~~Traffic~~ Signals

NCHRPR-073

~~TRAFFIC~~

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT **73**

~~IMPROVED CRITERIA FOR~~ TRAFFIC SIGNAL SYSTEMS ON URBAN ARTERIALS

HIGHWAY RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

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IMPROVED CRITERIA FOR TRAFFIC SIGNAL SYSTEMS ON URBAN ARTERIALS

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SUBJECT CLASSIFICATION:

TRAFFIC CONTROL AND OPERATIONS
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Bureau of Public Roads, United States Department of Transportation.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the U. S. Bureau of Public Roads. Individual fiscal agreements are executed annually by the Academy-Research Council, the Bureau of Public Roads, and participating state highway departments, members of the American Association of State Highway Officials.

This report was prepared by the contracting research agency. It has been reviewed by the appropriate Advisory Panel for clarity, documentation, and fulfillment of the contract. It has been accepted by the Highway Research Board and published in the interest of an effectual dissemination of findings and their application in the formulation of policies, procedures, and practices in the subject problem area.

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FOREWORD

By Staff

Highway Research Board

This report will be of interest to all traffic engineers, traffic control manufacturers, and operations research scientists responsible for the efficient timing of arterial traffic signal systems. By use of computer simulation techniques numerous methods of operating urban arterial signal systems were scientifically tested to determine the comparative effectiveness of timing methods. The research indicates that significant improvements in traffic operations may be achieved through application of the better signal timing methods. Eleven timing methods were analyzed and ranked in order from "best" to "worst." In the pilot study, non-rush-hour total delay was reduced nearly 40 percent and rush-hour delay was reduced almost 25 percent. For the practicing traffic engineer, a special chapter on applications is included, together with worksheets. A thorough search of the literature has been completed, and a bibliography containing 69 entries is presented in an appendix.

This report stems from NCHRP project 3-5/1 entitled "Improved Criteria for Designing and Timing Traffic Signal Systems." The initial research involved signal operations at the isolated intersection. The results of the isolated intersection phase have been published as *NCHRP Reports 3* and *32*. On completion of the isolated intersection phase of research, the NCHRP Advisory Panel for Traffic extended the research to include the study of signal timing methods on arterial highways.

The report comprehensively documents methods, results, interpretations, and applications of research on the development and closely controlled, scientific testing of the effectiveness of several advanced concepts for traffic signal system control on urban arterial streets. The results indicate the degree of significant improvement in traffic operations possible through application of advanced control methods.

The research agency's urban arterial and network simulation model, TRANS, was used to evaluate eleven alternative traffic signal operation test conditions employing various control concepts for a selected arterial street system in the city of Los Angeles. Subjected to tests were four strategic (fixed-time) control concepts applied in various combinations, one traffic-adjusted control concept, one experimental traffic-responsive concept, and one special mixed-cycle version of a strategic control concept. A total of 100 hours of traffic operation were simulated to produce statistically reliable results in conjunction with the effectiveness tests of alternatives.

The most influential strategic control concept tested was Webster's computational method for optimizing traffic signal cycles and splits. Three computer-assisted methods for formulating signal offset plans—the Yardeni time-space design model, the Little maximal bandwidth model, and the delay/difference-of-offset method—were thoroughly tested.

The traffic-adjusted concept of control tested was the commonly used cycle and offset selection mode. The experimental traffic-responsive control concept tested was the basic queue control mode previously studied for isolated intersections and reported in *NCHRP Report 32*. The mixed-cycle mode of signal operation, wherein a shorter signal cycle is employed at minor intersections along the arterial than is required at the critical intersection, was tested during the peak traffic period only.

The report contains a discussion of some important, immediate practical applications of the research findings, and outlines fruitful research areas that became evident as a result of this study. The report is well documented and includes in several appendices details on alternative strategic traffic signal timing plans, results of simulation tests, analysis of traffic-adjusted control systems, review of special signal system research projects, abstracts of selected papers and bibliographies on strategic signal techniques, platoon behavior, use of funnels and presignals, and traffic flow characteristics at intersections.

CONTENTS

ACKNOWLEDGMENTS

The research project reported herein was conducted by the traffic systems group of Planning Research Corporation (PRC), with Daniel L. Gerlough and Frederick A. Wagner, Jr., serving as Co-Principal Investigators. Frank C. Barnes made important contributions throughout the course of the project. Other key PRC personnel associated with the project included Nancy A. Bryant and David P. Stirling. Appreciation is extended to other members of the staff who assisted in technical aspects of the work.

Grateful acknowledgment is given to the Civil Engineering Systems Laboratory of the Massachusetts Institute of Technology and to International Business Machines Corporation for supplying computer programs that were tested as part of this research. Thanks also go to the Research Bureau, Department of Traffic, City of Los Angeles, for supplying information which facilitated selection of a pilot study arterial street system.

Finally, acknowledgment is made for the assistance of many individuals affiliated with governmental agencies, private industries, and universities who contributed pertinent information.

IMPROVED CRITERIA FOR TRAFFIC SIGNAL SYSTEMS ON URBAN ARTERIALS

SUMMARY

This research involved the development and comprehensive, closely controlled, scientific testing of several advanced concepts for operating traffic-signal systems on urban arterial streets. The results indicate the degree of significant improvement in traffic operation that is possible through application of advanced control methods.

Using established traffic-simulation methods, a total of 11 alternative signal-operation test conditions in a pilot study arterial system were thoroughly evaluated. Subjected to tests were: four strategic concepts applied in various combinations; one traffic-adjusted control concept, one experimental traffic-responsive concept; and one special version of a strategic concept.

The strategic concept that is most influential in improving traffic operations is Webster's straightforward computational method devised for optimizing traffic-signal cycles and splits. Three computer-assisted methods for formulating signal offset plans—the Yardeni Time-Space Design Model, the Little Maximal Bandwidth Model, and the delay/difference-of-offset method refined by the research agency—are also effective. All three offset methods significantly reduce delays in the arterial system when applied in combination with the Webster technique. The best strategic signal-control plan tested reduces total delay by 39 percent during the offpeak period and by 17 percent during the peak period.

The research indicated that the traffic-adjusted control concept, the commonly used cycle and offset selection mode, reduces delays in the pilot system by 39 percent under offpeak conditions and by 12 percent during the peak hour.

The experimental traffic-responsive control concept, the basic queue-control mode tested in prior research for isolated intersections, produces 20-percent and 22-percent reductions in delay during the offpeak and peak periods, respectively, in the signalized arterial system.

The special version of strategic (fixed-time) control, the mixed-cycle mode wherein the cycle length used at minor intersections is shorter than that used at the critical intersection, was the most effective peak-period technique tested. Total delay in the pilot system is reduced by 24 percent using this method during the peak hour.

Significant improvements in traffic operation on urban arterials can be achieved by applying the advanced strategic concepts studied. This can be accomplished without major capital expenditures for new control equipment. Comprehensive programs should immediately be organized to apply the effective new techniques for improving the operation of existing signal systems.

The research findings also contain useful information that is fundamental to cost-benefit analyses of major traffic-control improvement projects.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

INTRODUCTION

Research efforts on this project have been devoted to the development and controlled testing of alternative methods or concepts of traffic-signal control on urban arterials. This work was a logical extension of the research agency's earlier endeavors under the National Cooperative Highway Research Program (NCHRP). Therefore, it is important to briefly review the past work.

The research agency began its work on NCHRP Project 3-5, Improved Criteria for Designing and Timing Traffic Signal Systems, in July 1963. During the first two phases of research, extending through 1965, the scope was limited to the study of individual intersections.

The first year's accomplishments included: (1) a comprehensive state-of-the-art summary of individual intersection signalization and operation, documented by a lengthy bibliography and abstracts of selected papers; and (2) the formulation, programming, and preliminary testing of a multipurpose, microscopic, digital simulation model of traffic performance and control at individual intersections. This research was published in *NCHRP Report 3*.

In the next phase of research, the microscopic simulation model and other methods were applied to accomplish the following tasks: (1) study of a variety of measures of effectiveness of intersection performance, their interdependence, and their dependence on the mode of signal control; (2) development and programming of new concepts of individual signalized intersection control; (3) controlled testing and evaluation of alternative control concepts, using the simulation model; (4) execution of a comprehensive pilot field implementation study of the most promising new control concept, the basic queue-control mode; (5) special study of the relative effectiveness of various signal-phasing schemes incorporating protected left-turn intervals; and (6) a theoretically based special study to evaluate alternative policies of equalization of delay among intersection approaches. This research was published in *NCHRP Report 32*.

At the end of the first two phases, the research agency recommended that there was important, continuing research to be done in designing and timing traffic-signal systems by extending the general approach used to study individual intersections to the study of signalized arterials and networks.

RESEARCH APPROACH

The principal objective of this latest phase of research was to develop many alternative traffic-signal control concepts for arterial systems, and to pursue comprehensive controlled testing of the alternatives by means of an established urban arterial and network simulation model. More specifically, the technical approach consisted of the following tasks:

1. Conduct a thorough review of the literature covering traffic-signal systems to determine the scope and nature of knowledge being created and techniques being utilized by: (1) traffic-control improvement projects, (2) traffic-control research efforts, and (3) manufacturers or suppliers of traffic-control systems hardware components.
2. Conceive and/or formally define in functional terms several alternative control concepts for the operation of traffic-signal systems on arterial streets and networks. The concepts set forth were to include alternatives in the following categories: (1) existing fixed-time concepts of control, (2) strategic modifications of fixed-time concepts, (3) existing traffic-adjusted concepts, (4) experimental, fully traffic-responsive concepts of control, and possibly (5) advanced concepts that incorporate or integrate one or more of the foregoing concepts.
3. Modify the traffic network simulation program (TRANS), previously developed and applied by the research agency, to such an extent that it could serve as a valid model for pretesting the effectiveness of alternative control concepts, either existing or experimental.
4. Using the traffic network simulation model, conduct, analyze, and interpret comprehensive controlled tests of the effectiveness of the alternative control concepts set forth for a broad range of traffic-demand conditions on a street network representing a real-life situation.

CHAPTER TWO

FINDINGS

REVIEW OF RELATED LITERATURE AND ACTIVITIES

A substantial effort was devoted to acquiring an understanding of the current state-of-the-art of traffic-signal-systems research and applications engineering. Methods of information acquisition included a review of published literature, correspondence with individuals and organizations currently pursuing work in the subject area, and on-site visits for first-hand discussion about and observation of important traffic-control projects.

In reviewing published literature, an attempt was made to isolate works that pertain directly to the control of arterial and network systems. Furthermore, the search concentrated on relatively recent literature. The primary result of the literature search was a bibliography (Appendix G). Entries are listed under the following categories: (1) strategic and tactical techniques; (2) platoon behavior; (3) use of funnels and pre-signals; (4) traffic flow characteristics at intersections; and (5) general and miscellaneous.

It is important to differentiate here between strategic and tactical techniques of signal control:

1. Strategy: The science of planning and directing large-scale operations; specifically, of maneuvering into the most advantageous position prior to actual engagement.
2. Tactics: The science of arranging and maneuvering in action.

When applied to traffic-signal control, strategy implies the predetermination of one or more fixed-time signal-timing plans that are based on the best data at hand and each of which is intended to constitute the best plan for a certain time, day, or condition of traffic. Conventional multidial signal controllers make use of such strategic plans, usually selected on the basis of time of day. Noteworthy entries in the bibliography exemplifying the strategic approach include Brooks (*2*), Chang (*4*), Little (*18, 19*), Yardeni (*29*), Traffic Research Corporation (*71*), and Raus (*75*).

When applied to traffic-signal control, tactics implies instantaneous or short-term adjustments of traffic-signal operation based on on-line measurements of traffic characteristics. Tactical techniques can be applied independently to intersections in a system, or, alternatively, in some cases, they can be superimposed on predetermined, strategic timing plans to maintain some relationship between adjacent signals. Discussions of tactical techniques are found in Cobbe (*7*), Dunne and Potts (*10*), Hillier (*13*), Miller (*21*), Morris and Pak-Poy (*24*), and *NCHRP Report 32*. For the most part, these papers are concerned with the development, testing, or application of new, tactical concepts of control. Although they are omitted from the bibliography of current literature, the widely used standard semi-actuated, fully actuated, and volume-density signal controllers are also outstanding examples of the application of tactical techniques.

Another important traffic-signal-control concept widely used in the United States for arterial systems is the so-called traffic-adjusted or cycle-and-offset selection mode of control (for example, the PR or EC systems). In general, a traffic-adjusted control system can be considered to use a refined, strategic approach, wherein real-time selections are made from among a limited number of predetermined timing plans, based on on-line measurements of traffic flow characteristics. Additionally, however, it is possible to accomplish minor tactical modifications with traffic-adjusted systems by applying pedestrian actuation or minor side-street vehicle actuation features.

A small number of interesting papers were studied in greater detail and abstracts were prepared (see Appendix F).

The research agency corresponded with several U.S. manufacturers of traffic-control hardware, and contact was made by letter and personal discussions with representatives of European manufacturers attending the International Exhibition on Engineering for Road and Traffic Networks, held concurrently with the Fifth World Meeting of the International Road Federation. Not surprisingly, the manufacturers appear to be concentrating on equipment development, rather than on control techniques, and in general are reluctant to disclose, for competitive reasons, much detail about their most advanced developments. In the recent past, International Business Machines Corporation (IBM) has done a substantial amount of conceptual work in the course of developing digital control computer applications; representative entries appear in Appendix G. Most European manufacturers state explicitly that they are doing little technique development, but are depending on various ministries of transport to prepare specifications on which bids then are made. A notable exception is the Plessey Automation Group, in England, which is cooperating with the Road Research Laboratory on conceptual development work for the Glasgow, Scotland, project.

