot}5(I_{31}+I_{23}-2a+l_L+l_D)+5}{1-(q/s)_L-(q/s)_D}\text{ seconds}$$

*$(q/s)_{A,B}$ means $(q/s)_A$ or $(q/s)_B$, whichever is the greater

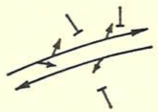
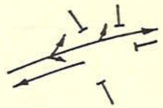
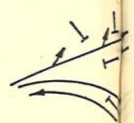
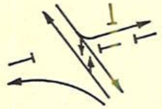
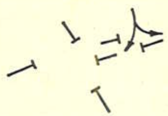
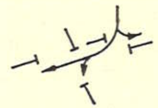


(a) Plan of intersection in worked example



Key
Each set of figures, e.g. 900/3000 (0·30), corresponds to flow/saturation flow (ratio of flow to saturation flow)

(b) Flows and saturation flows

STAGE	1		2		3		4		5		6		TOTAL
PERMITTED MOVEMENTS													
INTERGREEN TIME (seconds) (based on clearance times)		—		4		7*		4		—		4	
LOST TIME (1 second less than intergreen time)		0	5 (The whole of this stage is for clearance and should be counted as LOST TIME)	3		6		3		0		3	
FLOW / SATURATION FLOW OF ALL MOVEMENTS	← 0·30 (stages 1–3) → ← 0·20 →				← 0·15 → ← 0·40 (stages 3–4) →		← 0·10 → ← 0·15 →		← 0·10 (stages 5–6) →				
y VALUES (maximum flow / saturation flow for any stage)	← 0·20 →				← 0·15 →		← 0·15 →		← 0·10 (stages 5–6) →				0·60
LOST TIME (seconds)		0	5	3		7		3				3	21
ALTERNATIVE y VALUES					ALTERNATIVE ← 0·40 (stages 3–4) →								0·70
LOST TIME (seconds)						0							14

SUBSTITUTING IN THE CYCLE TIME FORMULA FOR THE TWO ALTERNATIVES GIVEN (1) $c = \frac{1\cdot5\,(21)+5}{1-0\cdot60} = 79$ SECONDS (2) $c = \frac{1\cdot5\,(14)+5}{1-0\cdot70} = 87$ SECONDS

THE CYCLE TIME OF 87 SECONDS IS THE CORRECT ONE AND THE PREDOMINANT MOVEMENT IN STAGES 3 AND 4 IS THE LEFT TURN FROM THE S.E. ARM (y VALUE OF 0·40)

*THE VALUE OF 7 SECONDS IS TO AVOID CONFLICT BETWEEN THE RIGHT TURN FROM THE S.E. ARM AND THE RIGHT TURN FROM THE W. ARM

(c) Table of calculations

FIG. 34. Calculation of y values for overlapping phases

where I_{31} is the intergreen period between phases 3 and 1, a is the amber period and l_L is the lost time for flow L. The meanings of the other symbols are obvious. In the second of the two cases

$$c_2 = \frac{1\cdot5(I_{31}+I_{12}+I_{23}-3a+l_C+l_{A,B}+l_D)+5}{1-(q/s)_C-(q/s)_{A,B}-(q/s)_D} \text{ seconds.}$$

The predominant arms are those whose y values give the larger value of cycle time.

If c_1 is the larger then the y values of phases 1 and 2 individually should be

$$\frac{(q/s)_C\,(q/s)_L}{(q/s)_C+(q/s)_{A,B}} \text{ and } \frac{(q/s)_{A,B}\,(q/s)_L}{(q/s)_C+(q/s)_{A,B}} \text{ respectively.}$$

(See 'Overlapping phases' and also worked example No. 10 in Appendix 7.)

Use of left filter with no exclusive left-turning lanes. In this case an estimate has to be made of the number of vehicles likely to be able to take advantage of the left filter, which will depend on the flow of left-turning traffic and on the geometry of the intersection. Both these factors can affect the likelihood of left-turners being blocked by those proceeding straight ahead. Vehicles which are able to take advantage of the left filter (expressed as a flow) should be subtracted from the combined flow, and the q/s value for this arm based on the flow so modified.

Overlapping phases. In many complicated signal systems several streams are allowed to flow concurrently without necessarily starting and stopping at the same time. The calculation of the y values and the optimum signal settings is complicated in these cases. To illustrate the method to be adopted an example has been worked out in Fig. 34. A plan of the intersection is shown in (a) and the flows, saturation flows and ratios of flow to saturation flow are given in (b) for the various main movements. The phasing is illustrated by the diagrams shown at the top of the table in (c). Since the various phases overlap it is convenient to divide the cycle time into the separate stages shown. Calculations of intergreen time based on the rules given earlier for clearances are shown next in the table. Most of the intergreens take the minimum value of 4 seconds, but one intergreen has to be made much higher to avoid possible conflict between right-turning vehicles starting up from the S.E. arm and the last right-turning vehicle from the W. arm. The dashes between certain stages indicate that there is no intergreen time. The lost times are one second less than the intergreen times. In the following row of the table the ratios of flow to saturation flow for all the main movements (indicated in the small diagrams at the top of the table) are given. Where a movement lasts for more than one stage this is indicated. It can be seen that for stages 3 and 4 there are several movements and combinations of movements taking place simultaneously. In the fifth row of the table the highest flow/saturation flow values are selected for the y values and the appropriate lost times are repeated in this row. It is not obvious which of the alternatives for stages 3 and 4 is the appropriate one to use so both are given. At the bottom of the table a calculation of cycle time indicates which of the alternatives is the correct one.

Practical points in setting signals

Criteria for optimum settings. Minimum overall delay has been used as the criterion in deducing optimum signal settings. Consistent with safety requirements this appears to be reasonable provided that the delay to any particular stream does not become excessively high in relation to that of other streams. The delay formula or tables may be used to check that delays are reasonable.

Other rules, which may be more arbitrary, could be applied in this connexion; for example, it may be considered that the average delay to any one stream should never exceed, say, three times the average delay to other streams or that not more than one per cent of the vehicles should be delayed more than a set time, e.g. 3 minutes.

Complicated situations. Many intersections are complicated either in layout, traffic composition or movement, but generally the effects of these are reflected in the measured values of saturation flow and lost time which are substituted in the signal formulae. However, unusually large numbers of right-turning vehicles, slow-moving vehicles, pedestrians, etc., can and should be catered for by special phases, all-red periods, etc., as discussed earlier.

Queues. If there are nearby junctions it is desirable to know how far the queue is likely to extend, and a formula has been derived[(1)] for predicting the average queue (N) at the beginning of the green period. This is generally the maximum queue in the cycle and is given approximately by

$$N = q\left(\frac{r}{2}+d\right), \text{ or } N = qr, \text{ whichever is larger} \quad \ldots\ldots\ldots\ldots\ldots(26)$$

where r is the effective red time, q is the flow (same units of time as r and d) and d is the average delay per vehicle.

This formula underestimates the extent of the queue by 5 to 10 per cent as it assumes that vehicles do not join the queue until they have reached the stop line. This assumption was made in order to simplify the theoretical model used for computing delays and queues. A correction for this formula is given in reference (1).

Of perhaps more importance than the average queue is the extent of the queue in certain infrequent cases, e.g. the queue which will be exceeded only once in 20 cycles and the queue which will be exceeded only once in 100 cycles. Values of these critical queues have been computed and are given in Tables 7 and 8.

Stops and starts. An expression for the proportion of vehicles which stop at least once (E) has been deduced:[(1)]

$$E = \frac{1-\lambda}{1-y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(27)$$

Formulae for the average number of stops and starts per vehicle have also been derived and are given in reference (1). These factors may be important when considering wear and tear of vehicles, fuel consumption and annoyance to drivers.

Table 7

Critical maximum queues (1 *in* 20)

Probability of the maximum queue in any cycle exceeding the critical value given in this table is 5 per cent

Degree of saturation	λ \ M	2·5	5·0	10·0	20·0	40·0
0·3	0·4	5	7	12	20	34
	0·6	4	5	9	15	24
	0·8	3	4	6	9	15
0·5	0·2	6	7	15	26	47
	0·4	5	7	12	20	35
	0·6	4	5	9	15	24
	0·8	3	4	6	9	15
0·7	0·2	7	9	15	25	44
	0·4	6	8	12	20	34
	0·6	5	7	9	15	25
	0·8	5	5	7	9	15
0·8	0·2	9	12	16	25	46
	0·4	8	11	14	21	35
	0·6	8	9	11	16	25
	0·8	7	8	9	11	16
0·9	0·2	19	18	22	30	49
	0·4	19	17	20	23	39
	0·6	19	16	17	21	34
	0·8	18	15	15	18	22
0·95	0·2	36	28	33	40	55
	0·4	35	27	30	35	47
	0·6	34	26	25	34	39
	0·8	34	25	27	27	32
0·975	0·2	74	63	65	62	84
	0·4	74	57	65	59	75
	0·6	69	61	62	54	65
	0·8	65	56	61	52	64

Table 8

Critical maximum queues (1 *in* 100)

Probability of the maximum queue in any cycle exceeding the critical value given in this table is 1 per cent

Degree of saturation	λ \ M	2·5	5·0	10·0	20·0	40·0
0·3	0·4	6	9	14	23	38
	0·6	5	6	11	17	28
	0·8	3	5	7	12	17
0·5	0·2	7	9	17	29	53
	0·4	6	9	14	23	38
	0·6	5	7	11	17	28
	0·8	4	5	7	12	18
0·7	0·2	9	12	17	28	50
	0·4	9	9	15	23	38
	0·6	8	9	12	18	28
	0·8	7	7	8	12	18
0·8	0·2	13	15	19	28	50
	0·4	12	13	17	24	39
	0·6	12	13	14	20	28
	0·8	11	12	12	15	18
0·9	0·2	29	25	29	38	55
	0·4	28	24	27	33	46
	0·6	27	24	26	28	42
	0·8	27	23	24	25	29
0·95	0·2	40	36	38	47	65
	0·4	40	34	37	44	55
	0·6	40	32	30	42	48
	0·8	39	32	34	36	40
0·975	0·2	82	70	79	69	93
	0·4	83	66	75	65	82
	0·6	82	70	69	58	79
	0·8	79	65	66	56	79

Number of fully saturated cycles. Curves have been produced showing the proportion of cycles which are fully saturated[1] (see Fig. 35). Measurement of this quantity may provide a relatively easy way of determining the degree of saturation at a junction, and hence the amount of reserve capacity.

It has been shown that the degree of saturation should be the same for all the predominant arms (maximum q/s values) of an intersection when the signal timings are optimum and is given by

$$x_o = \frac{2Y}{1+Y} \qquad \text{(28)}$$

where x_o is the degree of saturation under optimum conditions.

All the results given in this paper on delays and signal settings refer to equilibrium conditions. If, however, more traffic is arriving at an intersection than can be discharged the results obtained do not apply because the queue will steadily increase, and a method has been suggested in 'Effect of a parked vehicle on delay' for dealing with this situation.