Additional important insight was obtained through the on-site visits and discussions with representatives of traffic-control-improvement projects being conducted in Glasgow, Scotland; London, England; Toronto, Canada; and San Jose, California. Technical notes summarizing the findings of these visits appear in Appendix E.

DEVELOPMENT OF ALTERNATIVE CONTROL CONCEPTS

Strategic Concepts

The objective of the strategic approach to signal timing along an arterial is to determine the most effective plan of cycles, splits, and offsets for all the intersections in the system for a given set of traffic conditions. All of the strategic methods found in the literature were thoroughly studied and finally reduced to four techniques that were considered to have undergone sufficient development to merit testing in this project, namely:

1. Webster Optimization of Cycle and Splits.
2. Yardeni Time-Space Design Model.
3. Little Maximal Bandwidth Model.
4. Delay/Difference-of-Offset Method.

Because not all of these methods yield complete plans of cycle, splits, and offsets, they must be applied in various combinations. For example, the Webster method is limited to determination of cycles and splits, whereas the Little model and the delay/difference-of-offset method yield offset plans only, given the cycle and splits. The Yardeni model, however, can be used to derive a complete plan, cycle, splits, and offsets.

Other strategic methods that were studied in detail but not carried forward to pilot testing were Little's (*18*) mixed-integer linear programming method, which tends to be unwieldy, and SIGOP (*71*), which is undergoing further development and refinement.

Webster Optimization of Cycle and Splits

First published in a classic paper in 1958 (*28*), the Webster method of determining optimum cycles and splits for fixed-time signals is rapidly becoming a standard method. It is believed that no strategic approach can proceed effectively without incorporating a technique for cycle and split determination that is at least as well refined as Webster's method. Webster used a combination of theory and computer simulation to deduce a formula for average delay per vehicle at a signalized intersection. From that formula, he proceeded to derive expressions for cycle length and splits that minimize total delay at the intersection. For a two-phase intersection,

$$C_o = \frac{1.5L + 5}{1 - Y_A - Y_B} \tag{1}$$

in which

C_o = optimum cycle length, sec;
L = total lost time per cycle, sec (i.e., the sum of lost time of phases A and B, due to starting delays and reduced flows during the yellow periods); and
Y_A and Y_B = the maximum ratios of single-lane flow to saturation flow for phases A and B.

The splits should be established such that

$$\frac{GE_A}{GE_B} = \frac{Y_A}{Y_B} \tag{2}$$

in which GE_A and GE_B = effective green times of phases A and B, sec. (Effective green equals green plus yellow, minus lost time for a given phase.)

In other words, the effective green portion (GE) of the total cycle is apportioned to phase A and phase B effective greens in accordance with their respective Y values. For purposes of calculation, the optimum split equation is restated:

$$GE_A = \frac{Y_A(GE)}{Y_A + Y_B} \tag{3}$$

in which GE = cycle − L.

Then, GE_B is obtained by subtraction:

$$GE_B = GE - GE_A \tag{4}$$

In the normal strategic approach for an arterial, the system cycle length would be dictated by the requirements of the critical intersection as computed by Webster. However, this often results in the use of cycle lengths at minor intersections that far exceed those required to serve the traffic demand. As described later in this report, an experiment was conducted to determine if the use of a system cycle length for preservation of fixed relationships between adjacent signals is a necessary policy.

Yardeni Time-Space Design Model

Yardeni (*29*) has formulated a method and prepared a computer program for determination of fixed-time traffic-signal settings on arterial streets. The program, written in FORTRAN IV for use on the IBM 7040/44 or 7090/94 computers, accommodates up to 30 intersections. Another version of the program is available, written in FORTRAN II for the IBM 1620 computer. The FORTRAN IV version features an option for automatic off-line plotting of speed-volume curves and time-space diagrams.

The program provides results that are usable directly in the field. However, it requires fairly detailed input information, including the following traffic and physical characteristics:

1. A speed-volume relationship for the arterial under consideration.
2. Block lengths and number of effective lanes between signalized intersections.
3. Minimum cross-street green-plus-yellow time (usually pedestrian crossing time).
4. Total traffic-volume rate on each main-street approach to every signalized intersection.
5. Highest lane volume rate on each cross-street approach.
6. Range of cycle lengths to be considered.
7. Maximum allowable difference in speeds of opposing directions of traffic.

In addition to these input data, a number of control parameter and subroutine combinations are selected by the user. These include the following parameters:

1. System constant algorithm control, which selects the method by which the space-periodicity constant is determined (i.e., minimax or least squares fit).
2. Cycle time convergence control.
3. Efficiency factor for converting from free flow to pulsed flow using the given speed-volume curve.
4. Plot control, where plotting output may be either suppressed or used.
5. Maximum number of additional feasible solutions.

6. Cycle time precision, which controls rounding off of cycle length computations.
7. Offset allocation control, which controls effect of opposing traffic volumes on the proportioning of the available given time (split). It also controls selection of offsets relative to through-band interference by several methods.
8. Lead-trail justification and phase control, which provides for lead or trial justification of offsets and provides for use of three-phase timing.
9. Lead-lag left-turn control, which provides for excess green to be assigned to either lead or lag position at each intersection.
10. Individual intersection weighting is also provided for if the user finds it necessary to assign special importance to a particular signal.

First trial values are suggested for most of these parameters. The significance of some of the options has not been fully explored yet. It appears that some familiarity with the program must be developed before proceeding with the use of different values for these control parameters.

The computer program initially determines green ratios (splits) from the volumes and minimum pedestrian crossing times given. Then a set of alternative "system constants" (space-periodicity constants) is developed by minimizing deviations from ideal midpoints of the green bands for all intersections simultaneously, taking into account signal spacing, mean velocity in each direction, cycle length, and green splits. Next, a minimum cycle-length/maximum speed pair is determined, within the restrictions imposed by the given speed-volume relationship and intersection approach volumes. Finally, signal offsets are computed that theoretically maximize possible through-band volumes.

The tabular output from the program provides offsets and splits for each cycle length used. The percentage through-bandwidth is shown for both directions, together with suggested speeds and feasible through volumes. The limiting intersections for each direction are identified. Most of the input data are also printed out. Sample output is shown in Figure 1.

Little Maximal Bandwidth Model

Little and his associates (*18, 19*) reported the development of a technique for strategic determination of signal offsets along an arterial, given cycle length, splits, certain traffic characteristics, and signal spacing. The objective of their method is to achieve either (1) maximum, equal through-bandwidths for traffic in both directions of the arterial, or (2) a maximum through-bandwidth in only one direction, provided, however, that bandwidth in the opposite direction must be greater than zero.

Little prepared a computer program for rapid, automatic implementation of the maximal bandwidth model for the IBM 1620 computer. With relatively small effort, the research agency recoded Little's program for operation on the IBM 7094. The program can accommodate up to 50 signals.

For equal bandwidths in both directions, it was shown that half-cycle synchronization of all signals along the street results in the maximal bandwidth. Using this terminology, half-cycle synchronization refers to that timing method whereby the center of the red interval at each intersection occurs either at, or one-half the cycle length after, some time reference point in the signal cycle.

It was then shown how offsets may be shifted to increase bandwidth in one direction with a corresponding reduction in bandwidth in the opposite direction. By use of the traffic volumes in each direction, together with the saturation flow headway, the program attempts to adjust the relative bandwidths according to platoon length in the two directions.

Input data required by the computer program include the following:

1. Number of signals considered.
2. Signal cycle length (constant for the entire system of intersections).
3. Average traffic volumes in each direction on the arterial.
4. Saturation flow headway (reciprocal of single-lane saturation flow rate).
5. Distances from the first signal to all other signals.
6. Duration of the red interval on the arterial at each signal.
7. Average speeds in each direction between each pair of signals.

In addition to listing all the input data, the program outputs the following information:

1. Inbound and outbound bandwidths (BIN and BOUT).
2. Number of the restricting signal (LTBST).
3. Possible through-band hourly flow rates inbound and outbound (PLATI and PLATO).
4. Offsets to the beginning of the green phase at each signal with respect to that of the restricting signal, shown as a fraction of cycle length (PHASE).
5. Offsets to the beginning of the green phase at each signal with respect to the center of the red phase of the restricting signal, in seconds (WMIN).
6. Position of the leading edge of the outbound through-band, at the first signal, in fractions of cycle length (ALHS).
7. Position of the trailing edge of the inbound through-band at the first signal, in fractions of cycle length (BRHS).
8. The times for traversing the system, inbound and outbound, within the through-band (B and A).

Sample output from the Little program, modified for operation on the IBM 7094, is shown in Figure 2.

Delay/Difference-of-Offset Method

Hillier (*13*) has reported a technique developed at the Road Research Laboratory (RRL) in England for strategic optimization of offsets in a fixed-time signal-timing plan for an artery or a closed network. In this method, the traffic flows, the common cycle length, and the apportionment of green at each signal are given, and it is assumed that the delay to traffic along any unidirectional link of the network depends solely on the difference between the offsets of the signals at each end of the link. Given a delay/difference-of-offset relationship for each link (i.e., a relationship that permits the determination of delay on the link for every possible difference-of-offset value), a procedure is described for combining links, configured either in series or parallel, to yield signal settings that minimize delay in the network.

```
                                                                      DATA SET   1.
                                                                      DATE  1.23.67.
                                                       LOS ANGELES CALIF

           TIME-SPACE DIAGRAM FOR PICO BOULEVARD
           =================

                                 FROM LA BREA AVENUE  E  TO GENESEE AVENUE  W

 VOLUME INDEX VS. SPEED FUNCTION
    1  2  3  4  5  6  7  8  9 10 11 12 13 14 15 16 17 18 19 20  VOLUME UNITS
   44 44 43 43 43 42 42 42 41 41 40 40 40 39 39 39 38 38 37 37  FT. PER SEC.
   VOLUME UNIT IS  50. VEHICLES PER HOUR PER LANE

 INPUT DATA

 DISTANCES,VOLUMES AND COMPUTED GREEN RATIOS
 1          1032.  718.  600.   0.462 0.374    0
    2485
 2          1029.  864.  400.   0.563 0.519    0
    1105
 3          1054.  850.  250.   0.678 0.630    0
    1305
 4           901.  953.  400.   0.530 0.544    0
    1008
 5           903.  935.  250.   0.644 0.652    C
    1186
 6           904.  916.  250.   0.644 0.647    0

      NO.   SYSTEM      SYSTEM        WORST            MAXIMUM            STUDENTS      IKW  SYK
            CONSTANT    BASE        DEVIATION      BAND   RATIOS           T TEST            CNTRL

        1  1397.5000  -1397.5000      0.         0.4624      0.3743       0.            1    5

        2  1273.9049  -1273.9350      0.0000     0.4624      0.37431092528.7500          2    5

        3  1167.5701  -1167.5702      0.0000     0.4624      0.37435505958.0000          1    5


   IH  II   EL1  EL2       C      LC   HC   MAX.VOLUME  MAX.SPEED   N   NVD  KS  NPLOT   E       F    FRX  LTJ   CM   CMN
    1   6  2.00 2.00      50.0    50   70   2232  1918   42   43    5    0    3    0     0.     1.00  5.00   3   5.00 5.00

      RESULTS
      =========
                   THRU BAND  INTERSECTIONS  SUGGEST  MAX.VOLUME  TOWARDS       FEASIBLE    LIMITING
                    PERCENT    LEAD  TRAIL    SPEED    DEMAND   INTERSECTION    VOLUME   INTERSECTIONS
    DIRECTION 1      40.7        1     4     38.9 27    1032.        1           1253.    0
    DIRECTION 2      30.5        4     1     38.9 27     718.        1            939.    0

  TOTAL (DIR.1&2)    71.2                    38.9       1750.                    2192.

                  SELECTED OFFSETS   GREEN SPLITS
        SIGNAL     SEC. PERCENT        SECONDS
        ------    --------------     -----------
           1        0.      0.       28 32  22 38
           2        0.3     0.5      34 26  31 29
           3       31.9    53.2      41 19  38 22
           4       58.4    97.3      32 28  33 27
           5       31.7    52.8      39 21  39 21
           6        2.0     3.4      39 21  39 21
    IMAGINARY BASE INTERSECTION IS LOCATED AT A DISTANCE OF        -0.0 FEET
    FROM FIRST INTERSECTION WITH GREEN SPLITS EQUAL TO THOSE
    OF SIGNAL  1  IN DIRECTION 1, AND OF SIGNAL  1  IN DIRECTION 2.

                         ---.----..-----...-----..----.---

   IH  II   EL1  EL2       C      LC   HC   MAX.VOLUME  MAX.SPEED   N   NVD  KS  NPLOT   E       F    FRX  LTJ   CM   CMN
    1   6  2.00 2.00      50.0    50   70   2232  1918   42   43    5    0    6    0     0.     1.00  5.00   3   5.00 5.00

      RESULTS
      =========
                   THRU BAND  INTERSECTIONS  SUGGEST  MAX.VOLUME  TOWARDS       FEASIBLE    LIMITING
                    PERCENT    LEAD  TRAIL    SPEED    DEMAND   INTERSECTION    VOLUME   INTERSECTIONS
    DIRECTION 1      17.7        2     6     37.6 26    1032.        1            610.    4  6
    DIRECTION 2       8.6        6     2     37.6 26     718.        1            296.    4  6

  TOTAL (DIR.1&2)    26.3                    37.6       1750.                     905.

                  SELECTED OFFSETS   GREEN SPLITS
        SIGNAL     SEC. PERCENT        SECONDS
        ------    --------------     -----------
           1        0.      0.       23 27  19 31
           2       16.1    32.2      28 22  26 24
           3       45.5    91.0      34 16  31 19
           4       23.7    47.3      26 24  27 23
           5       46.8    93.7      32 18  33 17
           6       15.3    30.6      32 18  32 18
    IMAGINARY BASE INTERSECTION IS LOCATED AT A DISTANCE OF         0.0 FEET
    FROM FIRST INTERSECTION WITH GREEN SPLITS EQUAL TO THOSE
    OF SIGNAL  1  IN DIRECTION 1, AND OF SIGNAL  1  IN DIRECTION 2.

                         ---.----..-----...-----..----.---

    JOB WELL CONE. GO TO ANOTHER STREET
```

Figure 1. Sample output of Yardeni program.

NSIG	CYCLE	HEDWY	****	VOLIN	VOLOT	PLATI	PLATO	****	BIN	BOUT	LTBST
6	60.	2.030		953.	1054.	146.	798.		5.	27.	2

NO. SIG	X	SAVE	WMIN	PHASE
1	0.	33.0	23.3	0.113
2	2485.	23.4	11.7	0.
3	3590.	21.6	21.8	0.184
4	48950.	23.4	16.9	0.086
5	5903.	21.6	18.3	0.125
6	7089.	21.6	41.4	0.510

ALHS	BRHS	B	A
0.388	2.612	2.302	2.302

Figure 2. Sample output of Little program.

This offset optimization technique has perhaps the soundest logical base of the methods under study, because delay is taken under direct consideration and systematically minimized. The greatest difficulty in applying the technique is the determination of the delay/difference-of-offset relationship for each link in the network. It is impractical to empirically determine delay as a function of difference-of-offset link by link every time a new network is considered; this would require a series of field observations of delay for the entire range of offset values for every link. Obviously, an intolerable amount of field measurement would be required, and the process would disrupt normal operation in the network. Consequently, a practical prerequisite for application of the Road Research Laboratory technique was a generalized method for deriving the relationship between delay and difference-of-offset, given the unique characteristics of a specific link. The RRL and General Motors have been jointly pursuing research on this matter.

As one phase of this project, a theoretical method for deriving delay/difference-of-offset relationships was developed for use in conjunction with the RRL offset optimization technique. The derivation of this method appears in Appendix A. To rapidly compute the delay/difference-of-offset relationship for any link, given its pertinent geometrics, signal timing, and traffic characteristics, a computer program was prepared. The program facilitates implementation of the RRL technique for determining optimal offsets. A detailed description of the computer program also appears in Appendix A.

Traffic-Adjusted Concept

In general, a traffic-adjusted control system selects one of a limited number of strategic control schemes, based on counts of vehicles at a few selected locations. Minor tactical modifications are permitted at each intersection within the limits set by the strategic scheme in use.

Traffic-adjusted control is believed by many to be capable of providing a level of service above that of the normal multidial fixed-time coordinated control systems. Sampling detectors placed on the arterial street provide feedback of information about level of traffic demand to a master controller, which contains a set of strategic signal-timing schemes. One of these schemes is selected by comparing the data received from the sampling detectors with the levels of the control parameters set on the master controller by the traffic engineer. By means of either direct wire or radio communication links, the master controller transmits signals to the local intersection controllers, causing the selected strategic timing scheme to be put into effect. The change from one timing scheme to another is carried out in increments to avoid disrupting the progressive flow of traffic.

The local controller changes cycle lengths, splits, and offsets in response to commands received from the master controller. Certain fixed periods, such as yellow and all-red times and pedestrian intervals, are set on the local controller by the traffic engineer. It is also possible to provide for vehicle or pedestrian actuation on minor side streets, if desired. Actuations from the side-street detectors are used only at the local controller; they are not transmitted to the master controller.

Appendix D gives a detailed description of commercial traffic-adjusted control equipment, and it sets forth mathematical relationships by which a traffic-adjusted control system may be simulated.

Experimental Traffic-Responsive Concept

The experimental traffic-responsive concept tested in this project, the basic queue-control mode, is the same one developed and tested for individual intersection control in earlier NCHRP work by the research agency. Its functional description appears in *NCHRP Report 32,* and is repeated here. It is noteworthy that the basic queue-control concept was the most promising of any experimental mode tested in the previous single-intersection work, and was the one successfully implemented in a full-scale field experiment.

With this tactical control technique, signal timing is determined on a phase-by-phase basis. The duration of the green for each phase is dependent on queue lengths existing at the beginning of the phase. It is set equal to the time required, on the average, to discharge the longest queue existing at the beginning of the phase.

Detection

One queue-length detector is provided for each lane. The number of vehicles in queue in each lane of each approach to the intersection is measured continuously. No averaging of measurements is involved.

Control Logic

When it is time to begin green for a given phase, the period of time required to discharge the existing queue on each of the lanes of that phase is estimated as follows:

1. Assuming it is time to begin phase A green, compute for each phase A lane:

$$A_l = \frac{Q_l}{S_l} + L_l \qquad (5)$$

in which

A_l = estimated green period required to discharge the queue on lane l, in seconds;
Q_l = queue length currently existing on lane l, in vehicles;
S_l = saturation flow rate for lane l, in vehicles per second, a preset constant; and
L_l = lost time for lane l, in seconds, a preset constant.

2. Select the largest A_l. Call this A.
3. Compare A with minimum and maximum constraints.

If $A \leq A_{min}$, set phase A green $= A_{min}$ seconds; if $A \geq A_{max}$, set phase A green $= A_{max}$ seconds; if $A_{min} < A < A_{max}$, set phase A green $= A$ seconds, in which

A_{min} = minimum phase A green, in seconds, a preset constant; and
A_{max} = maximum phase A green, in seconds, a preset constant.

4. When it is time to begin phase B green, repeat steps 1 to 3, for the phase B lanes, to determine the required phase B green duration.
5. Yellow intervals are preset fixed intervals.

Controller Settings Required

The following controller settings are required:

1. Minimum phase A green.
2. Maximum phase A green.
3. Phase A yellow.
4. Minimum phase B green.
5. Maximum phase B green.
6. Phase B yellow.
7. Saturation flow rate for each lane.
8. Lost time for each lane.

APPLICATION TO A PILOT STUDY ARTERIAL SYSTEM

A section of signalized arterial street in the city of Los Angeles, California, was selected for the purpose of conducting comprehensive tests and comparative evaluation of the various alternative concepts of traffic-signal operation.

Description of Arterial System

The selected arterial system lies approximately 5 mi west of the Los Angeles central business district. Development in the area consists mainly of mixed, light commercial activities along the arterial, Pico Boulevard, and on the principal cross street, La Brea Avenue. Single-family dwellings are found on the other cross streets. A map of the test area (Fig. 3) shows the 6,100-ft section of Pico Boulevard passing through six signalized intersections with cross streets. In addition to the major north-south cross street, La Brea, five secondary signalized streets cross Pico Boulevard: Redondo, Cochran, Hauser, Curson, and Genesee. Spacing between the signalized intersections along Pico is relatively uniform, ranging from approximately 1,000 ft between Hauser and Curson to nearly 1,500 ft between La Brea and Redondo.

For purposes of ease of identification of street segments between signalized intersections (links) and to establish a formal network diagram for use in the computer simulation, a link diagram (Fig. 4) of the arterial system was prepared that includes only the signalized streets. Note that the system consists not only of the Pico street segments, but also the cross-street approaches to the signalized intersections. Other intervening minor cross streets are controlled by stop signs at their intersections with Pico; these were not included in the formal system that was simulated and analyzed.

Along the entire length of the system, Pico Boulevard is 70 ft wide, with painted median channelization. Traffic flows both eastbound and westbound in three lanes plus in a left-turn lane at each of the signalized intersection approaches. Storage capacity of the left-turn lanes averages six vehicles. Light-to-moderate parking narrows Pico to two moving lanes in each direction at midblock points but does not generally interfere with free-flow operation.

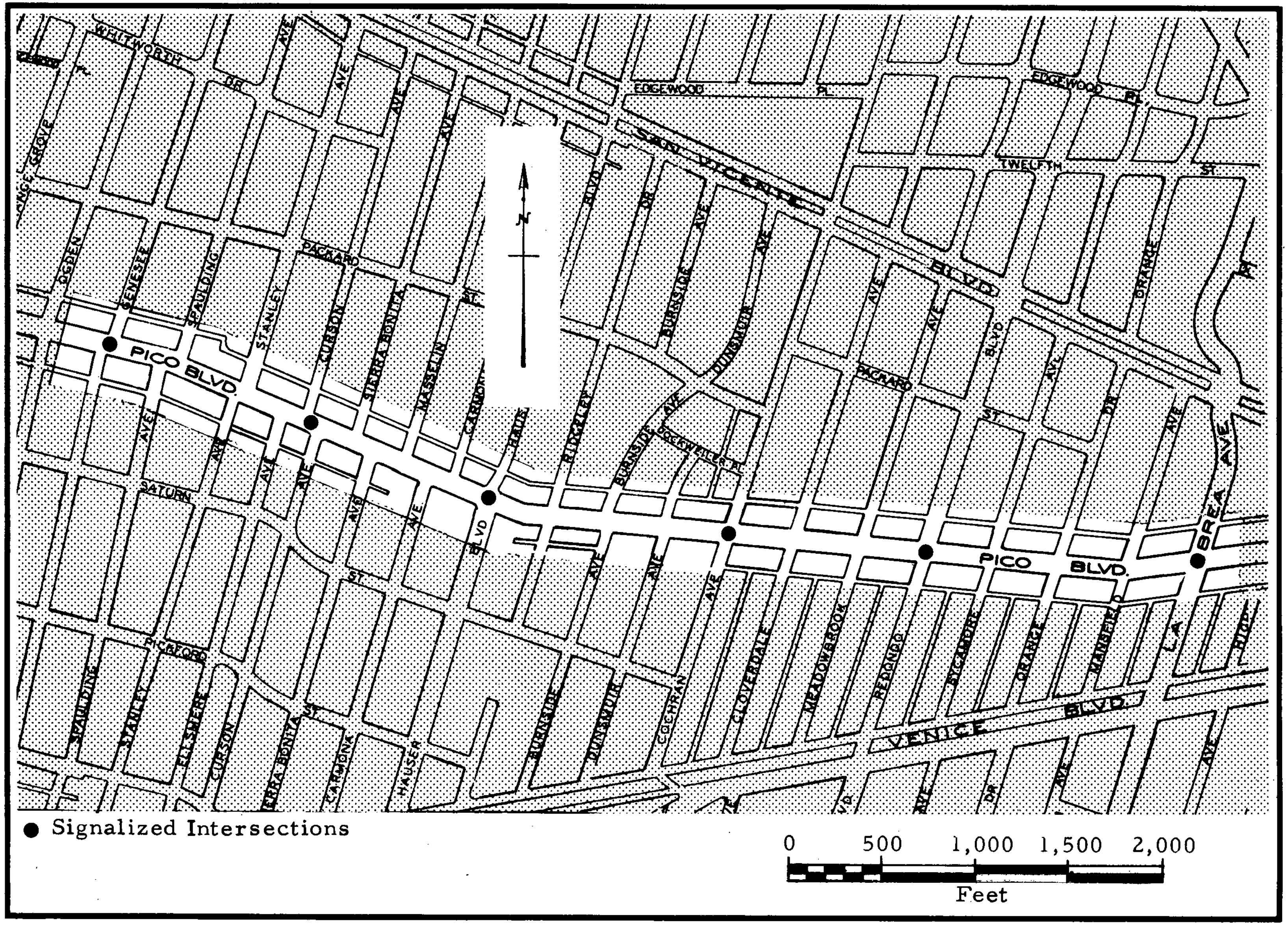

Figure 3. Section of Pico Boulevard used as test area.

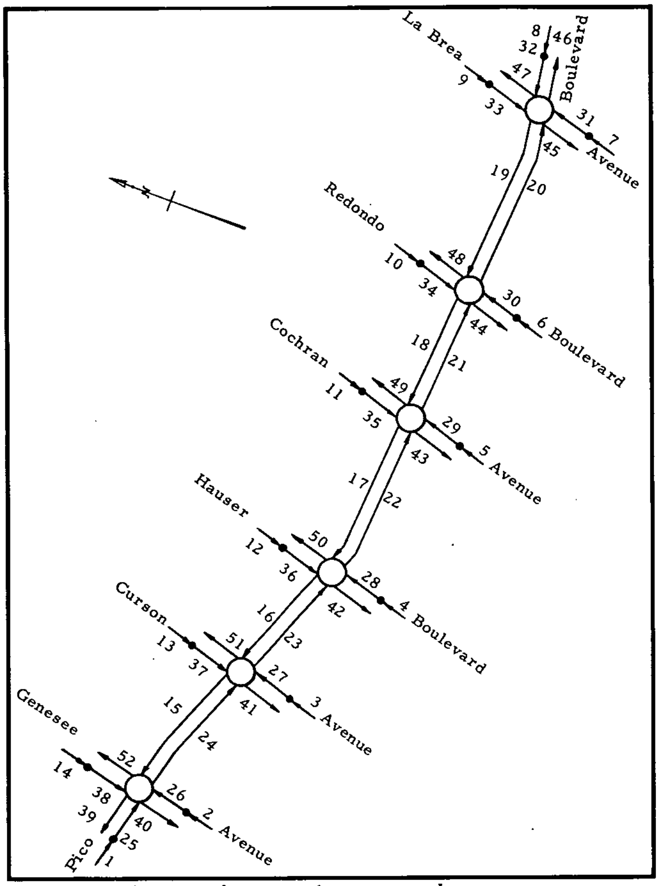

Figure 4. Schematic diagram of test network.