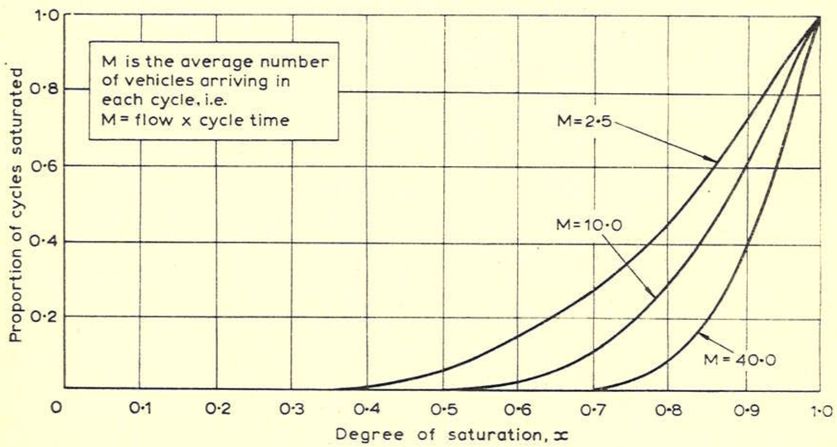


FIG. 35. The proportion of all cycles which are fully saturated

OPTIMUM SETTINGS: VEHICLE-ACTUATED SIGNALS

All the results given previously for fixed-time signals apply equally to vehicle-actuated signals which are operating on a fixed cycle because of heavy traffic demands. They can be used reasonably well for vehicle-actuated signals which are frequently 'running to maximum'. Where, however, there are frequent 'gap' changes these results are not applicable directly, and consequently more simulation work was carried out specifically for vehicle-actuated operation. As before, it was assumed that traffic was random and that saturation flow was constant. Examples of the variation of calculated delay with vehicle-extension period and maximum period are shown in Fig. 36. It would appear that for minimum delay

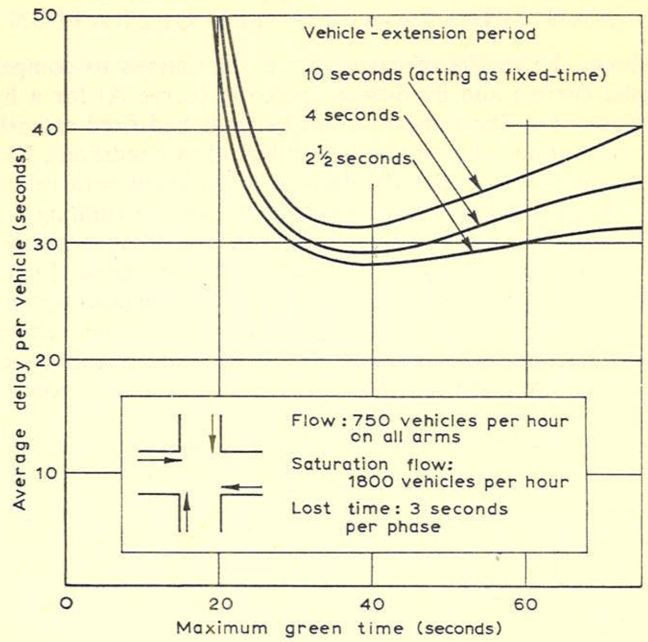


FIG. 36. Calculated effect of the vehicle-extension period and the maximum period on delay at vehicle-actuated signals

a fixed vehicle-extension period should be as short as is practicable so that the signals just allow the queue to disperse before changing to the other phase if there is waiting traffic. This is difficult to attain in practice because, unlike the theoretical model assumed, the discharge rate varies within a cycle and between cycles, and a vehicle-extension period which is just greater than the average discharge interval would under actual conditions frequently cut off part of the queue. This would have an adverse effect on delay. Furthermore, safety requirements demand that a fixed vehicle-extension period should not be too short (say, at least 4 seconds) and this state of affairs is not then likely to arise. Speed-timed extensions are terminated as soon as the vehicles initiating them have entered the junction, so that in general they will be shorter than the corresponding fixed-time setting which could be employed. It is not desirable that they should be further reduced.

It will be seen from Fig. 36 that the best value of the maximum period does not vary much with different vehicle-extension periods, and the results of the fixed-time work may reasonably be used as a guide to the value of the maximum period. In any case, the majority of vehicle-actuated signals act virtually as fixed-time signals in heavy traffic. However, if signal timings are not to be checked very often it would be advisable to set the maximum periods a little on the long side, to account for future growth, particularly if the vehicle-extension period is short. Alternatively, the variable maximum facility could be used, and in this case the pre-set maximum may be set somewhat on the short side.

COMPARISON OF FIXED-TIME AND VEHICLE-ACTUATED OPERATION

Figure 37 shows the results of carrying out simulations to compare vehicle-actuated (solid curves) and fixed-time operation (curve A) for a hypothetical four-arm intersection. The vehicle-actuated signals had fixed extension periods and pre-set maximum greens chosen to suit high-flow conditions. It can be seen that under light-flow conditions the delay is much lower with vehicle-actuated signals. At higher flows the vehicle-actuated signals are running to maximum more often and the two types of curve converge. The point at which this occurs depends largely on the value of the vehicle-extension period. In this example the saturation flow is assumed to be constant at 1800 vehicles per hour, corresponding to a discharge interval of 2 seconds, so that when the vehicle-extension period is $2\frac{1}{2}$ seconds the signals change soon after both queues have dispersed. With a longer vehicle-extension period the signals run to maximum at much lower flows.

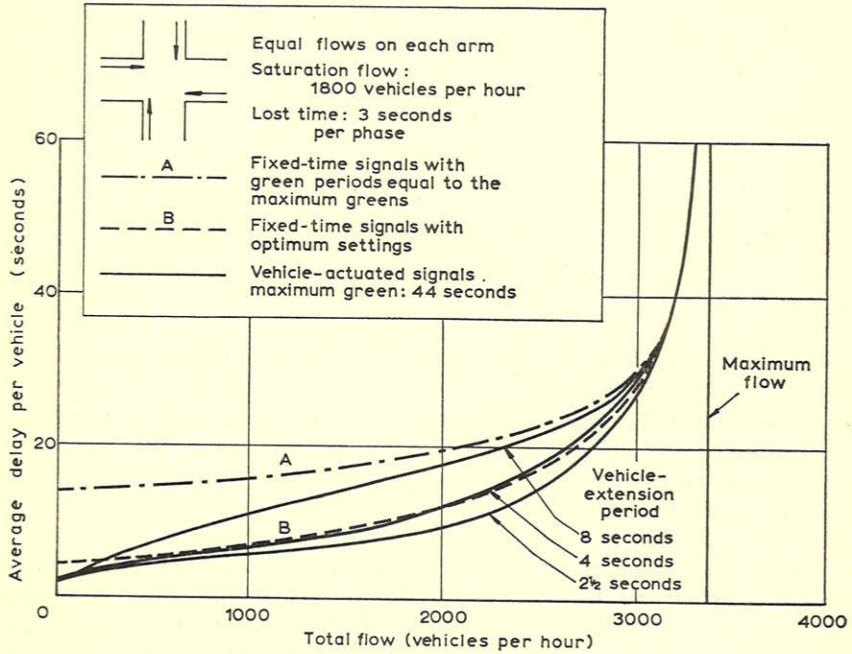


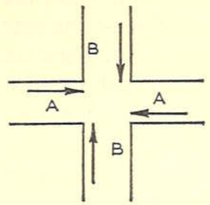
FIG. 37. Comparison of vehicle-actuated and fixed-time signals (4-way intersection)

In the fixed-time case (curve A) the green periods are equal to the maximum greens of the vehicle-actuated signal. However, when the green periods are optimum values (according to rules given in this Paper) for each distinct value of flow the delay is represented by curve B, which is practically identical with the vehicle-actuated curve corresponding to a vehicle-extension period of 4 seconds. Thus, vehicle-actuated signals with fixed extensions (particularly if they are long ones) are unlikely to be better (except under very light traffic conditions), and may be worse, than fixed-time signals adjusted to optimum settings. This result is mainly of theoretical interest,* however, because of the practical

*The result is of considerable importance in estimating delay at vehicle-actuated signals (see 'Delay with vehicle-actuated signals')

Table 9

Effect on delay of a change in traffic pattern



2500 vehicles per hour entering the junction

Saturation flow on each arm: 1800 vehicles per hour

Lost time: 3 seconds per phase

Maximum green times with vehicle-actuated signals: 44 seconds

Details of controller		Average delay per vehicle (seconds) Ratio of flows (A/B): 1	1·5	2·5
Vehicle-actuated signals with vehicle-extension periods of (seconds):	$2\frac{1}{2}$	15	15	14
	4	18	18	17
	8	22	24	33
Fixed-time signals	44 seconds green per phase	23	29	overloaded
	Optimum settings	18	17	16

difficulty of continuously adjusting fixed-time signals so that the settings are always optimum. (By comparison, one setting only of the vehicle-actuated signal is necessary to cater for the whole range of flows shown in Fig. 37.) There is also the further point that under light and even moderate traffic conditions practical limits have to be set on the duration of a green signal, thus preventing very short cycles with fixed-time signals and making such signals less efficient.

Vehicle-actuated signals with speed-timed extensions, however, should show an advantage over fixed-time signals even if the latter could be adjusted continuously to the flow for the particular time of day, since the speed-timed extensions are likely, on average, to be less than 4 seconds.

Because vehicle-actuated signals with short extensions are self-adjusting to traffic requirements it is possible safely to select somewhat longer maximum green periods than the appropriate green periods for fixed-time signals and so provide for short-period bursts of traffic of a non-random nature such as may arise, for example, from a nearby factory at closing time. In particular, it is often desirable to set the maximum of an early cut-off period to a longer setting than the calculated one having regard to the dislocating effect on the general traffic flow of right-turners which have been held.

A comparison of vehicle-actuated and fixed-time working has also been made for cases where the total flow entering the junction is assumed to be constant but the ratio of the flows on the two phases varies from 1:1 to $2\frac{1}{2}$:1. This is illustrated in Table 9, where it can be seen that the vehicle-actuated signal with short vehicle-extension periods (without any change in the signal settings) gives no increase in delay as the traffic pattern alters and only a slight increase when the vehicle-extension period is 8 seconds, whereas the fixed-time signal becomes overloaded with the $2\frac{1}{2}$:1 flow ratio. If the fixed-time controller could provide optimum settings for each flow ratio chosen then the delays would be comparable with those of the vehicle-actuated signal working with a short vehicle-extension period.

Thus, the real value of a vehicle-actuated signal is not that it caters for the random variation of traffic about the expected mean, i.e. variations in flow from cycle to cycle, but that it can deal satisfactorily with non-random bursts of traffic as well as with long-term variations both in the mean values of the total flow and in the flow ratio of the phases, *provided the vehicle-extension period is sufficiently short.* There is also, of course, the further advantage that, wherever possible, all signal changes are made during a gap in the traffic on the running phase, thus minimizing the risks inherent with arbitrary changes of right-of-way.

SPECIAL USES OF SIGNALS

Congested roundabouts

It is well known that a roundabout can 'lock' when the flow entering the roundabout approaches the capacity. The operation of the roundabout is said to be unstable in these regions of flow as driver behaviour may cause the roundabout to lock at flows well below the values which have been known to pass through it without any congestion.

In an attempt to improve capacity and reduce the chances of locking, traffic signals have been installed experimentally on the approaches to four roundabouts which formerly required police assistance during most peak periods (see reference (46) for details of one of these experiments). The signals were installed between 100 and 200 ft from each roundabout so that drivers, even though passing a green signal, would approach with caution and would not expect a 'guaranteed' free passage. Also, by setting the signals back it was intended that drivers would observe the zebra crossings, which had to be retained as the signals were operating only during peak periods. It was not possible to determine the improvement in capacity after the signals were installed as the maximum observed flows prior to signal control were found to vary over quite a large range, owing to the instability of operation. However, it was not uncommon for the flows under signal control to be 50 per cent greater than those which previously had led to locking of the roundabout. The performance with signals has therefore been compared with police control, with which there was no danger of locking taking place.

Signal control was found to increase the capacity by about 10 per cent above that with police control at two of the roundabouts, which were of the cross-roads type (four approaches). At two other roundabouts which were more complex, having six approaches, signal control gave no increase in capacity over police control. Locking was not entirely eliminated and usually occurred when the traffic signals temporarily allowed more traffic to enter the roundabout than could leave. It was found that increasing the all-red periods as traffic increased reduced quite substantially the number of occasions when locking took place, without causing a decrease in capacity.

The control equipment was arranged to enable the policeman on duty to extend manually the all-red time during any cycle, if conditions required it. More recently, a loop detector surrounding the central island has been used[47] together with its associated equipment, to assess the amount of traffic in the roundabout and to adjust automatically the all-red period in each cycle accordingly. The results have been very promising, and at the particular test site no lock-ups have been reported whilst the equipment has been in use.

It is interesting to note that at the two roundabouts of the cross-roads type, where the signals have been in operation for some years now, the capacity has increased quite appreciably. Although this is partly due to minor layout improvements and improvements in signal control it is quite likely that some improvement has also resulted from drivers becoming thoroughly used to the system.

An alternative treatment of congested roundabouts, which has proved very successful, is the use of priority signs on the approaches.[23][48] In some instances roundabouts have been removed entirely, and replaced by signals, where the volume and distribution of traffic have enabled a significant increase in capacity to be so achieved. Wherever this is contemplated the possibility of an increase in accidents should be borne in mind.[22]

High-speed roads

When vehicles are travelling at moderately high speeds on roads where there are traffic signals, drivers may sometimes find themselves unable either to stop in time when the signals change to amber or to continue at the same speed and pass through the junction before the 'red' commences. Investigations have been made[49] on a disused airfield to find comfortable stopping distances for most drivers, at various speeds, when the lights turn to amber and at what distance from the lights drivers decide to carry on and cross on the red. The results have been used for the design of a new type of control equipment, for use on fast roads, which will give each vehicle an extension of the green from the moment at which the vehicle can no longer make a comfortable stop. Normally, the extension of the green commences when a vehicle crosses the detector.