La Brea Avenue also has painted median channelization providing for three lanes plus left-turn lanes with storage capacity for 10 vehicles on its approaches to the Pico intersection. Parking is prohibited on the street during peak periods, and moderate midblock parking during the offpeak does not interfere with operation at the signalized intersection.

Three of the secondary cross streets—Redondo, Cochran, and Hauser—are marked for one lane of traffic, with midblock curbside parking permitted throughout the day. However, parking prohibition on the approaches to Pico results in two-lane operation at the intersections.

The remaining two cross streets—Curson and Genesee—are narrower, lighter-traffic-volume streets than the others, and they operate with single-lane approaches at the Pico intersections.

Traffic signals in the arterial system are currently operated by single-dial controllers. A 60-sec cycle and fixed splits and offsets are used throughout the day. Existing signal timing is given in Table 1.

Traffic Data Collection

Empirical traffic data were collected in the Pico arterial system for three purposes: (1) for input data necessary to exercise the various alternative strategic signal-timing determination programs discussed previously; (2) for input to the traffic simulation model; and (3) to evaluate the validity, or level of realism, of the traffic simulation model in representing existing traffic operations in the system.

Fortunately, it was possible to coordinate the field studies with the data requirements of a project being conducted for the U.S. Bureau of Public Roads in which the traffic simulation model was being comprehensively tested and further refined. Consequently, the empirical work was more extensive and resulted in greater statistical reliability than would have been possible otherwise.

It is important to note that the measurements of system operation encompassed both Pico Boulevard and all the specialized cross-street approaches. See Figure 4; link numbers 15 through 38 were included in the analysis.

The most important traffic-data-collection effort was the simultaneous acquisition of aerial photographic data, instrumented floating-car recordings of speed and delay characteristics, and manual counts to determine total volumes, turning movement probabilities, and pedestrian flow levels at the signalized intersections in the system. These data were acquired on five separate weekdays during an offpeak hour (2:30-3:30 PM) and a peak hour (4:30-5:30 PM). Supplementary data were collected to determine lane distribution characteristics, saturation flow rates, and left-turn gap acceptance distributions at the signalized intersections.

The aerial photographs were used to obtain successive observations of the number of vehicles contained on each link in the system. Averaged over each 1-hr study period and totaled for the network, these data, coupled with traffic-volume information, yielded fundamental measures of operational effectiveness, such as total travel time, total delay, and average speed.* The extensive traffic-volume counts enabled reliable computations of total vehicle-miles of travel, which is considered to be a primary measure of the magnitude of system utilization. Floating-car test runs, most importantly, gave reliable values for average running speeds between signalized intersections in the system (31 mph during both offpeak and peak periods), and also provided a reasonableness check against characteristics measured by aerial photography.

Table 2 is a summary of total travel time, total vehicle-miles, and average speed values from the field studies. Tables 3 and 4, for the offpeak and peak hours, respectively, give important traffic characteristics input data.

Summary of Signal Operation Test Conditions

A total of 11 alternative plans or concepts for operating the signals in the Pico arterial system were formulated for testing by simulation. Included were eight fixed-cycle strategic concepts, one traffic-adjusted concept, one experimental traffic-responsive concept, and one mixed-cycle strategic concept.

Strategic Concepts

The four methods of formulating strategic signal-timing plans described previously—Webster, Yardeni, Little, and

* Detailed discussion of the theory of measuring operational effectiveness using aerial photography is found elsewhere (*76*).

TABLE 1

SUMMARY OF EXISTING STRATEGIC SIGNAL-TIMING PLAN, PICO ARTERIAL SYSTEM

CROSS STREET	PICO OFFSETS (SEC)		PICO GREEN AND YELLOW (SEC)		CROSS-STREET GREEN AND YELLOW (SEC)	
	WB	EB	REF.	DURATION	REF.	DURATION
La Brea			6	28	34	32
	26	34				
Redondo			32	36	8	24
	24	36				
Cochran			56	38	34	22
	42	18				
Hauser			38	36	14	24
	56	4				
Curson			34	36	10	24
	26	34				
Genesee			0	30	30	30

TABLE 2

SUMMARY OF FIELD-STUDY RESULTS UNDER EXISTING SIGNAL-OPERATION CONDITIONS

STUDY DAY	2:30-3:30 PM TOTAL TRAVEL TIME (VEH-SEC)	TOTAL VEH-MILES	AVERAGE SPEED (MPH)	4:30-5:30 PM TOTAL TRAVEL TIME (VEH-SEC)	TOTAL VEH-MILES	AVERAGE SPEED (MPH)
Thur. 4/27/67	374,400	2,044	19.7	548,650	2,782	18.3
Mon. 5/1/67	381,520	2,095	19.8	542,810	2,842	18.8
Tues. 5/2/67	378,300	2,104	20.0	594,720	2,843	17.2
Wed. 5/3/67	380,160	2,204	20.9	580,680	2,944	18.3
Fri. 5/5/67	393,120	2,152	19.7	634,680	3,062	17.4
Average	381,500	2,120	20.0	580,310	2,895	18.0

delay/difference-of-offset—were applied in various combinations to derive seven modified plans as alternatives to the existing timing plan. The methods were exercised using both the offpeak period and peak period traffic characteristics data; so, in actuality, 14 modified plans resulted—7 for offpeak conditions and 7 for peak conditions. The alternative, strategic test conditions are:

1. Test condition 1: Existing signal timing.
2. Test condition 2a: Yardeni splits—Yardeni offsets.
3. Test condition 2b: Existing splits—Yardeni offsets.
4. Test condition 2c: Webster splits—Yardeni offsets.
5. Test condition 3a: Existing splits—Little offsets.
6. Test condition 3b: Webster splits—Little offsets.
7. Test condition 4a: Existing splits—delay/difference offsets.
8. Test condition 4b: Webster splits—delay/difference offsets.

In the case of the peak-period plans, all the alternatives used a system cycle length of 60 sec. Exercised for the offpeak period, test conditions 2c, 3b, and 4b used 40-sec cycle lengths; the remainder were 60 sec. Appendix B gives more detailed descriptions of the alternative strategic concepts, including time-space diagrams.

Tables 5 and 6 give comparative summaries of signal settings derived by exercising the various strategic concepts for the offpeak-period and peak-period conditions, respectively. Note that the alternative techniques did indeed result in significantly different timing plans. Findings with respect to the operational effectiveness of these alternatives follow in "Results of Effectiveness Tests."

Other Control Concepts

In addition to the wide range of strategic concepts tested, three other test conditions were investigated:

TABLE 3

IMPORTANT TRAFFIC CHARACTERISTICS INPUT DATA, 2:30 TO 3:30 PM, AVERAGED FOR THE FIVE STUDY DAYS

NET-WORK LINK NO.	VEH. FLOW RATE (VPH)[a]	NET-CHANGE RATE (VPH)[b]	PED. FLOW RATE (PPH)[c]	PROBABILITIES					
				TURNING			LANE DISTRIBUTION		
				STRAIGHT	LEFT	RIGHT	LANE 1	LANE 2	LANE 3
15	653	−1	15	0.94	0.02	0.04	0.49	0.49	0.02
16	640	−29	12	0.96	0.02	0.02	0.48	0.48	0.04
17	635	+35	29	0.88	0.07	0.05	0.42	0.51	0.07
18	616	−77	34	0.89	0.04	0.07	0.48	0.44	0.08
19	657	+40	16	0.89	0.07	0.04	0.47	0.45	0.08
20	674	−26	24	0.69	0.17	0.14	0.43	0.31	0.26
21	695	+16	18	0.88	0.05	0.07	0.44	0.45	0.11
22	658	−12	17	0.93	0.04	0.03	0.49	0.46	0.05
23	692	+13	27	0.86	0.06	0.08	0.48	0.42	0.10
24	662	−14	21	0.96	0.02	0.02	0.44	0.54	0.02
25	662	0	11	0.95	0.03	0.02	0.49	0.51	—
26	54	0	15	0.33	0.27	0.40	1.00	—	—
27	42	0	11	0.13	0.17	0.70	1.00	—	—
28	270	0	42	0.64	0.18	0.18	0.57	0.43	—
29	100	0	23	0.63	0.12	0.25	0.62	0.38	—
30	314	0	32	0.63	0.18	0.19	0.55	0.45	—
31	1167	0	31	0.75	0.11	0.14	0.42	0.32	0.26
32	593	0	17	0.72	0.24	0.04	0.59	0.36	0.05
33	1157	0	34	0.89	0.06	0.05	0.45	0.42	0.13
34	332	0	60	0.75	0.09	0.16	0.48	0.52	—
35	164	0	36	0.52	0.26	0.22	0.66	0.34	—
36	306	0	21	0.70	0.10	0.20	0.62	0.38	—
37	58	0	7	0.21	0.20	0.59	1.00	—	—
38	70	0	17	0.38	0.33	0.29	1.00	—	—

[a] Traffic flow rate at head of link.
[b] Net-change rate = Flow rate at head of link minus flow rate at tail of link.
[c] Pedestrian flow rate in signalized intersection crosswalk at head of link.

TABLE 4

IMPORTANT TRAFFIC CHARACTERISTICS INPUT DATA, 4:30 TO 5:30 PM, AVERAGED FOR THE FIVE STUDY DAYS

NET-WORK LINK NO.	VEH. FLOW RATE (VPH)[a]	NET-CHANGE RATE (VPH)[b]	PED. FLOW RATE (PPH)[c]	PROBABILITIES					
				TURNING			LANE DISTRIBUTION		
				STRAIGHT	LEFT	RIGHT	LANE 1	LANE 2	LANE 3
15	855	−46	10	0.96	0.02	0.02	0.51	0.45	0.04
16	869	−35	14	0.96	0.02	0.02	0.51	0.46	0.03
17	883	−41	28	0.88	0.08	0.04	0.50	0.44	0.06
18	935	−15	24	0.90	0.03	0.07	0.49	0.42	0.09
19	905	−156	20	0.89	0.09	0.02	0.52	0.42	0.06
20	954	+87	31	0.75	0.13	0.12	0.41	0.36	0.23
21	846	−26	41	0.86	0.05	0.09	0.44	0.45	0.11
22	815	+11	12	0.93	0.04	0.03	0.49	0.47	0.04
23	834	+4	20	0.84	0.06	0.10	0.41	0.50	0.09
24	808	−15	19	0.96	0.02	0.02	0.46	0.52	0.02
25	801	0	12	0.96	0.02	0.02	0.45	0.53	0.02
26	71	0	8	0.30	0.30	0.40	1.00	—	—
27	51	0	14	0.24	0.18	0.58	1.00	—	—
28	421	0	20	0.66	0.16	0.18	0.39	0.61	—
29	177	0	13	0.64	0.12	0.24	0.66	0.34	—
30	520	0	40	0.65	0.14	0.21	0.36	0.64	—
31	1677	0	50	0.78	0.09	0.13	0.47	0.34	0.19
32	1032	0	23	0.81	0.16	0.03	0.54	0.40	0.06
33	1780	0	30	0.92	0.04	0.04	0.41	0.41	0.18
34	565	0	70	0.82	0.05	0.13	0.37	0.63	—
35	366	0	19	0.65	0.19	0.16	0.64	0.36	—
36	473	0	23	0.80	0.06	0.14	0.70	0.30	—
37	209	0	13	0.62	0.10	0.28	1.00	—	—
38	153	0	9	0.59	0.20	0.21	1.00	—	—

[a] Traffic flow rate at head of link.
[b] Net-change rate = Flow rate at head of link minus flow rate at tail of link.
[c] Pedestrian flow rate in signalized intersection crosswalk at head of link.

TABLE 5

COMPARATIVE SUMMARY OF OFFPEAK-PERIOD STRATEGIC TIMING PLANS

	TEST CONDITION							
CROSS STREET	1	2A	2B	2C	3A	3B	4A	4B
	(a) REFERENCE OF START OF PICO GREEN INTERVAL (SEC)							
La Brea	0	0	0	0	0	0	0	0
Redondo	26	38	38	20	30	0	20	2
Cochran	50	6	8	30	0	20	50	18
Hauser	32	40	40	26	30	0	22	6
Curson	28	4	6	8	30	20	54	24
Genesee	54	56	56	16	0	20	26	10
	(b) DURATION OF PICO GREEN PLUS YELLOW (SEC)							
La Brea	28	24	28	18	28	18	28	18
Redondo	36	36	36	24	36	24	36	24
Cochran	38	36	38	24	38	24	38	24
Hauser	36	36	36	24	36	24	36	24
Curson	36	36	36	24	36	24	36	24
Genesee	30	36	30	24	30	24	30	24
	(c) SYSTEM CYCLE LENGTH (SEC)							
	60	60	60	40	60	40	60	40

TABLE 6

COMPARATIVE SUMMARY OF PEAK-PERIOD STRATEGIC TIMING PLANS

	TEST CONDITION							
CROSS STREET	1	2A	2B	2C	3A	3B	4A	4B
	(a) REFERENCE OF START OF PICO GREEN INTERVAL (SEC)							
La Brea	0	0	0	0	0	0	0	0
Redondo	26	26	30	26	30	30	22	26
Cochran	50	58	2	58	0	0	52	54
Hauser	32	26	26	26	30	30	22	28
Curson	28	0	0	0	26	0	54	56
Genesee	54	30	28	30	58	30	24	26
	(b) DURATION OF PICO GREEN PLUS YELLOW (SEC)							
La Brea	28	24	28	28	26	28	28	28
Redondo	36	34	36	34	36	34	36	34
Cochran	38	40	38	40	38	40	38	40
Hauser	36	34	36	36	36	36	36	36
Curson	36	38	36	40	36	40	36	40
Genesee	30	42	30	44	30	44	30	44
	(c) SYSTEM CYCLE LENGTH (SEC)							
	60	60	60	60	60	60	60	60

9. Test condition 5: Traffic-adjusted concept, cycle and offset selection mode.
10. Test condition 6: Experimental traffic-responsive concept, basic queue-control mode.
11. Test condition 7: Special strategic concept, mixed-cycle mode.

With respect to the traffic-adjusted concept, it was determined that traffic-flow fluctuations during the offpeak period tested are not great enough to stimulate changes in signal timing during the period. Therefore, the traffic-adjusted concept used the best of the offpeak strategic plans. During the peak hour, however, it was found that the traffic-adjusted concept brought three different strategic plans into play as traffic demands fluctuated. Cycle lengths used during the peak hours included 50, 64, and 70 sec. Different offset plans associated with the three cycle lengths were determined by the delay/difference-of-offset method.

The experimental traffic-responsive concept, basic queue-control mode, operated signals at each of the six intersections independently, i.e., without maintenance of a constant system cycle length. Individual signal phase durations, and in turn the cycle lengths, fluctuated substantially in the simulation tests of this mode of control. In correspondence to the findings of the single-intersection control experiments reported in *NCHRP Report 32*, the basic queue-control mode produces more widely fluctuating cycle-to-cycle signal operation during periods of heavy traffic demand than during offpeak periods. This finding is given in Table 7, which summarizes the range of phase durations produced during the simulation tests of basic queue control.

The special version of a strategic control concept, the mixed-cycle mode, was the last alternative tested. Application of the Webster method for peak-period traffic in the Pico arterial system indicated that at five of the six intersections a 40-sec cycle length is sufficient to accommodate

TABLE 7

RANGES OF INDIVIDUAL SIGNAL PHASE DURATIONS RESULTING FROM OPERATION OF THE BASIC QUEUE-CONTROL CONCEPT

	PICO BOULEVARD GREEN PLUS YELLOW (SEC)		CROSS STREET GREEN PLUS YELLOW (SEC)	
CROSS STREET	MINIMUM	MAXIMUM	MINIMUM	MAXIMUM
	(a) PEAK PERIOD (4:30-5:30 PM)			
La Brea	18	36	22	36
Redondo	16	26	16	18
Cochran	18	28	16	16
Hauser	16	26	16	20
Curson	18	28	16	16
Genesee	16	26	16	16
	(b) OFFPEAK PERIOD (2:30-3:30 PM)			
La Brea	18	20	22	24
Redondo	16	26	16	18
Cochran	18	20	16	16
Hauser	16	24	16	20
Curson	18	22	16	16
Genesee	16	20	16	16

traffic demand, but at the critical intersection of Pico and La Brea a 60-sec cycle is required. In the previously described strategic plans, the system cycle length was dictated by requirements of the critical intersection. For test condition 7, however, the critical intersection cycle was set to 60 sec, and the other five intersections were allowed to operate with 40-sec cycles. All splits were determined by the Webster method, and an offset plan for the five 40-sec cycle intersections was conceived by the delay/difference-of-offset technique. This special strategic concept was tested only for the peak-period conditions.

TRAFFIC SIMULATION MODEL

TRANS III, the traffic arterial and network simulation program, was used for executing comprehensive, controlled tests of the operational effectiveness of the traffic-signal-control-concept alternatives. TRANS III is the third version of the computer simulation program resulting from a U.S. Bureau of Public Roads project completed by the research agency in December 1966, in which the simulation model was refined and subjected to calibration, validation, and sensitivity testing. The model is described in detail and thoroughly documented in a report available through the Clearinghouse for Federal Scientific and Technical Information (*76*). An earlier version of TRANS is discussed in *NCHRP Report 32,* where it was used in conjunction with a study of various signal phasing schemes.

Because lengthy reports concerning the model and its applications are available elsewhere, the subsequent discussion is limited to a brief chronology of its development and the modifications required for one phase of this project.

Development of the TRANS III Model

The original TRANS model was developed in 1962 under contract with the District of Columbia Department of Highways and Traffic, with participation of the U.S. Bureau of Public Roads. The model was designed flexibly to permit its application to many sizes and configurations of signalized street networks. Its objective was to provide traffic engineers with a tool with which to assess the effects of varying traffic flows, traffic regulation and control characteristics, and geometric designs in networks of signalized streets. The model was termed "macroscopic" in that it used a unit-block concept wherein vehicles are aggregated for movement between signalized intersections. In discharging a queue at a traffic signal, however, vehicles are launched individually subject to the constraints of interaction with the control system and with other vehicles. Simulation is carried out in discrete time intervals of t-seconds. The program in its original form was especially coarse, using $t = 5$ sec. The initial testing of TRANS was for a large, complex street network in Washington, D.C., consisting of 80 signalized intersections.

In conjunction with a project sponsored by the Bureau of Public Roads, and for applications in previous NCHRP work, the following model refinements were incorporated:

1. The simulation scanning cycle was generalized, permitting t as small as 2 sec, to reduce the original coarseness of the model.
2. Left-turn interference logic was improved by incorporating probabilistic features.
3. An intralink volume change rate scheme was devised to account for gains or losses of traffic volume between signalized intersections.
4. A launch-table modifier was developed to permit the use of varying saturation flow rates on individual intersection approaches.

The version of the simulation model infused with these refinements was titled TRANS II.

Subsequently, in further work for the Bureau of Public Roads, the research agency integrated additional refinements:

1. The input format was altered to provide a greater range of pertinent information that is easier for the user to understand.
2. Logic representing the conflict between pedestrians and turning vehicles at signalized intersections was formulated.
3. Left-turn interference logic was further refined to make the model's representation of this behavior more realistic.
4. Logic pertaining to the right-turn-on-red maneuver was included.

These refinements were made operational in late 1966 in the third version of the simulation model, titled TRANS III.

Finally, in conjunction with this project, subroutines were added to TRANS III to enable simulation of the experimental traffic-responsive concept of control, the basic queue-control mode. In this connection, it was also necessary to make minor modifications in the input format and to add to the program output a statistical summary of simulated traffic-signal operation.

Model Validation for Existing Conditions

Data from the coordinated field studies of the Pico arterial system were used to investigate the ability of the TRANS III model to represent traffic operation in the system with a sufficient degree of realism for practical purposes. The repetition of data collection on five separate weekdays permitted estimation of the precision of the field data in the form of confidence interval estimates. Similarly, five 1-hr replications of the computer simulation were performed, with input data representing existing field conditions, to yield estimates of the operating characteristics with known statistical precision. A comparative summary of the model validation results is given in Table 8. The results incorporate both the Pico Boulevard links and the signalized cross-street links.

In correspondence with the research agency's previous experience in testing TRANS III, the model very accurately reproduced the magnitude of system utilization as measured by total vehicle-miles traveled in the system. Also, in line with past experience, the model tended to slightly underestimate the operational effectiveness measure —total travel time in the system—and to slightly over-

TABLE 8

SIMULATION MODEL VALIDATION COMPARISONS

ITEM	TOTAL TRAVEL TIME (VEH-SEC) FIELD MEASUREMENT		TOTAL TRAVEL TIME (VEH-SEC) SIMULATION	TOTAL VEH-MILES FIELD MEASUREMENT		TOTAL VEH-MILES SIMULATION	AVERAGE SPEED (MPH) FIELD MEASUREMENT		AVERAGE SPEED (MPH) SIMULATION
(a) 2:30-3:30 PM									
Average	381,500		336,720	2,120		2,101	20.00		22.06
% difference		−11.7			−0.9			+10.3	
95% confidence interval	371,750-391,250		321,940-351,500	2.036-2,204		2,030-2,172	19.91-20.09		21.98-22.14
(b) 4:30-5:30 PM									
Average	580,310		520,750	2,895		2,929	17.96		20.24
% difference		−10.3			+1.2			+12.7	
95% confidence interval	528,510-632,110		513,920-527,580	2,772-3,018		2,901-2,957	17.13-18.79		20.08-20.40

estimate average speed attained. In other words, traffic is processed in the simulation model with somewhat less interference than is encountered under actual operating conditions. The differences, although statistically significant, are relatively small—on the order of 11 percent—and, more important, the differences are consistent rather than erratic.

Close inspection of the field-study results indicates that operational effectiveness characteristics fluctuate widely from day to day, particularly during peak periods. This factor, illustrated in Table 8 by the rather wide confidence-interval estimates for peak-period field measurements, reinforces one of the key arguments for the use of simulation for comparing system alternatives. It is almost hopeless to attempt to closely reproduce traffic demands in field studies of many system improvement alternatives. Simulation models do this, however, as a matter of course.

It is also worth noting that it is possible to force the simulation model to more closely fit empirically derived characteristics by artificially adjusting certain important parameters in the model. The research agency believes that such calibration adjustments are not necessarily useful, and did not pursue them in this project. What is more important is for the model to produce reliable relative values of operational effectiveness under different system conditions, such as modified concepts of traffic-signal operation. The research agency believes that at the time of the study the TRANS III simulation model was the best tool available for the pursuit of the project objectives.

RESULTS OF EFFECTIVENESS TESTS

Simulation Exercises

Using TRANS III, traffic operations in the Pico arterial system were simulated under each of the traffic-signal-control alternatives. Traffic characteristics input variables were identical for all control concept tests and represented the average characteristics measured on five separate days in the field. Simulations of peak-period and offpeak-period conditions were performed separately. In simulating either peak hour or offpeak hour, the traffic volumes and turning movement probabilities were modified every 15 min to correspond to field data.

For each hour, under each test condition, the simulation exercise was repeated five times, with each run utilizing a different starting random number. Replications were executed because the simulation model is a stochastic process, and measurements obtained from the simulation output have statistical variability. To interpret the results of simulation experiments, just as in the case of physical experimentation, it is necessary to estimate the precision associated with the measurements and the corresponding levels of confidence of the estimates.

Overall, a total of 100 real hours of traffic operation were simulated during the course of the experiment. The analysis of simulation results incorporates the operations characteristics on both Pico Boulevard and the signalized cross-street approaches.

Statistical Analysis of Results

Appendix C contains detailed results of the simulation tests. Tabulated are total vehicle-miles of travel in the arterial system, which was used as the principal measure of magnitude of system utilization, and total travel time and average speed in the system, which were used as measures of operational effectiveness.

Tables 9 and 10 summarize the 95 percent confidence-interval estimates of vehicle miles, total travel time, and average speed for all test conditions for the offpeak period and peak period, respectively. A convenient method of testing the significant difference between two sample estimates is to observe whether or not the confidence intervals overlap. Applying this technique, each of the test conditions was compared with the existing signal-timing plan to determine when the sample estimates of both total travel time and average speed differ significantly from the

TABLE 9

SUMMARY OF OFFPEAK-PERIOD SIMULATION RESULTS, CONFIDENCE INTERVALS

TEST CON-DITION	SPLITS	OFFSETS	95% CONFIDENCE INTERVAL ESTIMATES: VEH-MILES	TOTAL TRAVEL TIME (VEH-SEC)	AVERAGE SPEED (MPH)
		(a) STRATEGIC CONCEPTS			
1	Existing	Existing	2,035-2,175	331,150-355,850	21.98-22.14
2a	Yardeni	Yardeni	2,030-2,172	321,940-351,500	22.29-22.63
*2b	Existing	Yardeni	2,045-2,137	335,985-348,915	21.79-22.17
2c	Webster	Yardeni	2,026-2,122	303,785-319,445	23.88-24.04
3a	Existing	Little	1,979-2,165	308,100-337,800	23.03-23.17
*3b	Webster	Little	2,048-2,148	302,010-318,170	24.24-24.48
4a	Existing	Delay/ difference	1,988-2,130	314,410-335,590	22.63-22.97
*4b	Webster	Delay/ difference	2,058-2,134	297,865-308,955	24.76-24.98
		(b) OTHER CONCEPTS			
*5	Cycle and offset selection mode		Performance equals best strategic plan		
6	Experimental traffic-responsive concept—basic queue-control mode		2,052-2,150	314,672-331,662	23.24-23.56
7	Special strategic concept—mixed-cycle mode		Not applicable to offpeak conditions		

* Comparing modified and signal timing conditions with existing timing, sample estimates of both total travel time and average speed are significantly different at the 0.05 level.