Two detectors are required, one at about 500 ft from the stop line and the other at the normal distance. When a vehicle crosses the outer detector its speed is measured and the time is computed at which the vehicle, continuing at constant speed, will reach the position at which it could no longer stop with comfort if the lights should turn to amber. Whilst this period is being timed off the signals are not prevented from changing to the side road, but if they have not changed at the expiry of this time the vehicle-extension period commences in the normal way. The second detector provides a 'fine control' and modifies the extension based on a new estimate of the vehicle's speed.

Until this new controller is completely developed a system known as double detection is being used. A detector is installed at both 130 and 240 ft (approximately) from the stop line and vehicles receive an extension of right-of-way on passing over each detector, the length of the extension being just sufficient to take the vehicle at its measured speed from one detector to the next and from the second detector to the intersection. The disadvantages with this method are (1) 240 ft is not far enough away from the intersection for most vehicles travelling at over 45 mile/h to stop in time and (2) slow vehicles are given two fairly long extensions of right-of-way which decrease the chances of a 'gap' change of right-of-way. Sometimes the more distant detector is placed only in the faster lanes, thus minimizing the second shortcoming.

Shuttle working (roadworks, bridges, etc.)

The formulae for delay, green times and cycle time given in this Paper can be adapted for use at roadworks and bridges where shuttle working is controlled by traffic signals. Saturation flow should be obtained by counting the number of vehicles (n) entering the controlled section from the time the first vehicle in the queue passes to the time the last vehicle in the queue passes and recording this time interval (f seconds). Vehicles arriving after the main body of the queue has passed should be ignored as they are not passing through at the maximum rate.

The saturation flow is

$$s = \frac{3600\,(n-1)}{f} \text{ vehicles per hour} \quad \ldots\ldots(29)$$

About half-an-hour's observation should be sufficient to give a reasonable result. The saturation flow should be measured in both directions—more or less the same results should be obtained. In addition to this information the time taken for the last vehicle in the queue to traverse the controlled section should be noted on a few occasions and the mean value found (t seconds). The signal cycle to give the least overall delay is

$$c = \frac{3t+5}{1-\frac{q_A}{s_A}-\frac{q_B}{s_B}} \text{ seconds} \quad \ldots\ldots (30)$$

where q_A and q_B are the flows (vehicles per hour) in the two directions and s_A and s_B are the corresponding saturation flows. For fixed-time working the green times, g_A and g_B (assuming $s_A = s_B$) are given by

$$g_A = \frac{c-2t}{1+\frac{q_B}{q_A}} \text{ seconds} \quad \ldots\ldots (31)$$

$$\text{and } g_B = c - 2t - g_A \quad \ldots\ldots (32)$$

and the all-red time at each change of right-of-way should be t seconds. With vehicle-actuated working the maximum green times may be set somewhat longer. If detectors for extending the all-red period are installed the all-red extension should be sufficient to ensure that the slowest vehicle will travel from one detector to the next before the extension has expired and the maximum all-red should be sufficient to cater for the slowest vehicle. It should be noted that where all-red extending detectors are installed it is usual for the last one or two to be unidirectional, responding only to vehicles entering, so that the signals can change as the clearing vehicle is running out of the controlled area.

Fixed-time signals should be reset (manually or by time switch) for each peak period, off-peak (daytime) and off-peak (night-time) working in order to give reasonable service.

APPLICATION OF INFORMATION GIVEN IN THIS PAPER

The information given in this Paper can be used as follows:

(1) In the various stages of design of a signal scheme at an intersection, e.g. phasing, geometrical layout of the intersection for a given capacity, siting of components, carriageway markings, optimum signal timings to give the least overall delay to vehicles.

(2) In evaluating the delay, queues, capacity, etc., for any existing or projected signal-controlled intersection which does not form part of a linked system.

(3) In estimating in advance the effect on delay and capacity of proposed changes in the numbers of right-turning vehicles, goods vehicles, buses or parked vehicles. Similarly, the effect of widening an approach to an intersection can be estimated, and an economic assessment of the benefits of an improvement scheme can be related to its cost.

(4) In designing new intersections to fit the expected traffic, various types of intersection can be compared from the standpoint of delays, accidents (using results published elsewhere)[22][23] capacity and cost.

(5) In the design of a linked system of traffic signals.

APPENDIX 1

GLOSSARY, SYMBOLS AND USEFUL EQUATIONS

The following symbols are in alphabetical order, with the only Greek symbol appearing at the end. Useful equations are also included in this list.

a Amber period (3 seconds in Great Britain)

c Cycle time

c_o Optimum cycle time—the cycle time which gives the least average delay to all vehicles using the intersection

$$c_o = \frac{1{\cdot}5\,L+5}{1-Y} \text{ seconds } (L \text{ in seconds}) \quad \ldots\ldots\ldots\ldots(22)$$

c_m Minimum cycle time—the cycle time which is theoretically just long enough to pass the traffic through the intersection.

$$c_m = \frac{L}{1-Y} \quad \ldots\ldots\ldots\ldots(11)$$

d Average delay per vehicle on a single approach to an intersection. It is the difference between the average journey time through the intersection and the time for a run which is not stopped or slowed down by the signals

$$d = \frac{c\,(1-\lambda)^2}{2\,(1-\lambda x)} + \frac{x^2}{2q\,(1-x)} - 0{\cdot}65 \left(\frac{c}{q^2}\right)^{\frac{1}{3}} x^{(2+5\lambda)} \quad \ldots\ldots\ldots\ldots(15)$$

Approximate form of d

$$d = \frac{9}{10}\left\{\frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)}\right\} \quad \ldots\ldots\ldots\ldots(16)$$

Using Tables 3, 4 and 5 for values A, B and C

$$d = cA + \frac{B}{q} - C \quad \ldots\ldots\ldots\ldots(17)$$

E Proportion of vehicles which are stopped by the signals at least once

$$E = \frac{1-\lambda}{1-y} \quad \ldots\ldots\ldots\ldots(27)$$

G Combined green and amber periods

$$G = k + a = g + l$$

g Effective green time—the sum of the green period and the amber period less the lost time for the particular phase

$$g = G - l \quad \ldots\ldots\ldots\ldots(5)$$

For optimum conditions

$$\frac{g_1}{g_2} = \frac{y_1}{y_2} \quad \ldots\ldots\ldots\ldots(23)$$

where suffix 1 and suffix 2 refer to phase 1 and phase 2

$$g_1 = \frac{y_1}{Y}\,(c_o - L)\,,\ldots \quad \ldots\ldots\ldots\ldots(24)$$

I Intergreen time—time from end of the green period of the phase losing right-of-way to the beginning of the green period of the phase gaining right-of-way

k Controller setting of green time

$$k = G - a$$

l Lost time for a single phase—the time in a cycle which is effectively lost to traffic movement in the phase because of starting delays and the falling-off of the discharge rate which occurs during the amber period

$$l = G - g$$

L Total lost time per cycle—the sum of the lost times for each phase and those periods when all signals show red or red with amber

$$L = \Sigma\,(I-a) + \Sigma l$$

If l=2 seconds and a=3 seconds, $L = \Sigma\,(I-1)$ seconds

M Average number of vehicles arriving per cycle on the particular approach

$$M = qc$$

N Average queue at the beginning of the green period

$$N = q\left(\frac{r}{2}+d\right) \text{ or } N = qr, \text{ whichever is larger} \ldots\ldots\ldots\ldots(26)$$

q Flow—average number of vehicles passing a given point on the road in the same direction per unit of time

r Effective red time—time during which the signal is red or red with amber on a particular phase, plus the lost time for that phase

$$r = c - g$$

s Saturation flow—average rate of flow past the stop line over that portion of the green period during which there is a queue, but ignoring the first few seconds of green whilst the rate of discharge is increasing $s = 160\ w$ for w between 18 ft and 60 ft. For w between 10 and 17 ft see 'Estimation of saturation flow'

w Width of approach at the stop line—measured from kerb to inside of pedestrian refuge or centre line, whichever is the nearer, or to inside of central reserve in the case of a dual carriageway. For approaches of non-constant width (e.g. flares, bell-shapes) w refers to the effective width (see 'Estimation of saturation flow')

x Degree of saturation—ratio of the flow to the maximum flow which can be passed through the intersection from the particular approach

$$x = \frac{q}{\lambda s}$$

x_o Degree of saturation when the cycle time and green times have optimum values

$$x_o = \frac{2Y}{1+Y} \qquad \text{.....................................(28)}$$

y Maximum ratio of flow to saturation flow for a given phase

$$y = \frac{q}{s}$$

Y Summation for the whole intersection of the y values corresponding to each phase

$$Y = \Sigma y$$

λ Proportion of the cycle which is effectively green for a particular phase

$$\lambda = \frac{g}{c}$$

Phase A phase is the sequence of conditions applied to one or more streams of traffic which, during the cycle, receive simultaneous identical signal indications

APPENDIX 2

TAILOR-MADE LINKED SIGNAL SYSTEMS

The term 'tailor-made' has been used for the linking systems described in this Appendix because of the extensive variety of links and linking combinations available to meet differing traffic requirements. It follows, therefore, that wherever linking is contemplated a careful assessment of traffic needs is essential if the most effective linking arrangement is to be selected.

Linking is generally employed either to facilitate movement of traffic through two or more junctions (forward linking), or to prevent the build-up of a queue at one junction from interfering with the preceding junction (backward linking). In either case it is unusual for any two or more junctions which are linked together to be of equal importance; generally one junction will be of greater importance and this becomes the 'key' intersection—it originates the linking pulses which are fed to the 'controlled' intersection(s). Where several junctions are linked together it is not essential for all controlled intersections to be linked directly to the key intersection. A controlled intersection from the key intersection may itself be the controlling intersection for a third intersection, thus giving a form of cascade control through the system. Provided the intersections along a route are in decreasing order of importance a cascade control using the first intersection as key may offer advantages, but where, say, the third intersection along the route from the key intersection is more important than the second, it will generally be advisable to control both directly from the key intersection. Special forms of linking are available for the unusual situation of linking between intersections of equal importance.

In order to facilitate consideration of the various links available, they have been numbered and also divided into three groups. Groups I and II are concerned with linking between junctions of unequal importance and Group III with linking between junctions of equal importance. In Group I, the pulses from the key intersection influence the controlled intersection immediately, and these links are generally suitable either where intersections are close together (so that vehicle transit time between intersections does not have to be taken into account) or where the purpose of the link is to prevent build-up of queues between the intersections. In Group II, the facilities are similar except that the pulses from the key intersection are delayed before they influence the controlled intersection: the delay may correspond to vehicle transit time, to provide progression, or with the period required to fill a reservoir space between intersections.

Group I—No delay links

Facility No.	Description
1. *Detector*	Commencement of a particular phase at the key intersection causes a demand for a selected phase at the controlled intersection, and, in addition, detector operations

on one or more approaches during the particular phase at the key intersection are repeated to the selected phase equipment at the controlled intersection.

This is the loosest form of link available and is suitable where there is no need to influence the local control other than by providing advance indication of the approach of vehicles to the controlled intersection. Under light traffic conditions it will tend to reduce delays in the direction of progression, but will have negligible effect under heavy traffic conditions.

1a. *Lock* (Note 1) — As facility No. 1 above, and, in addition, right-of-way is maintained on the selected phase at the controlled intersection for the duration of the particular phase at the key intersection.

This link is useful where there is no need to force controllers to be in step, but where it is desirable to hold them in step if they happen to be so. It will tend to reduce delays in the direction of progression under light and moderate traffic conditions but will have little effect under heavy traffic conditions.

2. *Forced change* (Notes 1 and 4) — Commencement of a particular phase at the key intersection causes a forced change to the selected phase at the controlled intersection (either immediately or, if the minimum green period is in operation, at the end of such period).

This link is appropriate where it is unnecessary to provide progression from the key intersection to the controlled intersection (the street may be one-way in the opposite direction) but where, because of restricted reservoir space available between the junctions, it is desired to ensure that the cycle time at the controlled intersection does not exceed that at the key intersection (so that a platoon of traffic is passed from the controlled to the key intersection for each cycle of the latter). The link simply pulls the controlled intersection into synchronism if it tends to lag.