TABLE 10

SUMMARY OF PEAK-PERIOD SIMULATION RESULTS, CONFIDENCE INTERVALS

TEST CON-DITION	SPLITS	OFFSETS	95% CONFIDENCE INTERVAL ESTIMATES: VEH-MILES	TOTAL TRAVEL TIME (VEH-SEC)	AVERAGE SPEED (MPH)
		(a) STRATEGIC CONCEPTS			
1	Existing	Existing	2,901-2,957	513,920-527,580	20.08-20.40
2a	Yardeni	Yardeni	2,812-3,048	478,720-519,600	20.44-21.82
*2b	Existing	Yardeni	2,839-3,013	487,590-509,480	20.88-21.36
*2c	Webster	Yardeni	2,854-2,970	481,930-504,930	21.10-21.34
3a	Existing	Little	2,946-3,098	503,060-528,540	20.88-21.30
*3b	Webster	Little	2,859-3,043	479,320-511,360	21.27-21.61
4a	Existing	Delay/ difference	2,942-3,036	500,620-519,500	20.96-21.22
*4b	Webster	Delay/ difference	2,851-3,015	471,360-508,140	21.44-21.66
		(b) OTHER CONCEPTS			
5	Traffic-adjusted concept—cycle and offset selection mode		2,911-3,019	488,530-516,650	20.93-21.57
*6	Experimental traffic-responsive concept—basic queue-control mode		2,869-2,969	470,984-486,644	21.71-22.19
*7	Special strategic concept—mixed-cycle mode		2,894-3,004	465,048-494,640	21.82-22.44

* Comparing modified signal timing conditions with existing timing, sample estimates of both total travel time and average speed are significantly different at the 0.05 level.

base condition. Results of the significance tests are indicated by asterisks in Tables 9 and 10. Level of significance, $\alpha = 0.05$, applies in all cases.

For the offpeak-period results (Table 9), it is interesting to note that when any of the offset determination concepts (Yardeni, Little, or delay/difference-of-offset) is used in combination with modified splits, as derived by the Webster method, the resulting traffic operation is significantly better than operation under the existing signal-timing plan. However, in no case are the differences in total travel time statistically significant when the offset improvement concepts are applied without improvement of the splits. With one exception, the peak-period results regarding the strategic concepts (Table 10) are similar to the offpeak results. Again, when offset improvement concepts are coupled with Webster splits, the resulting operational effectiveness is significantly better than under existing signal timing. Conversely, with the exception of the Yardeni concept, application of the various offset improvement concepts without changing the signal splits did not significantly improve total travel time.

With regard to the traffic-adjusted concept, cycle and offset selection mode, the offpeak-period results (Table 9) at least equal the best strategic plan and therefore represent significant improvements over existing signal operation. For the peak period (Table 10), total travel time in the system under the cycle and offset selection mode did not differ significantly from existing operation.

Comparing existing timing with the experimental traffic responsive concept, basic queue control, average speeds were significantly improved during both offpeak and peak periods. Using basic queue control, total travel time in the system was significantly reduced during the peak period, but during the offpeak period the improvement was not quite statistically significant.

Finally, application of the special strategic concept, mixed-cycle mode, which was tested for peak-period conditions only, produced clearly significant improvements in both total travel time and average speed.

It is important to note that, whether during the offpeak-hour or the peak-hour tests, the confidence-interval estimates for total vehicle-miles traveled in the system always overlapped. This means that, as intended, all the test conditions were subjected to magnitudes of traffic system utilization that did not differ significantly.

Figure 5 shows graphically the mean values of total travel time and average speed for all test conditions.

Ranking of Alternative Control Concepts

Based on the mean values of the traffic characteristics obtained by simulation, the alternative control concepts were ranked according to average speed and total travel time.

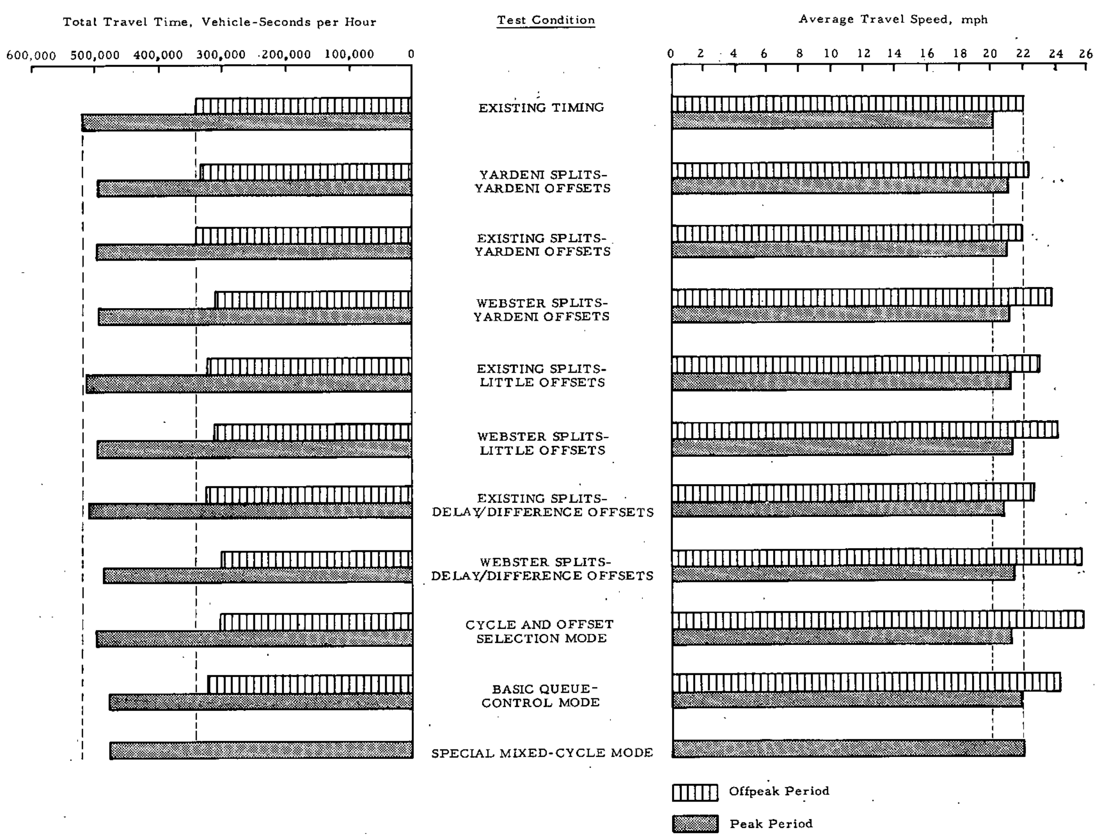

Figure 5. Comparison of operational effectiveness of all traffic-signal-control alternatives.

Results of these rankings are given in Tables 11 and 12 for offpeak and peak periods, respectively, along with indicated percentage improvements of the modified plans as compared with the existing timing plan.

Interrelationships of Vehicle-Miles, Total Travel Time, and Average Speed

A commonly used method of graphically depicting performance of a traffic system is to plot operational effectiveness as a function of system demand or utilization. This was done to compare operation under existing signal timing with operation under each of the alternative control concepts tested. Figures 6 through 9 show graphs of total travel time as a function of total vehicle-miles traveled. The sloped reference lines superimposed on the graphs represent average speed in the system. Each plotted point represents values of the variables obtained from 1 hr of traffic simulation. It is important to note again, on these graphs, that the offset determination concepts are more effective when used in conjunction with Webster splits than when applied independently.

Uniformity of Speeds on Individual Links

More detailed inspection of the experimental results uncovered interesting differences in traffic operation with respect to the uniformity of average speeds on individual links in the network. Figure 10 shows frequency distributions of average speed for individual links for three of the signal operation test conditions: (1) the existing timing plan, (2) the best of the strategic alternatives, and (3) the experimental basic queue-control mode. It can be seen that the existing timing plan produces highly variable average speeds, link by link, where in general the Pico Boulevard links are characterized by high average speeds, and lower speeds are found on the street crossing the arterial. Applying the best strategic plan, employing Webster splits and delay/difference offsets, appears to slightly reduce the range of average speeds, but accentuates the differential between performance on the arterial and the cross streets. Operating the signal system by the basic queue-control concept, however, brings the system performance into much better balance, with a striking reduction of the variability of average speeds on individual links.

Total Delay in the System

Using total travel time or average speed in the network can be somewhat misleading; this is because these characteristics take into account the time spent in the immediate vicinity of the signalized intersection as well as all time spent traveling between signalized points. Delays at the signalized intersections are often more representative measures of effectiveness of alternative signal control concepts. Total delay in the system can be considered the marginal portion of total travel time; i.e., that portion of total travel time in excess of total undelayed travel time.

By definition,

$$DT = TT - UTT \tag{6}$$

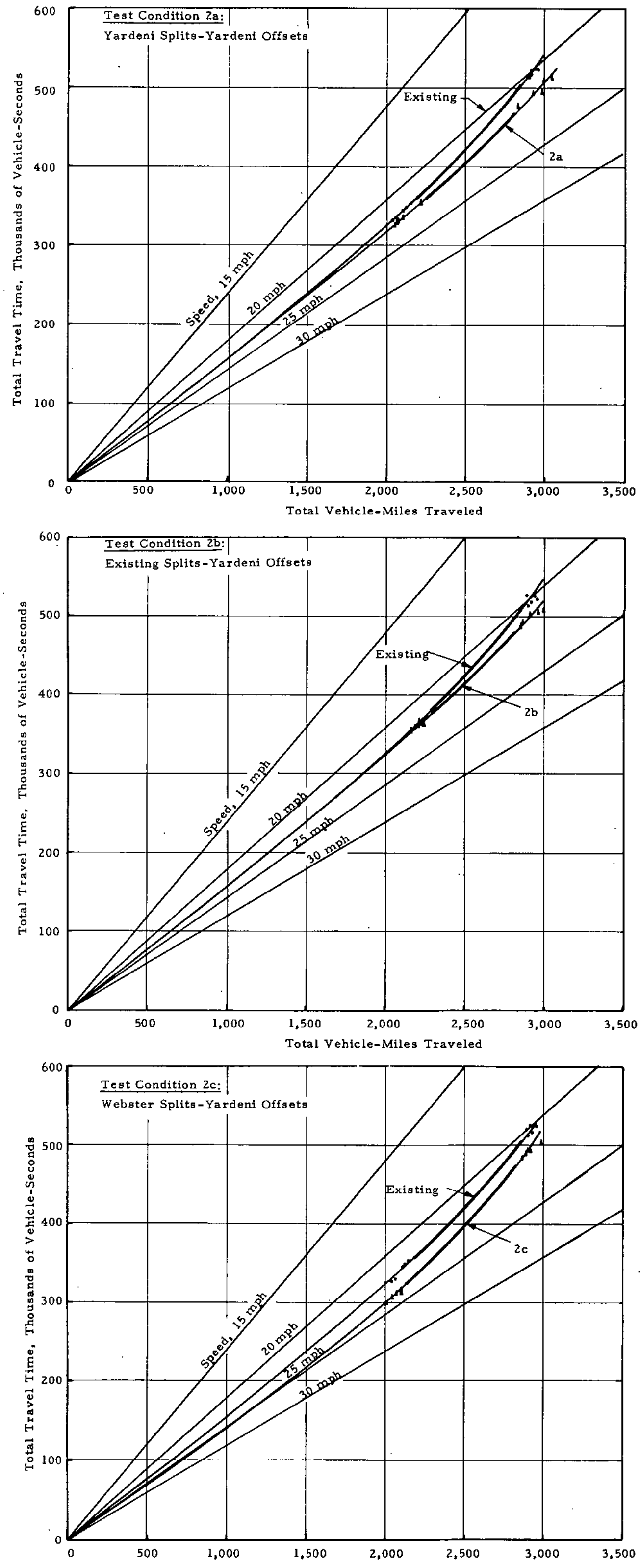

Figure 6. Total travel time as a function of total vehicle-miles: existing timing compared with alternatives employing Yardeni offsets.

TABLE 11

RANKING OF ALTERNATIVE CONTROL CONCEPTS FOR OFFPEAK PERIOD

RANK			TEST CONDITION	IMPROVEMENT OVER EXISTING PLAN (%)
			(a) RANKING ACCORDING TO AVERAGE SPEED	
Best	tie	1	Cycle and offset selection mode	12.7
		1	Webster splits—delay/difference offsets	12.7
		3	Webster splits—Little offsets	10.4
		4	Webster splits—Yardeni offsets	8.6
		5	Basic queue-control mode	6.1
		6	Existing splits—Little offsets	4.7
		7	Existing splits—delay/difference offsets	3.4
		8	Yardeni splits—Yardeni offsets	1.8
		9	Existing signal timing plan	—
Worst		10	Existing splits—Yardeni offsets	−0.4
			(b) RANKING ACCORDING TO TOTAL TRAVEL TIME	
Best	tie	1	Cycle and offset selection mode	11.7
		1	Webster splits—delay/difference offsets	11.7
		3	Webster splits—Little offsets	9.7
		4	Webster splits—Yardeni offsets	9.3
		5	Existing splits—Little offsets	6.0
		6	Basic queue-control mode	5.9
		7	Existing splits—delay/difference offsets	5.4
		8	Yardeni splits—Yardeni offsets	2.0
		9	Existing splits—Yardeni offsets	0.3
Worst		10	Existing signal timing plan	—

TABLE 12

RANKING OF ALTERNATIVE CONTROL CONCEPTS FOR PEAK PERIOD

RANK			TEST CONDITION	IMPROVEMENT OVER EXISTING PLAN (%)
			(a) RANKING ACCORDING TO AVERAGE SPEED	
Best		1	Mixed cycle mode	9.3
		2	Basic queue-control mode	8.4
		3	Webster splits—delay/difference offsets	6.5
		4	Webster splits—Little offsets	5.9
		5	Cycle and offset selection mode	5.0
		6	Webster splits—Yardeni offsets	4.8
		7	Yardeni splits—Yardeni offsets	4.4
		8	Existing splits—Yardeni offsets	4.3
	tie	9	Existing splits—delay/difference offsets	4.2
		9	Existing splits—Little offsets	4.2
Worst		10	Existing signal timing plan	—
			(b) RANKING ACCORDING TO TOTAL TRAVEL TIME	
Best		1	Basic queue-control mode	8.1
		2	Mixed-cycle mode	7.9
		3	Webster splits—delay/difference offsets	6.0
		4	Webster splits—Yardeni offsets	5.2
		5	Webster splits—Little offsets	4.9
		6	Existing splits—Yardeni offsets	4.3
		7	Yardeni splits—Yardeni offsets	4.1
		8	Cycle and offset selection mode	3.5
		9	Existing splits—delay/difference offsets	2.1
		10	Existing splits—Little offsets	1.0
Worst		11	Existing signal timing plan	—