2a. *Forced change and lock* (Notes 1 and 4) — As facility No. 2, and, in addition, the forced change is followed by right-of-way maintained at the controlled intersection for the duration of the particular phase at the key intersection.

This arrangement can be used where it is desirable for controllers to be in step, but where it is not essential to override local conditions to achieve this. Whilst they are in step, however, the controlled intersection cannot make a 'gap' change. This form of linking provides a bias in favour of the route from the key to the controlled intersection without ensuring progression.

2b. *Forced change and re-set maximum* (Notes 1 and 4) — As facility No. 2, and, in addition, if the selected phase is already running, the maximum timer is re-set (Note 2).

The signals remain with phases linked as long as platoons are sufficiently concentrated to prevent gap changes. The link is useful, therefore, where it is desired to favour concentrated platoons, but where difficulties will not arise if gap changes occur. Where a more rigid link is needed this facility (2b) may be combined with 2a to prevent gap changes, and in this form it can be useful both to facilitate progression and to prevent queue formation.

2c. *Forced change and disconnected maximum* (Notes 1 and 4) — As facility No. 2, and, in addition, the maximum timer for the selected phase is disconnected for the duration of the particular phase at the key intersection.

This link is particularly useful where it is desired to limit the running time of a particular phase at the controlled intersection except when a platoon is passing through both junctions. It is of especial use where long cycle times may occur at peak periods at the key junction, and on these occasions a shorter cycle time is permissible at the controlled junction. At such times the controlled junction can be arranged to give a sequence: side-road, short main-road, side-road, long main-road, etc. As the cycle time at the key intersection shortens, the second side-road phase at the controlled intersection is progressively reduced until it is eliminated and the two junctions then work to a common cycle time. With this facility gap changes are permissible.

2d. *Forced change and disconnected timers* — As facility No. 2, and, in addition, the vehicle timer and maximum timers are disconnected for the duration of the particular phase at the key intersection (Note 3).

This link is the most rigid one and makes the controlled intersection a slave of the key intersection during the phase in which the link is applied. It can be used for progression or for queue prevention.

Group II—Delay links

These links are similar to those in Group I, except that a delay period is incorporated to allow for the transit time between intersections, or the queue build-up time. The links available are:

3a. *Delayed lock* (Note 1) — As 1a, except that the start of the linking pulse is subjected to a pre-selected delay period.

4–4d. *Delayed forced change etc.* (Notes 1 and 4) — As 2–2d respectively, except that the start of the linking pulse is subjected to a pre-selected delay period.

Group III—Links for junctions of equal importance

5. *Maximum* — Maximum timers on selected phases of two controllers are disconnected until both controllers are showing green on the selected phases.

This is the loosest form of Group III linking.

5a. *Maximum and pulse* — Maximum timers on selected phases of two controllers are disconnected until both controllers are showing green on the selected phases and, in addition, commencement of the selected phase at either controller causes a demand for the selected phase at the other controller (Note 1).

This link gives advance indication at one junction of a demand originating at the other.

5b. *Maximum and detector* — This link is an extension of 5a in that the detector operations during the selected phase at either controller are repeated to the selected-phase equipment at the other controller (Note 1).

This provides a fairly rigid link between two controllers. Where an even more rigid link is needed consideration should be given to using the more rigid links of Groups I or II or, alternatively, of using a single controller for the two junctions.

Note 1. This facility may be made conditional on a demand having been received from a local source for the selected phase at the controlled intersection, in which case it may be described as 'conditional ...... linking'. Alternatively it may be conditional on two, three or four demands having been received for the selected phase, in which case it may be described as 'two, three or four vehicle conditional ...... linking'.

Note 2. Facility 2b (or 4b) may be combined with facility 2a (or 4a) where linking is required to be more rigid than 2b (or 4b), but where the special conditions for 2c and 2d (or 4c and 4d) are not met.

Note 3. Facility 2d (or 4d) may be used only where there is a continuous artificial demand for a phase other than the particular phase at the key intersection, or where the particular phase is always automatically followed by another phase; or alternatively, an additional timer may be provided to limit the maximum running period on the selected phase.

Note 4. When an arbitrary change is brought about under 2, 2a-d, or 4, 4a-d, arrangements are made for the right-of-way to return to the phase losing right-of-way.

APPENDIX 3

THE MANUAL PREPARATION OF TIME-AND-DISTANCE DIAGRAMS

This Appendix is a copy of a paper produced by the Ministry of Transport and based on a paper originally prepared by the Vehicle-Actuated Road Signal Development Association, who have given permission for its reproduction.

1. Object

With the ever-increasing volume of traffic in towns, there is a corresponding increase in the number of signal-controlled intersections. In order to reduce cumulative delays to vehicles on and crossing a particular route it is desirable to link controllers together in such a manner as to provide a progressive flow along the route. Before the linking can be achieved, a time-and-distance diagram of the route must be prepared.

2. Initial information (See also Appendix B)

2.1. A traffic count should be taken at each intersection of the proposed link. This requires an initial decision on the number of intersections to be included in the scheme which may have to be modified later, depending on what is practicable. A record should be made of the number of vehicles in each hour of a 16-hour period on every approach and a note made of whether vehicles entering the junction from each approach go straight on, turn right or turn left. Two or more counts may be taken to illustrate wet and fine conditions, or any other special consideration.

2.2. If the available figures are out of date, then an 'increase' factor based on estimated traffic growth should be used. Consideration should in any case be given to the effect of traffic growth.

2.3. A scale diagram of the intersections showing gradients and any proposed civil engineering modifications should be provided. From a knowledge of the route a desired progression speed (or speeds) throughout the system can be estimated.

3. Number of phases

The determination of the number of phases does not form part of this note but is an essential prerequisite to the production of time-and-distance diagrams.

4. Determination of cycle time and split

4.1. The optimum cycle time for peak-period conditions (as assessed from the traffic count) for each intersection should be calculated (see Appendix A) from a knowledge of the number of phases, the length of any all-red periods, saturation flow and lost time. It may be necessary to perform several calculations for an intersection if the traffic census shows discrete patterns for different times of day, or days of week.

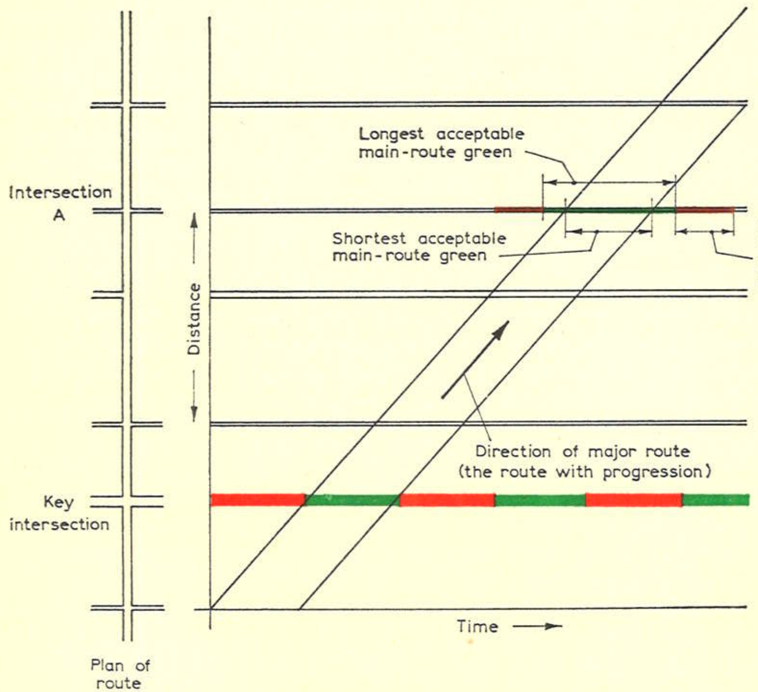
4.2. The intersection that is most heavily loaded and which requires, therefore, the longest cycle time, is designated the key intersection. The cycle time for this intersection should be used throughout the link. A different key intersection (and a different cycle time) may have to be designated for each of the different traffic situations that occur during a week, e.g. peak and off-peak, weekday and week-end. It will then be possible to decide on the number of plans required to meet the various traffic situations.

4.3. For each plan the actual times for each phase at the key intersection should be determined. The shortest and longest acceptable green times for the main route (i.e. the route with progression) should be determined for all other intersections (see Appendix A). These calculations will also give the longest and shortest green times for the side-road phase(s).

5. Preparation of the diagram

5.1. The preparation of time-and-distance diagrams as outlined in this note is essentially a matter of trial and error, the aim being to make each attempt an improvement on its predecessor. As a start, the proposed progression speed line (see paragraph 5.2.) for the direction of major flow is drawn through the origin of the time-and-distance diagram. Horizontal lines are then drawn across the diagram for each intersection at the appropriate distance from the start of the route (see diagram). The green and red periods (for the main route) at each intersection are now located, preferably on separate pieces of paper which can be adjusted as in 5.3, and the first time-and-distance diagram is thus formed. There will be a progression band along the main route at the desired speed.

5.2. The existing speed or travel times may be modified to allow for expected changes when the signals are installed and/or linked. In general an improvement can be anticipated. A survey should reveal whether a constant speed is appropriate. Physical characteristics of the road, e.g. gradients and curves, or traffic volumes may suggest that different speeds are appropriate for different sections of the route.



Preparation of time-and-distance diagram

5.3. In general, the longest acceptable main-route green periods at other intersections will be longer than that at the key intersection. The 'spare' green should be allocated as required to clear traffic turning into the main route at preceding intersections (to prevent these vehicles delaying the through platoons), to traffic in the opposite direction on the main route and to meet any other local traffic conditions.

5.4. The green and red periods for the opposite direction should be examined to see if they are suitable for the amount of traffic in that direction. Particular care should be taken to ensure that important side roads cannot be blocked by stored traffic and that satisfactory provision has been made for any significant traffic movements which use the main route for only a short distance within the main progression. The positions of green periods at intersections may be varied to assist other movements provided the cumulative adverse effect on the main progression is limited.

5.5. Where two linked systems cross one another, that intersection may be the key, and each system will have the same cycle time.

6. Comments

6.1. In general, opportunities for leaving the system should be greater than for entering.

6.2. To achieve an acceptable progression for both directions a change in the cyclic order of multi-phase intersections can be of assistance. Occasionally a change in cycle time is also of assistance but this usually means a longer cycle and lower speeds unless alterations to traffic movements at the key intersection permit a reduction.

6.3. It may be found that a time-and-distance diagram produces a zone in which cross-traffic cannot be accommodated. This will mean that either the diagram will have to be adjusted or another possible progression used, or that the cross-traffic at these points will have to be eliminated (by using one-way streets, for instance). It should be pointed out that within these 'impossible' zones, the main-route traffic can more easily make right turns. As a corollary to this, it is difficult to provide right turns at common points, and consideration may have to be given to prohibiting such movements.

7. Use of the diagram

When the optimum time-and-distance diagram has been drawn the pulse positions appropriate to the equipment should be determined from the diagram and the linking equipment can then be set up. The diagram indicates the cycle time for the system, the split at each intersection, and the offset time between successive intersections along the route.

8. Conclusions

8.1. The time-and-distance diagram is an aid in the setting up of a linked system. It is built up on a series of approximations and the designer must have a good knowledge of traffic behaviour.

8.2. After the linked system has been put into operation it may be necessary to make adjustments to the system to meet any changed traffic conditions. The diagram will then have to be re-drawn.

8.3. It is obvious that the difficulty of obtaining a satisfactory time-and-distance diagram is increased with every new intersection considered. It would appear that in practice the manual production of a time-and-distance diagram becomes very laborious if the number of controlled intersections on the route exceeds six.

8.4. The procedure for preparing a time-and-distance diagram for a one-way system is similar to, although simpler than, the procedure described above.

February, 1965

APPENDIX A

CALCULATION OF CYCLE TIME AND RANGE OF ACCEPTABLE GREEN TIMES TO PASS TRAFFIC ON ROUTES WITH PROGRESSION

Cycle time

For an isolated signal installation, where the mean traffic level is constant, and where vehicle arrivals on the approaches are at random it has been shown* that the cycle time for minimum delay is given by the expression (see below for definitions of terms and units)

$$c_o = \frac{1{\cdot}5L+5}{1-Y} \text{ seconds}$$

and the cycle time which is just sufficient to pass the traffic offered is given by

$$c_m = \frac{L}{1-Y} \qquad \text{......(1)}$$

This is the minimum possible cycle time and is associated with excessively long delays. In designing linked signal installations a cycle time should be chosen which provides a margin over the cycle time just sufficient to pass the traffic through the key intersection. In practice it will generally be appropriate if the minimum cycle time chosen is such that the installation is then loaded to 90 per cent of its capacity, i.e.

$$c_{\text{pract.}} = \frac{0{\cdot}9L}{0{\cdot}9-Y} \qquad \text{......(2)}$$

Cycle times should not normally exceed 120 seconds. Storage problems etc. may require the use of a lower value. If a cycle time lower than that given by equation (2) has to be used, then a check should be made, using equation (1), to ensure that the system will be capable of carrying the traffic.

Range of acceptable green times

After the cycle time for the linked system (c_l), has been determined the effective green times for the various phases at the key intersection will be

$$\frac{y_1}{Y}(c_l-L), \ldots$$

With an assumed standard amber of 3 seconds and a standard lost time per phase of 2 seconds, the actual green times are

$$\frac{y_1}{Y}(c_l-L)-1, \ldots \qquad \text{......(3)}$$

*WEBSTER, F. V. 'Traffic Signal Settings'. *Road Research Technical Paper* No. 39. London, 1958 (H.M. Stationery Office)

This formula gives the actual green times for the key intersection. For all other intersections in the system the formula may be regarded as giving the *shortest acceptable green time* (see diagram) for the main route (i.e. the route with progression). To obtain the *longest acceptable green time* for the main route a calculation is made of the shortest acceptable green time(s) for the side-road phase(s), i.e. the time just sufficient to pass side-road traffic with the side road(s) loaded to 90 per cent of capacity.

With the same assumptions as before of amber time and lost time per phase, the shortest acceptable side-road green time

$$= \frac{y_{\text{SIDE}}c_l}{0{\cdot}9} - 1 \qquad \text{......(4)}$$

The longest acceptable main-route green time will then be c_l minus the sum of the side-road greens (on separate phases) calculated according to equation (4) and minus intergreen times. It should be noted that where c_l is less than the value calculated according to equation (2) the longest acceptable main-route green times calculated by this method may be less than the values given by equation (3), in which case the equation (3) figures should be used and should not be varied (see example below).

On occasions it may also be necessary to check the minimum time required for traffic in progression to pass through an intersection in one direction or the other (particularly where the flow bands on the two directions do not correspond in time). Equation (3) should be used with the ratio of flow to saturation flow for the direction in question inserted for y.

All these quantities (except Y) should be in seconds.

c_o = cycle time for minimum delay with random traffic
c_m = minimum cycle time to pass traffic
$c_{\text{pract.}}$ = practical minimum cycle time
c_l = common cycle time for linked system
L = total lost time per cycle = $\Sigma(I-1)$, where the amber period is 3 seconds and the lost time per phase is 2 seconds
I = intergreen time
$Y = \Sigma y_1$ where y_1 is the maximum ratio of flow to saturation flow for phase 1

Example

(a) Suppose that at the key intersection $y_{\text{MAIN}} = y_{\text{SIDE}} = 0{\cdot}4$ and $\Sigma I = 12$ seconds.
It follows that $Y=0{\cdot}8$ and $L=10$ seconds.
According to equation (2)

$$c_{\text{pract.}} = \frac{9}{0{\cdot}9-0{\cdot}8} = 90 \text{ seconds,}$$

and according to equation (3) the green times should be

$$\frac{0{\cdot}4}{0{\cdot}8}(80)-1 = 39 \text{ seconds.}$$

This is the shortest acceptable green time for the MAIN route.

If at intersection A (see diagram) $y_{MAIN}=0{\cdot}4$ and $y_{SIDE}=0{\cdot}38$ and $\Sigma I=12$ seconds then according to equation (4) the shortest acceptable green time for the SIDE road would be

$$\frac{0{\cdot}38}{0{\cdot}9}(90)-1 = 37 \text{ seconds.}$$

The longest acceptable MAIN-route green is therefore 90—12—37 = 41 seconds.

This is 2 seconds more than the shortest acceptable MAIN-route green (i.e. green bandwidth for the progression).

(b) Suppose that the cycle time cannot be made as long as 90 seconds, but only 60 seconds.

Equation (3) gives the green times at the key intersection as

$$\frac{0{\cdot}4}{0{\cdot}8}(50)-1 = 24 \text{ seconds.}$$

This is the shortest acceptable green time for the MAIN route.

At intersection A the shortest acceptable SIDE-road green (from equation (4)) is

$$\frac{0{\cdot}38}{0{\cdot}9}(60)-1 = 24{\cdot}3 \text{ seconds}$$

and the longest acceptable MAIN-route green is $60-12-24{\cdot}3 = 23{\cdot}7$ seconds. This is less than the shortest acceptable value calculated above (24 seconds). The value of 24 seconds should therefore be taken as the green time for the MAIN route at intersection A.

A check can be made to see whether the SIDE road is given sufficient green as a result of increasing the MAIN-route green at this intersection; the SIDE road needs at least 0·38 (60)—1=21·8 seconds. The value given is 60—12—24=24 seconds and is therefore satisfactory.

APPENDIX B

PRELIMINARY WORK INVOLVED IN PREPARATION OF TIME-AND-DISTANCE DIAGRAM

1. *Drawings*
   a. Layout of drawing—time on horizontal axis, distance on vertical axis
   b. Decide scales to be used for time and distance
   c. Decide details to be shown to identify intersections and phases
   d. Prepare master copy of blank diagram
   e. Reproduce blank prints for working use
   f. Heading on diagram to show name of main route, cycle time and times of operation (or plan number)
2. *Surveys*
   a. Measure flows and distribution at all intersections
   b. Measure journey times along route
   c. Investigate need for any traffic engineering measures, e.g. waiting and/or loading restrictions
   d. Note sections of route where storage of vehicles possible and estimate storage capacity
   e. Note any particular need for clearance to prevent obstruction of side movements, i.e. where no storage can be allowed

APPENDIX 4

EFFECT OF A FLARED APPROACH AT A SIGNAL-CONTROLLED INTERSECTION

Figure 38 shows an approach with a half-width $(w+w_1)$ at the intersection, tapering to the normal half-width w over a distance d_1. Two cases are considered; case 1 where the length of flare d_1 is less than d'_1 (the length of approach occupied by the queue which can just pass through the intersection during a fully saturated green period) and case 2, where d_1 is greater than d'_1.

In the first case the saturation flow can be considered to be constant and the effective green period to be increased because of the extra vehicles which the taper can accommodate; in the second case, although the saturation flow falls off steadily throughout the green period to a value appropriate to the width at A (see Fig. 38 (case 2)) it is convenient to regard this value as the appropriate saturation flow and to assume the effective green time to be increased because of the extra vehicles accommodated in the triangle ABC.

In case 2, unlike case 1, the saturation-flow level and the gain in effective green time depend on the length of the green period.

It can be seen that

$$d'_1 = s_u w_u g v = s_u g A$$

where s_u = saturation flow per unit width
w_u = width of a lane
g = effective green period
v = average distance between successive vehicles (front to front) in a stationary queue
$A = v w_u$ = effective queuing area of a vehicle (which may be taken as about 220 ft²)

Case 1 $(d'_1 > d_1)$

The saturation flow in this case is considered to be constant at a value appropriate to the width w and the effective green time is increased.

From Fig. 38 (case 1) it can be seen that the area of the flare is $\frac{w_1 d_1}{2}$. To a first approximation this space could hold $\frac{w_1 d_1}{2A}$ vehicles.

The time taken by these vehicles to discharge at a saturation flow of $s_u w$ is $\frac{w_1 d_1}{2A s_u w}$, which is the gain in effective green time.

Case 2 $(d'_1 < d_1)$

In this case both the saturation flow and the effective green time are increased.

The saturation flow (s_A) appropriate to the width at A is given by

$$s_A = s_u (w + w'_1)$$

The increase in effective green time is determined as follows:

The triangle ABC has the area $\dfrac{d'_1 (w_1 - w'_1)}{2}$ and can contain $\dfrac{d'_1 (w_1 - w'_1)}{2A}$ vehicles.

The time taken by these vehicles to discharge at a saturation flow of s_A would be

$$\frac{d'_1 (w_1 - w'_1)}{2As_u (w + w_1)}$$

which is the gain in effective green time.

The value of d'_1 for optimum settings is difficult to determine but a method is given in Appendix 6.

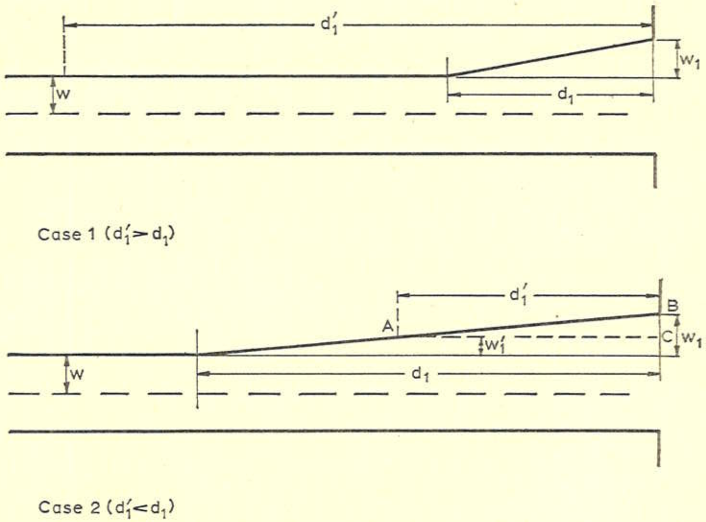


FIG. 38. Flared approaches at signal-controlled intersection

APPENDIX 5

NUMBER OF RIGHT-TURNING VEHICLES ABLE TO DISCHARGE THROUGH GAPS IN THE OPPOSING FLOW

The only period in the cycle when waiting right-turning vehicles can discharge through gaps in the opposing flow is from the end of the saturated flow of the opposing stream to the beginning of the red period. The first step is to calculate the length of this period.

It is assumed that there are no vehicles remaining in the opposing queue at the end of the green period—this is approximately correct with vehicle-actuated signals operating not too near to capacity.

If the saturated green time for the opposing flow is denoted by g_s then the number of vehicles discharged during this time is $(r+g_s)q$ where r is the effective red time and q is the opposing flow. The time they take to discharge is

$$g_s = \frac{(r+g_s)q}{s}$$

where s is the opposing saturation flow.

Thus $$g_s = \frac{rq}{s-q} = \frac{(c-g)q}{s-q} \qquad \text{.........................(5·1)}$$

where c and g are the cycle time and effective green time respectively.

If the unsaturated green time is denoted by g_u, then

$$g_u = g - g_s$$

Substituting from equation (5·1)

$$g_u = \frac{gs - qc}{s-q} \qquad \text{.................................(5·2)}$$

The proportion of unsaturated time, g_u/c, is

$$\frac{gs - qc}{c(s-q)} \qquad \text{.....................................(5·3)}$$

The second step is to calculate the effective right-turning saturation flow, i.e. the maximum flow through gaps in the opposing flow assuming 100 per cent unsaturated green time. Use is made of Tanner's formula[21] to give the curve shown in Fig. 22 (p.42) where two typical cases are considered.

Case 1. Single-lane opposing flow

The minimum acceptable gap, α, in the opposing flow through which one right-turner can discharge is assumed to be 5 seconds. The minimum headway in the opposing stream, β_o, is assumed to be 3 seconds and in the right-turning stream, β_r, $2\frac{1}{2}$ seconds. This gives a saturation flow of 1200 vehicles per hour for the opposing flow and 1440 vehicles per hour for the unobstructed right-turners (i.e. when there is no opposing flow).

Case 2. Opposing flow in two or more lanes

The numerical values chosen for α, β_o and β_r are 6, 1 and $2\frac{1}{2}$ seconds respectively. This gives a saturation flow of the opposing arm of 3600 vehicles per hour. The value of 6 seconds for α is slightly greater than in case 1, to allow for the greater distance in crossing the opposing flow.

Over most of the range there is little difference in the two curves shown in Fig. 22 and it was felt that no additional curves, based on other values of the parameters, were necessary.