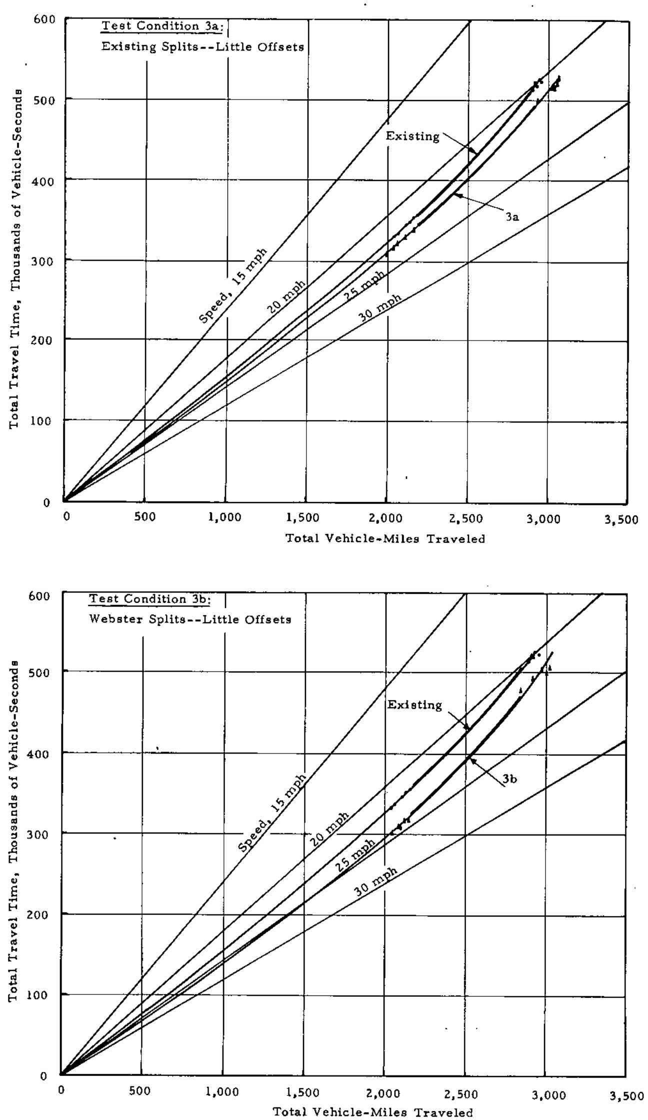

Figure 7. Total travel time as a function of total vehicle-miles: existing timing compared with alternatives employing Little offsets.

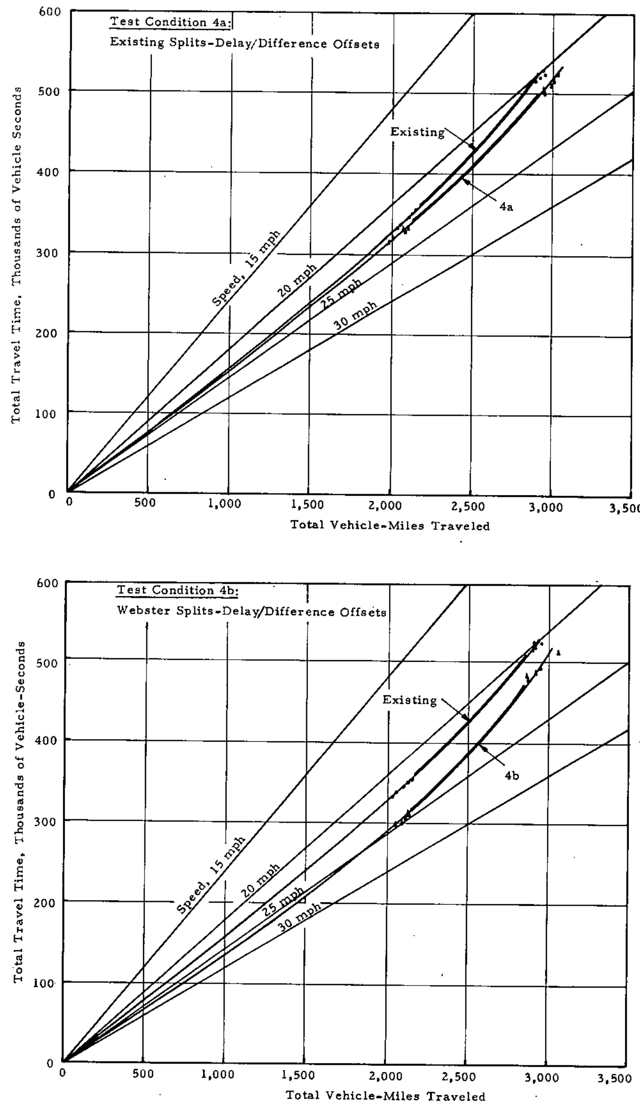

Figure 8. Total travel time as a function of total vehicle-miles: existing timing compared with alternatives employing delay/difference offsets.

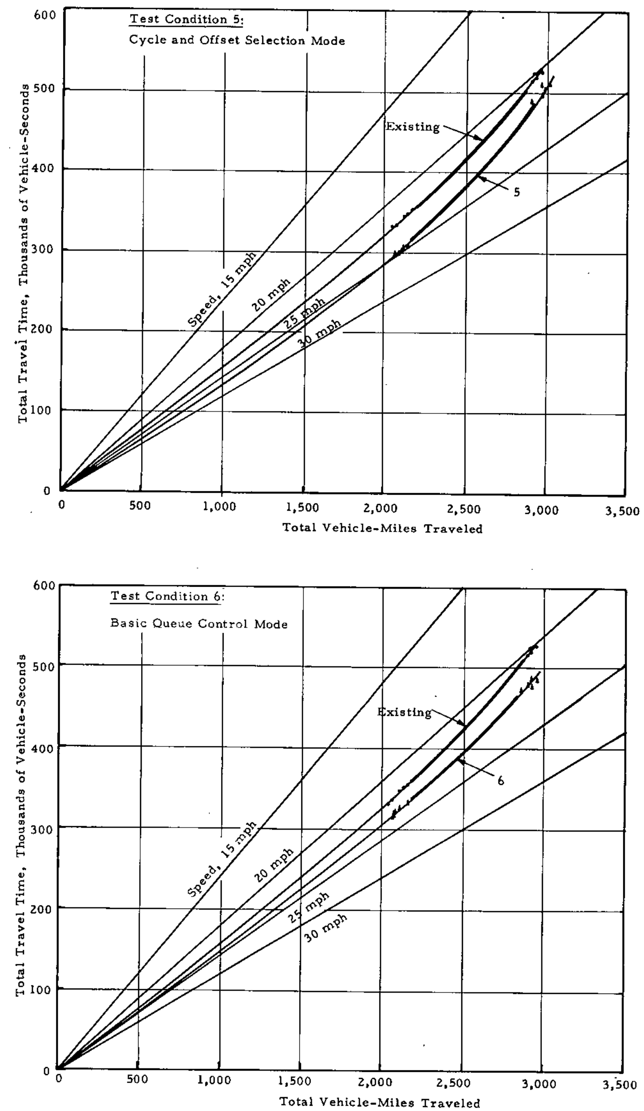

Figure 9. Total travel time as a function of total vehicle-miles: existing timing compared with traffic-adjusted and traffic-responsive concepts.

in which

$DT =$ total delay in system, veh-sec;
$TT =$ total travel in system, veh-sec;
$UTT =$ total undelayed travel time in system, veh-sec;

and

$$UTT = \frac{3{,}600\ VM}{S_{FF}} \tag{7}$$

in which

$VM =$ total vehicle-miles traveled;
$S_{FF} =$ average running speed, mph; and
$3{,}600 =$ sec per hr.

Using the 31-mph average running speed determined from empirical data, total delays were computed for each of the test conditions from the simulation results. Table 13 gives the results of these computations, with the alternative control concepts ranked from best to worst. Percentage improvements with respect to existing conditions are also indicated. It is evident that the use of total delay as a measure of effectiveness accentuates the relative degree of improvement produced by alternative traffic-signal-control methods.

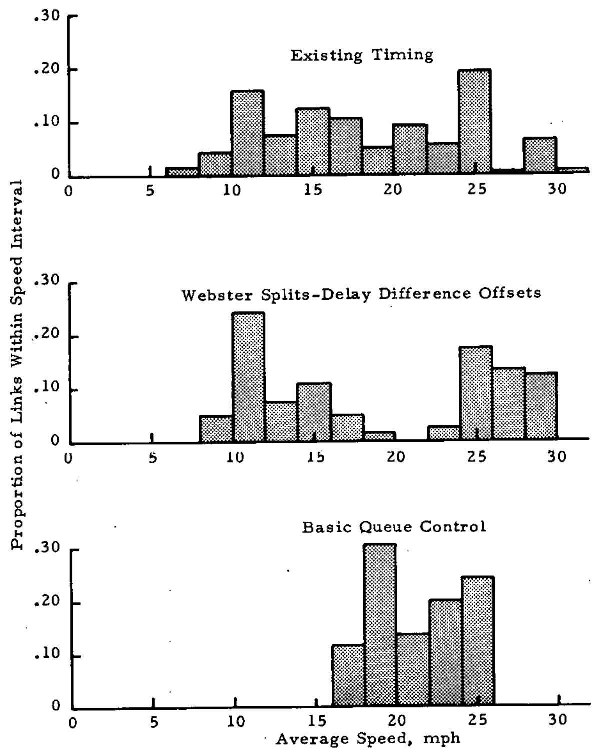

Figure 10. Frequency distribution of average speeds on individual network links, 4:30 to 5:30 PM.

TABLE 13

RANKING OF ALTERNATIVE CONTROL CONCEPTS IN ACCORDANCE WITH TOTAL DELAY IN THE SYSTEM

RANK		TEST CONDITION	TOTAL DELAY (VEH-SEC)	IMPROVEMENT OVER EXISTING PLAN (%)
		(*a*) OFFPEAK PERIOD		
Best tie	1	Cycle and offset selection mode	60,002	39.4
Best tie	1	Webster splits—delay/difference offsets	60,002	39.4
	3	Webster splits—Little offsets	66,449	33.0
	4	Webster splits—Yardeni offsets	70,761	28.5
	5	Basic queue-control mode	79,178	20.1
	6	Existing splits—Little offsets	82,329	16.9
	7	Existing splits—delay/difference offsets	85,888	13.3
	8	Yardeni splits—Yardeni offsets	92,731	6.4
	9	Existing signal timing plan	99,046	—
Worst	10	Yardeni splits—Yardeni offsets	99,622	—0.6
		(*b*) PEAK PERIOD		
Best	1	Mixed-cycle mode	137,377	23.8
	2	Basic queue-control mode	139,831	22.5
	3	Webster splits—delay/difference offsets	149,141	17.3
	4	Webster splits—Little offsets	152,640	15.4
	5	Webster splits—Yardeni offsets	155,759	13.7
	6	Cycle and offset selection mode	158,265	12.3
	7	Existing splits—Yardeni offsets	158,739	12.1
	8	Yardeni splits—Yardeni offsets	158,901	12.0
	9	Existing splits—delay/difference offsets	162,947	9.7
	10	Existing splits—Little offsets	164,855	8.6
Worst	11	Existing signal timing plan	180,605	—

CHAPTER THREE

INTERPRETATION

The results of this research indicate that significant improvements in traffic operation in urban signalized arterial systems can be obtained by applying either strategic or tactical control concepts. In some cases the relative improvements in operational effectiveness were small, and in numerous instances were not statistically significant. It was most encouraging, however, that practically all the improvement methodologies tested by simulation produced numerical improvements when compared with the existing signal-operation plan. There was only one exception, during the peak period, when a small, nonsignificant reduction in average speed occurred as a result of a modification in signal timing. Thus, it appears that whatever efforts traffic engineers can devote to pursuing signal operation improvements by methods such as those tested should result in measurable benefits.

The relative degree of improvement of traffic operation in the pilot arterial system was especially impressive when total delay was used as the measure of effectiveness. Assessed in this way, reductions in total delay produced by the best alternative control concepts tested were 39.4 percent and 23.8 percent for the offpeak hour and peak hour, respectively. In terms of total travel time or average speed (in which case all the time vehicles spend in the system is accounted for rather than just the delays in the immediate vicinity of signalized intersections) the best control alternatives yielded improvements on the order of 12 percent for the offpeak and 9 percent for the peak period. This illustrates that "percentage improvement" statements should be made with care, making sure to define precisely the basis of comparison.

Regarding the comprehensive tests of alternative strategic concepts, the Webster optimization of cycle and splits appeared to be the most effective course of action. Regardless of which concept of offset improvement was combined with the Webster method, increased operational effectiveness resulted as compared with test conditions utilizing existing cycle and splits. During the offpeak period, this is undoubtedly a result of the reduction in cycle length given by the Webster concept. However, even during the peak period, when a 60-sec system cycle length was used for all standard strategic tests, incorporating Webster splits with the various offset plans yielded the best results.