The effective right-turning saturation flow s_r (through the opposing gaps) can be read off the curves in Fig. 22. The actual flow of right-turners through gaps in the opposing stream, q_r, can be estimated for the given signal settings by multiplying s_r by the proportion of unsaturated green time (equation (5·3)), i.e.

$$q_r = s_r \left\{ \frac{gs-qc}{c(s-q)} \right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5{\cdot}4)$$

This can be expressed in terms of the number of right-turning vehicles per cycle (n_r), i.e.

$$n_r = s_r \left(\frac{gs-qc}{s-q} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5{\cdot}5)$$

APPENDIX 6

OPTIMUM SETTINGS WHEN SATURATION FLOW FALLS OFF WITH INCREASING GREEN TIME

Observations have indicated that at most sites where saturation flow falls off gradually with increasing green time, the saturation flow histogram (see Fig. 30 on p.60) can be represented approximately by a quadrilateral, shown by the dashed lines in Fig. 30, where the area enclosed by the quadrilateral is equal to the area under the histogram.

It is assumed that in calculations of optimum settings a histogram of saturation flow is available as a starting point. The appropriate value of saturation flow to be used in calculating the optimum settings is that obtaining just as the amber period begins, because it is this value which determines the variation, caused by small changes in the green time, in the numbers of vehicles crossing the stop line under capacity conditions. Over a small range of green times about this particular value the saturation flow can be considered to be constant. The problem then is to find the length of green time which corresponds to a saturation flow (just as amber begins) and which, when substituted in the formulae for the optimum settings, produces the original green time. This has to be arrived at through successive approximations. Before a procedure for doing this is outlined some of the expressions used in this procedure will be deduced.

Theory

Suppose initial observations of saturation flow on phase 1 of a 2-phase intersection give the histogram shown in Fig. 30. Assume for simplicity that the other phase has a normal constant saturation flow. The quadrilateral ABEF, shown by the dashed lines, can be drawn from the original histogram, where E is arranged to be at the beginning of the amber period and the two triangles ABG and EHF have the same areas as the corresponding portions under the histograms. Let s_B and s_E be the saturation-flow levels at B and E respectively. If the green time ended earlier than shown, say at C, then the saturation flow at the commencement of amber would be s_1 as shown. Let us assume that the fall-off during the amber period would be as shown by the dotted line CD, i.e. taking time β as before, then

$$s_1 = s_B - \frac{\gamma_1}{\gamma}(s_B - s_E) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6{\cdot}1)$$

where γ_1 and γ are as shown in the diagram.

The effective green time, g_1, can be calculated from the areas shown on the diagram.

The total area of the quadrilateral ABCD is $\frac{1}{2}s_B\,\alpha + \frac{1}{2}(s_B+s_1)\,\gamma_1 + \frac{1}{2}s_1\beta$.

It is also, from the definition of effective green time, equal to $g_1 s_1$.

Thus, $$g_1 = \frac{(\alpha+\gamma_1)s_B + (\beta+\gamma_1)s_1}{2s_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6{\cdot}2)$$

If G is the combined green plus amber period the lost time is

$$l_1 = G - g_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6{\cdot}3)$$

The value of g_1 is not necessarily the optimum green time. A first approximation to the optimum green time (which can then be compared with g_1) will be calculated assuming (probably wrongly at this stage) that s_1 and l_1 are the appropriate values of saturation flow and lost time.

The optimum cycle time is

$$c_o = \frac{1{\cdot}5L+5}{1-Y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6{\cdot}4)$$

where L is the total lost time in the cycle and $Y = y_1 + y_2$. The suffixes refer to phase 1 and phase 2 and the individual y values are given by

$$y_1 = \frac{q_1}{s_1} \text{ and } y_2 = \frac{q_2}{s_2}$$

where q and s denote the flows and saturation flows respectively.

The value of L is given by

$$L = l_1 + l_2 + I_1 + I_2 - 6 \text{ seconds} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6{\cdot}5)$$

where I is the intergreen time in seconds (the 6 seconds in the expression is the duration of the two amber periods).

The total effective green time in the cycle, $c_o - L$, should be divided between the phases in the ratio y_1/y_2. Call these green times g_1' and g_2. Thus

$$g_1' = \frac{y_1}{Y}(c_o - L)$$

Substituting for c_o from equation (6·4) we have

$$g_1' = \frac{y_1}{Y(1-Y)}\left\{5 + L\left(Y + \tfrac{1}{2}\right)\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6{\cdot}6)$$

The value of g_1' so obtained should be compared with the value of g_1 used initially. A value halfway between the two effective green times should be adopted and the calculations repeated to find a second approximation to the optimum effective green time.

Thus, calling the new value of G, G',

$$G' = G + \frac{g_1' - g_1}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6{\cdot}7)$$

After a few iterations there should be little difference between g_1 and g'_1; in other words, values of saturation flow, lost time and effective green time have been found which satisfy the requirements of the histogram and also the formulae for optimum signal settings.

Procedure

The procedure for calculating the optimum settings for a 2-phase intersection where phase 1 has a falling-off saturation flow and phase 2 has a constant value is given below.

1. Construct the quadrilateral ABEF on the histogram of the observed saturation flow (see Fig. 30).
   Make the areas ABG, GBEH, and EHF equal to the appropriate areas under the histogram, and arrange that the point E comes at the beginning of the amber period. If the histogram continues beyond F this portion should be included in the area EHF.
2. Read off s_B, s_E, α, β and γ.
3. Select a value for G as the starting point for optimum timing calculations.
4. Calculate $\gamma_1 = G - a - \alpha$, where a is the amber period.
5. Calculate $s_1 = s_B - \frac{\gamma_1}{\gamma}(s_B - s_E) \quad \ldots\ldots\ldots\ldots\ldots (6{\cdot}1)$
6. Calculate the effective green time

   $$g_1 = \frac{(\alpha + \gamma_1)\, s_B + (\beta + \gamma_1)\, s_1}{2 s_1} \quad \ldots\ldots\ldots\ldots\ldots (6{\cdot}2)$$

7. Calculate the lost time (this may be negative)

   $$l_1 = G - g_1 \quad \ldots\ldots\ldots\ldots\ldots (6{\cdot}3)$$

8. Calculate the total lost time per cycle

   $$L = l_1 + l_2 + I_1 + I_2 - 6 \text{ seconds} \quad \ldots\ldots\ldots\ldots\ldots (6{\cdot}5)$$

   where I is the intergreen time and the suffixes refer to phases 1 and 2 respectively.
9. Calculate $y_1 = \frac{q_1}{s_1}$ and $y_2 = \frac{q_2}{s_2}$

   where q_1 and q_2 are the flows on the two phases and s_2 is the constant saturation flow of phase 2.
10. Calculate $Y = y_1 + y_2$.
11. Calculate a first approximation to the optimum green time assuming s_1 and l_1 are the appropriate values.

    $$g_1 = \frac{y_1}{Y(1-Y)}\left\{5 + L\left(Y + \tfrac{1}{2}\right)\right\} \quad \ldots\ldots\ldots\ldots\ldots (6{\cdot}6)$$

12. Select a new value of G, call it G'

    $$G' = G + \frac{g_1 - g_1}{2} \quad \ldots\ldots\ldots\ldots\ldots (6{\cdot}7)$$

13. Return to step 4 and work through the calculations again, substituting G' for G and at each step using the newly derived values of the parameters. The values which remain constant are α, β, γ, s_B, s_E, a, q_1, q_2, s_2, y_2, l_2, I_1 and I_2.
14. Continue with the iterations until the optimum green time deduced, the saturation flow and the lost time are all correctly related.
15. Calculate the optimum effective green time of phase 2 by substituting the newly derived values in the following equation

    $$g_2 = g_1 \frac{y_2}{y_1}$$

    See worked example No. 9 in Appendix 7.

APPENDIX 7

WORKED EXAMPLES TO ILLUSTRATE METHODS GIVEN IN THE PAPER

Example 1. Approach widths

The estimated flows in the design year are 300 vehicles per hour on the south arm of a T-junction and 2400 vehicles per hour on the east and west arms. What ratio of approach widths should be considered as a first step in the design of the signal-controlled junction and what green-time ratio would be necessary?

From equation (3) in 'The approach'

$$\frac{w_{EW}}{w_S} = \sqrt{\frac{q_{EW}}{2q_S}} = \sqrt{\frac{2400}{600}} = 2$$

$$\text{and } \frac{g_{EW}}{g_S} = \sqrt{\frac{2q_{EW}}{q_S}} = \sqrt{\frac{4800}{300}} = 4$$

Thus, consideration should be given, as a first step, to making the east-west approaches twice as wide as the south approach and having four times as much green time.

Example 2. Right-turning vehicles

Part 1. At an intersection where the right-turners from a particular arm have their own lane and are not hindered by an opposing flow, they follow a path having an average radius of curvature of 30 ft. What is the estimated saturation flow of this stream?

From equation (7) in 'Effect of right-turning traffic'

$$s = \frac{1800}{1+5/30} \text{ p.c.u./h} = \frac{1800\times 30}{35} = 1545 \text{ p.c.u./h}$$

If the layout is altered so that the right-turners can proceed in double file with an average radius of curvature for both lanes of 35 ft, what is the estimated saturation flow for the combined stream?

From equation (8)

$$s = \frac{3000}{1+5/35} = \frac{3000\times 35}{40} = 2625 \text{ p.c.u./h}$$

Part 2. What fixed early cut-off period is necessary to cater for a right-turning flow of 570 vehicles per hour at an intersection where the right-turners are in a single lane with a headway under saturated conditions of $2\frac{1}{2}$ seconds per vehicle and where there is an opposing flow of 750 vehicles per hour in two lanes? The effective green time for the opposing flow is 30 seconds and the cycle time is 60 seconds. The saturation flow of the opposing arm is 4750 vehicles per hour.

From Fig. 22 (p. 42), s_r can be seen to be 510 vehicles per hour, and substituting in equation (9) in 'Effect of right-turning traffic' gives

$$n_r = \frac{510}{3600}\left\{\frac{30(4750)-750(60)}{4750-750}\right\}$$
$$= 3{\cdot}5 \text{ vehicles per cycle}$$

The right-turning flow of 570 vehicles per hour gives an average of 9·5 vehicles per cycle.

Thus, on average, six right-turners will be left at the end of the green period and they will take $6\times 2\frac{1}{2}=15$ seconds to discharge.

A fixed early cut-off period plus the following intergreen period should in this example be 15 seconds (see Fig. 23, p. 43). Thus, assuming the intergreen period is 4 seconds, the early cut-off period should be set at 11 seconds (or the nearest controller setting).

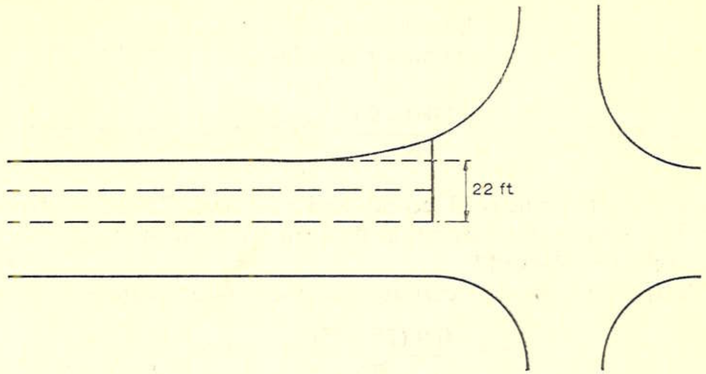


FIG. 39. Layout used in worked example No. 3

Example 3. Saturation flow

Part 1. An approach to a signal-controlled intersection has the layout shown in Fig. 39. The intersection is in a busy shopping street with many pedestrians. There is a 3 per cent uphill gradient. What is the saturation flow of the approach in p.c.u./h?

The standard saturation flow is $160w$ (see 'Estimation of saturation flow').

The site can be classed as a 'poor' site, thus $s = \frac{160\times 85}{100} w$.

To correct for the gradient, multiply by $\frac{91}{100}$, i.e.

$$s = 160\times 0{\cdot}85\times 0{\cdot}91w$$
$$= 124w$$

Since w=22 ft, s=2730 p.c.u./h.

Part 2. If 20 per cent of the vehicles turn right, and they are not given exclusive lanes for queueing in, what is the saturation flow under these conditions of the approach described in Part 1?

Each right-turner can be said to be equivalent to $1\frac{3}{4}$ straight-ahead vehicles. Out of every 100 vehicles the 20 which turn right are equivalent to 35 straight-ahead vehicles. Thus, 100 vehicles of mixed turning movements are equivalent to 115 vehicles going straight ahead.

The saturation flow is therefore $\frac{2730\times 100}{115} = 2380$ p.c.u./h.

Part 3. Taking the same intersection as above and assuming that the traffic consists of 61 per cent light vehicles, 20 per cent heavy vehicles, 9 per cent motorcycles and 10 per cent pedal cycles, what is the saturation flow in *motor* vehicles per hour?

Out of every 100 vehicles	61 'lights' are equivalent to	61 p.c.u.
	20 'heavies' are equivalent to	35 p.c.u.
	9 motorcycles are equivalent to	3 p.c.u.
	10 pedal cycles are equivalent to	2 p.c.u.

$$\left.\begin{array}{l}\text{100 vehicles}\\ \text{90 motor vehicles}\end{array}\right\}\text{are equivalent to 101 p.c.u.}$$

Thus the saturation flow is $\dfrac{2380\times 90}{101} = 2120$ motor vehicles per hour.

Part 4. If the green time is 30 seconds and a car parks 75 ft clear distance from the stop line, what is the saturation flow (in p.c.u./h) of the above approach whilst the vehicle is parked?

The effective reduction in carriageway width (from equation (10)) will be

$$5{\cdot}5-\frac{0{\cdot}9\,(75-25)}{30}\text{ ft} = 4\text{ ft}$$

The approach now has the equivalent of an 18-ft width instead of a 22-ft width.

Since the saturation flow is linear over this range the saturation flow, whilst the vehicle is parked, is

$$2380\times\frac{18}{22}\text{ p.c.u./h} = 1950\text{ p.c.u./h}$$

The result obtained in Part 2 is used here because the answer is required in p.c.u./h.

Part 5. What is the loss in capacity of the above approach caused by the parked vehicle? Assume the effective green time is 50 per cent of the cycle time.

Without the parked vehicle s=2380 p.c.u./h. The capacity is 50 per cent of this, i.e. 1190 p.c.u./h.

With the parked vehicle present s=1950 p.c.u./h and the capacity is 975 p.c.u./h.

The loss in capacity is therefore 215 p.c.u./h.

Example 4. Practical capacity

In designing a signal-controlled four-way cross-roads, where the flows are 2700 vehicles per hour on the east and west arms and 300 vehicles per hour on the north and south arms, what is the practical limiting value of Y, and what saturation flows should the approaches accommodate? The total lost time, L, per cycle can be assumed to be 10 seconds.

From equation (13) in 'Capacity of the whole intersection'

$$Y_{\text{pract.}} = 0{\cdot}9-0{\cdot}0075\,(10) = 0{\cdot}825$$

From equation (1) in 'The approach'

$$\frac{w_{\text{EW}}}{w_{\text{NS}}} = \sqrt{\frac{q_{\text{EW}}}{q_{\text{NS}}}} = \sqrt{\frac{2700}{300}} = 3$$

The saturation flows will also be in this ratio.

Thus $\qquad s_{\text{EW}} = 3s_{\text{NS}}$ and $q_{\text{EW}} = 9\,q_{\text{NS}}$

Therefore $\qquad y_{\text{EW}} = 3\,y_{\text{NS}}$

For $Y_{\text{pract.}}=0{\cdot}825$, this gives $y_{\text{EW}}=0{\cdot}619$ and $y_{\text{NS}}=0{\cdot}206$.

For flows of 2700 and 300 vehicles per hour the saturation flows are 4360 vehicles per hour for the east-west arms and 1455 for the north-south arms.

Example 5. Delay with fixed-time signals

One approach to an intersection has a flow of 1020 vehicles per hour, a saturation flow of 2400 vehicles per hour, a combined green and amber period of 32 seconds and a cycle time of 1 minute. The lost time owing to starting delays and the reduced flow during the amber period can be assumed to be 2 seconds. What is the average delay per vehicle?

The delay is given by equation (17) in 'Delay with fixed-time signals':

$$d = cA+\frac{B}{q}-C$$

$$c = 60\text{ seconds}$$

$$g = G-l = 32-2 = 30\text{ seconds}$$

$$\lambda = \frac{g}{c} = 0{\cdot}5$$

$$x = \frac{q}{\lambda s} = \frac{1020}{0{\cdot}5(2400)} = 0{\cdot}85$$

From Table 3 (p. 50) $A = 0{\cdot}217$

$cA = 13{\cdot}0$

From Table 4 (p. 51) $B = 2{\cdot}41$

$$\frac{B}{q} = \frac{2{\cdot}41}{1020/3600} = 8{\cdot}5$$

$$M = \frac{1020}{3600}(60) = 17{\cdot}0$$

From Table 5 (p. 52) $C = 12{\cdot}3$ per cent of first two terms

$$\begin{aligned} d &= 13{\cdot}0+8{\cdot}5-C\\ &= 21{\cdot}5-\frac{12{\cdot}3}{100}(21{\cdot}5)\\ &= 21{\cdot}5-2{\cdot}6\\ d &= 18{\cdot}9\text{ seconds.}\end{aligned}$$

Example 6. Delay caused by vehicle parked on the approach

What is the total duration of the effect of a car which parks for 30 minutes at an intersection where the flow is 1125 p.c.u./h? The saturation flows of the approach with and without the parked car are 1950 and 2380 p.c.u./h respectively and λ is 0·50.

The parked vehicle raises the degree of saturation to greater than unity; equation (20) on p. 55 gives the duration of the effect as

$$\frac{(q-\lambda S)\,T}{\lambda s-q}=\frac{(1125-975)\;30}{1190-1125}=69 \text{ minutes}$$

The effect of the parked vehicle lasts for 39 minutes after the vehicle has left, during which time drivers arriving will be unaware of the cause of the extra delay. What is the maximum individual delay?

This is given by equation (21) as

$$x(1-\frac{1}{X})T$$

where $x=\dfrac{1125}{1190}=0{\cdot}945$ and $X=\dfrac{1125}{975}=1{\cdot}154.$

The maximum individual delay is therefore

$$\frac{0{\cdot}945\;(0{\cdot}154)\;30}{1{\cdot}154}\text{ minutes}=3{\cdot}8\text{ minutes.}$$

Example 7. Fixed-time signals

Part 1. *Optimum settings.* Flows and saturation flows at a 2-phase signal-controlled intersection are as given in the table below. Both intergreen periods are 9 seconds and the lost times due to starting delays etc. are 2 seconds per phase. What are the optimum cycle time and optimum green times for minimum overall delay?

	North	*South*	*East*	*West*
Flow (q) in vehicles/hour	600	450	900	750
Saturation flow (s) in vehicles/hour	2400	2000	3000	3000
Ratio q/s	0·250	0·225	0·300	0·250
y values	0·250		0·300	

An expression for the lost time per cycle is given in 'Capacity of the whole intersection':

$$L=\Sigma(I-a)+\Sigma l$$
$$=6+6+2+2 \text{ seconds}$$
$$=16 \text{ seconds}$$

The optimum cycle time can be obtained by substitution in equation (22) in 'Cycle time':

$$c_o=\frac{1{\cdot}5\;(16)+5}{1-0{\cdot}250-0{\cdot}300}=\frac{29}{0{\cdot}450}=64\text{ seconds}$$

The total effective green time per cycle is c_o-L which is

$$64-16=48\text{ seconds}$$

The effective green times can be obtained from equation (24) in 'Green times':

$$g_{NS}=\frac{y_{NS}}{Y}\,(c_o-L)=\frac{0{\cdot}250}{0{\cdot}550}\,(48)=22\text{ seconds}$$

$$g_{EW}=\frac{0{\cdot}300}{0{\cdot}550}\,(48)=26\text{ seconds}$$

Since $G=g+l$,
$G_{NS}=24$ seconds and G_{EW}=28 seconds.

Part 2. *Reserve capacity.* What is the percentage reserve capacity of the intersection described in Part 1? Assume that the practical capacity is 90 per cent of the flow which can be accommodated with a 120-second cycle with the flows in the same ratios as given above.

When the cycle time is 120 seconds, $c-L$ will be 104 seconds (L retains its value of 16 seconds). The green times should therefore be

$$g_{NS}=\frac{0{\cdot}250}{0{\cdot}550}\,(104)=47\text{ seconds}$$

$$g_{EW}=\frac{0{\cdot}300}{0{\cdot}550}\,(104)=57\text{ seconds}$$

	North	*South*	*East*	*West*
Ultimate capacity $\left(\dfrac{gs}{c}\right)$ in vehicles/hour	940	780	1425	1425
90% of ultimate capacity in vehicles/hour	850	700	1280	1280
Present flow (q) in vehicles/hour	600	450	900	750
Reserve capacity in vehicles/hour	250	250	380	530
Percentage reserve capacity	42	56	42	71

The intersection therefore has a 42 per cent reserve capacity.

This result could, of course, have been obtained much more simply using equation (14) in 'Capacity of the whole intersection':

$$\text{Percentage reserve capacity}=\frac{100\,(Y_{\text{pract.}}-Y)}{Y}$$

From equation (13) $Y_{\text{pract.}}=0{\cdot}9-0{\cdot}0075L$
$=0{\cdot}9-0{\cdot}0075\;(16)$
$=0{\cdot}78$

From Part 1 of this example, $Y=0{\cdot}55$

Therefore, substituting in equation (14) gives reserve capacity=42 per cent.

Example 8. Average delay with vehicle-actuated signals

What is the estimated average delay at vehicle-actuated traffic signals working with a speed-timed vehicle-extension period which, on average, approximates to (a) 4 seconds (b) 7 seconds? The maximum flows, saturation flows and maximum periods are as given below. In this example, both arms of phase 1 are identical and both arms of phase 2 are identical. The total lost time, L, is 10 seconds.

	Phase 1	*Phase* 2
Flow in vehicles/hour	400	600
Saturation flow in vehicles/hour	2000	2000
y values	0·2	0·3

The optimum cycle time with fixed-time signals would be

$$c_o = \frac{15+5}{1-0{\cdot}5} = 40 \text{ seconds}$$

The green times would be $g_1 = \frac{0{\cdot}2}{0{\cdot}5}(30) = 12$ seconds

$$\text{and } g_2 = \frac{0{\cdot}3}{0{\cdot}5}(30) = 18 \text{ seconds}$$

The steps involved in the calculation of delays using Tables 3, 4 and 5 are set out below.

	Delay with optimum settings		*Delay with maximum settings*	
	Phase 1	*Phase* 2	*Phase* 1	*Phase* 2
Maximum period (seconds)	—	—	30	45
G (seconds)	—	—	33	48
g (seconds)	12	18	31	46
c (seconds)	40	40	87	87
λ	0·300	0·450	0·356	0·530
x	0·667	0·667	0·561	0·567
M (vehicles/cycle)	5	7	10	15
Average delay (seconds)	16·1	11·3	24·4	15·3
Weighted mean delay (seconds)	13·3		18·9	

Thus, with a 4-second vehicle-extension period the mean delay would be expected to be 13 seconds; with a much longer vehicle-extension period, when the signals generally run to maximum, the delay would be about 19 seconds, and taking a value midway between the two for a 7-second vehicle-extension period gives 16 seconds' delay.

Example 9. Saturation flow falling off with green time

Observations of saturation flow at an intersection indicate that the rate of flow falls off on phase 1 in a similar way to that shown in Fig. 30 (p. 60), but on phase 2 the saturation flow is constant. The flows on phases 1 and 2 are 600 and

1000 vehicles per hour respectively. The saturation flows at points B and E in Fig. 30 for phase 1 are 3600 vehicles per hour (i.e. 1·0 vehicles per second) and 2340 vehicles per hour (i.e. 0·65 vehicles per second) respectively and for phase 2 the saturation flow is 2400 vehicles per hour. The values of α, β and γ are 7, 4 and 13 seconds respectively. Lost time due to starting delays on phase 2 is 2 seconds and the intergreen times between both phases are 5 seconds each. What are the optimum green times?

The procedure given in Appendix 6 is followed step by step below.

Step no.	First run	Second run	Third run
3	Select $G=23$ seconds	Select 17 seconds	Select 16 seconds
4	$\gamma_1=13$ seconds	7 seconds	6 seconds
5	$s_1=1{\cdot}0-\frac{13}{13}(0{\cdot}35)=0{\cdot}65$ vehicles/second	0·81 vehicles/second	0·84 vehicles/second
6	$g_1=\frac{20\,(1{\cdot}0)+17(0{\cdot}65)}{1{\cdot}30}=23{\cdot}9$ seconds	14·1 seconds	12·8 seconds
7	$l_1=23-23{\cdot}9=-0{\cdot}9$ seconds	2·9 seconds	3·2 seconds
8	$L=-0{\cdot}9+2+10-6=5{\cdot}1$ seconds	8·9 seconds	9·2 seconds
9	$y_1=\frac{600}{3600\,(0{\cdot}65)}=0{\cdot}256$	0·206	0·199
	$y_2=\frac{1000}{2400}=0{\cdot}417$	0·417	0·417
10	$Y=0{\cdot}256+0{\cdot}417=0{\cdot}673$	0·623	0·616
11	$g'_1=\frac{0{\cdot}256[5+5{\cdot}1(1{\cdot}173)]}{0{\cdot}673\,(0{\cdot}327)}=12{\cdot}8$ seconds	13·1 seconds	12·9 seconds
12	$G'=23+\frac{12{\cdot}8-23{\cdot}9}{2}=17{\cdot}4$ seconds	16·5 seconds	16·1 seconds

The value G' obtained from the third run is so close to the value selected for that run that no more runs are necessary. A value of 16 seconds can be accepted as the optimum G value for phase 1. The saturation flow is 0·84 vehicles/second, the lost time 3·2 seconds and the effective green time 12·8 seconds. The effective green time of phase 2 using the normal rules will be

$$g_2 = \frac{y_2}{y_1}(g_1) = \frac{0{\cdot}417}{0{\cdot}199}(12{\cdot}8) = 26{\cdot}8 \text{ seconds}$$

$$G_2 = 26{\cdot}8+2{\cdot}0 = 28{\cdot}8 \text{ seconds}$$

Thus, G_1 and G_2 should take the values 16 and 29 seconds respectively.

Example 10. *y* **values with left filter**

At a junction of the type shown in Fig. 33 (p. 63) the flows and saturation flows take the values shown in the table below. The left-turning traffic from arm A has an exclusive lane and is allowed to flow on a filter during the green period for arm C. The lost time for each traffic stream is 2 seconds and the intergreen times are 4 seconds each. What are the appropriate y values?

	Arm A		Arm B	Arm C	Arm D
	Left-turning	Straight ahead and right-turning			
Flow (vehicles/hour)	450	900	600	300	400
Saturation flow (vehicles/hour)	1000	3600	1800	2000	1600
Flow/saturation flow	0·45	0·25	0·33	0·15	0·25

The method used is that given on pp. 62–6 If the left-turning flow predominates

$$c_1 = \frac{1{\cdot}5\,(4+4-6+2+2)+5}{1-0{\cdot}45-0{\cdot}25}$$

$$= \frac{14}{0{\cdot}3} = 46{\cdot}7 \text{ seconds.}$$

If streams A or B, and C predominate

$$c_2 = \frac{1{\cdot}5\,(4+4+4-9+2+2+2)+5}{1-0{\cdot}33-0{\cdot}15-0{\cdot}25}$$

$$= \frac{18{\cdot}5}{0{\cdot}27} = 68{\cdot}5 \text{ seconds.}$$

Since c_2 is larger than c_1, arms B, C and D are the predominant ones and the y values are 0·15 (phase 1), 0·33 (phase 2) and 0·25 (phase 3). The value of Y is 0·73.

REFERENCES

(1) WEBSTER, F. V. Traffic signal settings. *Department of Scientific and Industrial Research, Road Research Technical Paper* No. 39. London, 1958 (H.M. Stationery Office).

(2) MINISTRY OF TRANSPORT. Advisory memorandum on urban traffic engineering techniques. London, 1965 (H.M. Stationery Office).

(3) MINISTRY OF TRANSPORT. Manual on roads in urban areas. London, 1966 (H.M. Stationery Office).

(4) MORGAN, J. T., and J. D. C. LITTLE. Synchronizing traffic signals for maximal bandwidth. *Ops Res.*, 1964, **12** (6), 896-912.

(5) MATSON, T. M., W. S. SMITH and F. W. HURD. Traffic engineering. London, 1955 (McGraw-Hill Book Co. Inc.).

(6) PETTERMAN, J. L. Timing progressive signal systems. *Traff. Engng*, 1947, **17**, Part I (February), 194-9, Part II (March) 242-9.

(7) RAUS, J. A method of computing offset patterns for multi-cycle signal systems. *Traff. Engng*, 1963, **33** (8), 25-8.

(8) BOWERS, D. A. Progressive timing for traffic signals. *Proc. Inst. Traff. Engrs*, 1947, 93-100.

(9) DAVIDSON, B. M. Design of signal systems by graphical solutions. *Traff. Engng*, 1960, **31** (2), 32-8.

(10) EVANS, H. K. Traffic engineering handbook. New Haven, 1950 (Institute of Traffic Engineers), 2nd edition. (A third edition was published in 1965.)

(11) NEWELL, G. F. The flow of highway traffic through a sequence of synchronized traffic signals. *Ops Res.*, 1960, **8** (3), 390-405.

(12) NEWELL, G. F. Synchronization of traffic lights for high flow. *Q.appl. Math.*, 1964, **21** (4), 315-24.

(13) DICK, A. C. A method for calculating vehicular delay at linked traffic signals. *Traff. Engng Control*, 1965, **7** (3), 224-9.

(14) FRANCIS, J. G. F., and R. S. LOTT. A simulation programme for linked traffic signals. *Proceedings of the 2nd International Symposium on the Theory of Road Traffic Flow, London, 1963.* Paris, 1965 (Organisation for Economic Co-operation and Development), pp. 257-9.

(15) HILLIER, J. A. The area traffic control experiment in Glasgow. *Proceedings of the 35th Annual Meeting of the Institute of Traffic Engineers, "A World Traffic Engineering Conference", held at Boston, Mass., 20th October, 1965.* (In the press.) Reprinted as "Glasgow's experiment in area traffic control". *Traff. Engng Control*, 1965/66, **7** (8), 502-9, (9), 569-71.

(16) WEBSTER, F. V. Possible developments in traffic signal techniques. *Proc. Aust. Road Res. Bd*, 1962, **1** (*Part* 1), 585-94: Discussion, 594-5.

(17) HILLIER, J. A. A review of developments in area traffic control. *Proc. Aust. Road Res. Bd*, 1962, **1** (*Part* 1), 416-26; Discussion, 426-9.

(18) COBBE, B. M. Development of traffic signals for area traffic control. *Traff. Engng Control*, 1964, **5** (9), 543-6.

(19) MINISTRY OF TRANSPORT. The traffic signs regulations and general directions, 1964. *Statutory Instruments*, 1964, No. 1857. London, 1964 (H.M. Stationery Office).

(20) BRITISH STANDARDS INSTITUTION. British Standard 505:1939. Road traffic control (electric) light signals. London, 1939 (British Standards Institution).

(21) TANNER, J. C. A theoretical analysis of delays at an uncontrolled intersection. *Biometrika*, 1962, **49** (1/2), 163-70.

(22) GARWOOD, F., and J. C. TANNER. Accident studies before and after road changes. *Publ. Wks munic. Services Congr.*, 1956, *Final Report*, 329-54; Discussion, 374-80.

(23) WEBSTER, F. V., and R. F. NEWBY. Research into the relative merits of roundabouts and traffic-signal-controlled intersections. *Proc. Instn civ. Engrs*, 1964, **27** (Jan.), 47-76; Discussion, 1965, **30** (April), 752-71.

(24) NATIONAL JOINT COMMITTEE ON UNIFORM TRAFFIC CONTROL DEVICES. Manual on uniform traffic control devices for streets and highways. Washington, D.C., 1961 (U.S. Department of Commerce, Bureau of Public Roads).

(25) COBBE, B. M. Mutual obligations of turning traffic and pedestrians at signal controlled intersections and appropriate indication thereof. *Sixth International Study Week in Traffic Engineering 10–14 September 1962, Salzburg, Austria.* London, 1962 (World Touring and Automobile Organisation).

(26) HIGHWAY RESEARCH BOARD. Channelization; the design of highway intersections at grade. *Special Report* 74. Washington, D.C., 1962 (National Research Council).

(27) FEUCHTINGER, M. Planning traffic intersections 1. Channelized intersections. *Traff. Engng Control*, 1961, **3** (5), 287-90.

(28) KORTE, J. W., P. A. MACKE and R. LAPIERRE. Design of traffic facilities, Part I: Intersections. (Gestaltung von Strassenverkehrsanlagen 1. Teil: Kreuzungsanlagen). *Forschungsberichte des Landes Nordrhein-Westfalen*, Nr. 610: Cologne and Opladen, 1959 (Westdeutsche Verlag).

(29) SMEED, R. J., and J. A. HILLIER. Traffic engineering research—some recent examples. *Proceedings of the Conference on Transportation Engineering, held at the Central Hall, Westminster, London, June 22nd–24th, 1965.* London, 1965 (Institution of Civil Engineers).

(30) HERZOG, A. Design of intersections in urban areas from the capacity point of view. *Sixth International Study Week in Traffic Engineering 10–14 September, 1962, Salzburg, Austria.* London, 1962 (World Touring and Automobile Organisation).

(31) CHRISTOFFERS, C. Practical experience in the design of high-capacity urban intersections. *Sixth International Study Week in Traffic Engineering 10–14 September, 1962, Salzburg, Austria.* London, 1962 (World Touring and Automobile Organisation).

(32) GULSTAD, E. Design of intersections in urban areas from the capacity point of view. *Sixth International Study Week in Traffic Engineering 10–14 September 1962, Salzburg, Austria.* London, 1962 (World Touring and Automobile Organisation).

(33) MOORE, R. L. Psychological factors of importance in traffic engineering. *International Study Week in Traffic Engineering 1–6 October 1956, Stresa, Italy.* London, 1956 (World Touring and Automobile Organisation).

(34) ROAD RESEARCH LABORATORY. A method of measuring saturation flow at traffic signals. *Department of Scientific and Industrial Research, Road Note* No. 34. London, 1963 (H.M. Stationery Office).

(35) DICK, A. C. Effect of gradients on saturation flow at traffic signals. *Traff. Engng Control*, 1963, **5** (5), 293-4.

(36) HOLROYD, E. M. Effect of motorcycles and pedal cycles on saturation flow at traffic signals. *Rds Rd Constr.*, 1963, **41** (490), 315-6.

(37) WEBSTER, F. V. Experiments on saturation flow of right-turning vehicles at traffic signals. *Traff. Engng Control*, 1964, **6** (7), 427-30, 434.

(38) WEBSTER, F. V. Traffic signals. *Traffic engineering practice.* London, 1963 (E. & F. N. Spon Ltd.), pp. 117-50.

(39) ARCHER, R. J. G., R. I. HALL and S. EILON. Effect of turning vehicles on traffic flow through a signal-controlled junction. *Traff. Engng Control*, 1963, **5** (5), 295-7.

(40) HILLIER, J. A., P. D. WHITING and J. G. WARDROP. The automatic delay computer. *Department of Scientific and Industrial Research, Road Research Laboratory, Research Note* No. RN/2291/JAH. PDW. JGW. Harmondsworth, 1954. (Unpublished.)

(41) MILLER, A. J. Settings for fixed-cycle traffic signals. *Opl Res. Q.*, 1963, **14** (4), 373-86.

(42) DICK, A. C. A nomogram for solution of the RRL traffic signal delay formula. *Traff. Engng Control*, 1964, **6** (2), 106-8.

(43) BERRY, D. S. Field measurement of delay at signalized intersections. *Proc. Highw. Res. Bd, Wash.*, 1956, **35**, 505-22.

(44) SMEED, R. J. Traffic flow. *Opl Res. Q.*, 1957, **8** (3), 115-23; Discussion, 142-8.

(45) SMEED, R. J. The traffic problem in towns. Manchester, 1961 (Manchester Statistical Society).

(46) WEBSTER, F. V. Greenford roundabout experiment. *Traff. Engng Control*, 1962, **2** (5), 266-71.

(47) MACE, D. G. W., P. B. ELLSON and M. J. F. JEANES. A congestion detector to reduce lock-ups in a roundabout. *Surveyor, Lond.*, 1965, **125** (3796), 25-7.

(48) BLACKMORE, F. C. Priority at roundabouts. *Traff. Engng Control*, 1963, **5** (2), 104-6.

(49) WEBSTER, F. V., and P. B. ELLSON. Traffic signals for high-speed roads. *Ministry of Transport, Road Research Technical Paper* No. 74. London, 1965 (H.M. Stationery Office).

INDEX

HM 909 Dd.127106. K40.5/66   S.O. Code No 55-421-56