Comparing the strategic concepts used to derive alternative offset plans, the delay/difference-of-offset method consistently achieved the greatest increase in operational effectiveness when coupled with Webster splits. The next most effective offset improvement concept was Little's Maximal Bandwidth Model. The Yardeni Time-Space Design Model produced offset plans that effected the smallest improvements in operation of the three offset methods tested in combination with Webster splits.

It is believed that the delay/difference-of-offset concept yields better results because, unlike the other two offset determination schemes, it takes into account turning movement patterns at the link tail and gains or losses of traffic at intervening points between the signalized intersections. The Little and Yardeni methods, on the other hand, are derived by manipulation of the through bands. Furthermore, the delay/difference-of-offset technique can be applied to any network configuration, whereas the other offset concepts are limited to arterial systems.

The results of the strategic control concept experiments indicate that, when devising improved signal timing, attention should be devoted to all three variables—cycles, splits, and offsets—rather than concentrating on one of the timing variables independently. This requires that the alternative concepts investigated in this project must be applied in combinations to formulate effective signal plans. The one concept tested here that can be applied to determine a fully integrated plan with cycle, splits, and offset modifications—the Yardeni method—was not particularly effective when used by itself. Other strategic control concepts that are designed to produce complete plans are in various stages of development and refinement elsewhere, and it is anticipated that application of these further advanced methods could prove to be very effective.

One of the most interesting findings of the experimental work was the impressive showing of the special strategic concept—the mixed-cycle mode—tested for peak-period conditions. The tests illustrated that the use of a common signal cycle to maintain fixed relationships between all the signals along an arterial is not necessarily the best policy. Although this conclusion perhaps runs contrary to standard traffic engineering practice, it is believed that a large number of situations exist in arterial systems in which operations could be improved significantly through the use of mixed-signal cycles.

The comprehensive simulation tests of traffic-adjusted and traffic-responsive concepts of control also yielded encouraging results. The cycle and offset mode of control was shown to be equally as effective as the best strategic plan conceived for offpeak-period operation. Although it did not yield the best results during the peak period, the cycle and offset selection mode nevertheless provided significant improvements over the existing signal operation plan. It should be noted at this point that whereas strategic concepts are only as good as the traffic flow data on which they are based, the traffic-adjusted concept has the ability to respond in a limited way to major changes in traffic volumes on the arterial street. Thus, one can expect the relative merit of the traffic-adjusted concept to improve when tested over longer periods of time. Conversely, strategic plans can deteriorate over time, unless comprehensive programs of periodic traffic data collection are instituted that detect major alterations of traffic volume patterns.

Tests of the experimental traffic-responsive concept, the basic queue-control mode, also indicated significant upgrading of operational effectiveness. During the peak period, in particular, basic queue control ranked second among the alternatives. The results were especially interesting, inasmuch as this mode of control operated each signalized intersection independently without regard to maintenance of offsets between signals. Even though the signals were rather closely spaced (ranging from 1,000 to 1,500 ft apart), a high level of operational effectiveness was achieved. The peak-period results were not so good, but they were still significantly better than existing control in the arterial system. The implication of these results takes on greater significance when it is considered that traffic-responsive control concepts take full account of changes in traffic patterns and magnitudes in the system that occur over time and respond appropriately. Therefore, as compared with strategic concepts, traffic-responsive control concepts have an even greater potential to improve over time than does the traffic-adjusted control concept.

It is anticipated that, given the indicated degree of improvement possible through an independently operating traffic-responsive control concept, future systems that integrate full traffic responsiveness with the positive features of effective strategic plans should achieve even greater benefits.

CHAPTER FOUR

APPLICATIONS

The following describes some important, immediate practical applications of the research findings.

First, the research suggests that traffic engineers should initiate systematic programs to apply the recently developed methods for determining improved strategic (fixed-time) control plans. Such programs could proceed without major capital expenditures for new control hardware and would result in significant improvements in operational effectiveness on the signalized arterials in the city. The results indicate that all three offset determination concepts, when used in conjunction with the Webster method for optimizing cycles and splits, produce measurable improvements in operation. The Webster method requires only simple hand calculations, given the appropriate traffic data. The three offset concepts investigated require the use of special computer programs, for which documentation is readily obtainable from agencies that prepared the programs.

The theory and derivation of the Webster method for determining optimum cycles and offsets are thoroughly documented in the literature (*28, 79*) and are discussed in Chapter Two. To illustrate the relative simplicity of the calculations, a sample worksheet for a single intersection is shown in Figure 11. Table 14 is an example summary of the Webster method results for the six-intersection Pico arterial system studied in this project.

As reviewed in this report, the input data requirements for the three offset computation techniques are varied, and in one instance (the Yardeni method) quite complicated. The general character of the input data requirements of each of the three computer programs is summarized in Table 15.

The Little Maximal Bandwidth Model was the second most effective offset concept tested and appeared to be the simplest to use. Documentation of the original program is available through the Civil Engineering Systems Laboratory of the Massachusetts Institute of Technology (*77*). Potential users of this program should obtain the documentation from MIT in order to study the operation of the program and the input data requirements in greater detail than can be appropriately given in this report.

The Yardeni Time-Space Design Model is slightly more difficult to apply because of the rather complicated input data requirements. In addition to the data summarized in Table 14, there are a number of control parameters and subroutine options that require considerable judgment and experience with the program. The computer program is available to potential users through the SHARE General Program Library; inquiries regarding its use should be directed to International Business Machines Corporation. The available documentation is comprehensive (*78*).

The delay/difference-of-offset concept appeared to be the most effective of the three tested. It is somewhat more difficult to utilize in its present stage of development because some hand processing of the computer program results is still required to finalize an offset plan. Inquiries concerning the delay/difference-of-offset computer program should be directed to Planning Research Corporation. Detailed documentation for this program appears in Appendix A of this report. Figure 12 shows a full set of coded input data for the six-intersection Pico arterial system.

In applying strategic concepts for the improvement of operation on urban arterials, the traffic engineer should not overlook potential application of the special mixed-cycle mode. Positive results can frequently be obtained by oper-

Intersection Pico Boulevard and La Brea Avenue
Period of Day 4:30 PM to 5:30 PM Completed by Barnes Date 1 March 67

PHASE A

Direction	west bound	east bound
Max. Lane Probability	.41	.54
Saturation Flow Rate	.469 vps	.469 vps
Lost Time	3.15 sec	3.15 sec
Approach Volume	954 vph	1032 vph
Approach Volume	.2650 vps	.2867 vps
Ratio $\frac{\text{Max. Lane Vol.}}{\text{Saturation Flow}}$	$\frac{.1087}{.469} = .232$	$\frac{.1548}{.469} = .330$

$Lost_A$ = 3.15 sec Y_A = .330

PHASE B

Direction	north bound	south bound
Max. Lane Probability	.47	.41
Saturation Flow Rate	.558 vps	.558 vps
Lost Time	3.55 sec	3.55 sec
Approach Volume	1677 vph	1780 vph
Approach Volume	.4658 vps	.4944 vps
Ratio $\frac{\text{Max. Lane Vol.}}{\text{Saturation Flow}}$	$\frac{.2189}{.558} = .392$	$\frac{.2027}{.558} = .363$

$Lost_B$ = 3.55 sec Y_B = .392

$$\text{Optimum Cycle, } C_o = \frac{1.5\,(Lost_A + Lost_B) + 5}{1 - Y_A - Y_B} = \frac{1.5(6.70) + 5}{1 - .330 - .392}$$

$$= \frac{15.05}{.278} = 54.1 \text{ sec}$$

CYCLE = 60 sec

Effective Green, GE = CYCLE - $Lost_A$ - $Lost_B$ = 53.30 sec

Phase A Effective Green, $GE_A = \frac{Y_A\,(GE)}{Y_A + Y_B} = \frac{.330(53.30)}{.722} =$ 24.40 sec

Phase A Green and Yellow, $(G + Y)_A = GE_A + Lost_A =$ 27.55 sec

Phase B Effective Green, $GE_B = GE - GE_A =$ 28.90 sec

Phase B Green + Yellow, $(G + Y)_A = GE_B + Lost_B =$ 32.45 sec

Figure 11. Sample worksheet, Webster method.

ating a series of minor intersections along an arterial with a shorter cycle-length than nearby critical intersections. This technique requires use of the Webster computations for optimum cycles and splits and, additionally, the application of one of the offset determination concepts for any series of intersections having a common signal cycle. What is being suggested here, in effect, is that fixed-time plans should be more uniquely tailored to traffic demands at individual intersections, rather than burdening the entire system with the cycle-length requirements of critical intersections.

Another important application of the research findings pertains to the planning of major traffic-signal system improvements. Result of the comprehensive tests of alternative control concepts have immediate practical value to cities and states undertaking major traffic control improvement projects. The comparative data on the degree of increase in operational effectiveness attributed to alternative control concepts serve as fundamental data for comprehensive cost-benefit analyses of these large-scale undertakings. In the final analysis, this will lead to the selection and implementation of more efficient systems for the resources available.

TABLE 14

SAMPLE RESULTS, WEBSTER METHOD, PICO ARTERIAL SYSTEM, 4:30 TO 5:30 PM

INTERSECTION NUMBER	PHASE A (PICO BOULEVARD) LOST TIME (SEC)	SATURATION FLOW RATE (VPS)	MAX. LANE VOL. (VPH)	Y_A	PHASE B (CROSS STREETS) LOST TIME (SEC)	SATURATION FLOW RATE (VPS)	MAX. LANE VOL. (VPH)	Y_B	CYCLE (SEC) OPTIMUM	SELECTED	EFFECTIVE GREEN (SEC) TOTAL	PHASE A	GREEN PLUS YELLOW (SEC) PHASE A	PHASE B
1	3.15	0.469	558	0.330	3.55	0.558	788	0.392	54.1	60	53.30	24.4	28	32
2	3.15	0.469	471	0.279	2.73	0.503	356	0.196	26.3	60	54.12	31.8	35	25
3	3.15	0.469	459	0.271	2.73	0.503	234	0.129	23.0	60	54.12	36.7	40	20
4	3.15	0.469	442	0.262	2.73	0.503	331	0.183	24.9	60	54.12	31.9	35	25
5	3.15	0.469	453	0.263	2.73	0.503	209	0.116	22.3	60	54.12	37.6	41	19
6	3.15	0.469	436	0.258	2.73	0.503	153	0.0845	21.0	60	54.12	40.8	44	16

TABLE 15

COMPARATIVE INPUT DATA REQUIREMENTS FOR TRAFFIC-SIGNAL TIMING PROGRAMS

DATA	LITTLE MAXIMAL BANDWIDTH	DELAY/ DIFFERENCE-OF-OFFSET	YARDENI TIME-SPACE DESIGN
Block lengths	X	X	X
Number of lanes (each block)	—	X	X
Directional traffic counts (average)	X	—	—
Directional traffic counts (each main street approach)	—	—	X
Turning traffic counts (each intersection)	—	X	—
Lane traffic counts (each cross street approach)	—	—	X
Saturation flow and lost time (each main approach)	—	X	—
Headways (average)	X	—	—
Running speeds (each block)	X	X	—
Maximum speed difference (inbound vs outbound)	—	—	X
Speed-volume relationship (average)	—	—	X
Signal cycle length (system)	X	X	—
Range of cycle lengths (system)	—	—	X
Minimum main street red interval (each intersection)	X	—	X
Main street green and yellow intervals (each intersection)	—	X	—

Run Number (link number)	Signal Cycle Length (sec)	Green Interval, Intersection 1 (sec)	Green Interval, Intersection 2 (sec)	Amber Interval (sec)	Single-Lane Saturation Flow (vps)	Lost Time Per Green Interval (sec)	Distance Between Signals (feet)	Number of Through Lanes, Intersection 2	Straight-Through Traffic Arriving at Tail (vph)	Left Turns Arriving at Tail (vph)	Right Turns Arriving at Tail (vph)	Total Volume at Head (vph)	Average Free-Flow Velocity (ft/sec)
1 2 3 (4)	5 6	7 8	9 10	11	12 13 14 15	16	17 18 19 20	21 22	23 24 25 26 27	28 29 30 31 32	33 34 35 36 37	38 39 40 41 42	43 44
P15	60	31	27	3	.469	3	1208	2.	832.	9.	59.	855.	45
P16	60	34	31	3	.469	3	988	2.	774.	66.	65.	870.	45
P17	60	35	34	3	.469	3	1306	2.	843.	21.	60.	883.	45
P18	60	34	35	3	.469	3	1098	2.	805.	74.	72.	935.	45
P19	60	24	34	3	.469	3	1484	2.	838.	153.	70.	906.	45
P20	60	34	24	3	.469	3	1458	2.	724.	30.	108.	954.	45
P21	60	35	34	3	.469	3	1080	2.	761.	70.	42.	847.	45
P22	60	34	35	3	.469	3	1306	2.	699.	28.	78.	815.	45
P23	60	31	34	3	.469	3	1066	2.	780.	21.	30.	834.	45
P24	60	27	31	3	.469	3	1164	2.	765.	30.	28.	809.	45

Figure 12. Delay/difference-of-offset input data, Pico arterial system, 4:30 to 5:30 PM, existing splits.

CHAPTER FIVE

CONCLUSIONS

A literature search and review of related activities was conducted to acquire an understanding of the recent state-of-the-art of traffic-signal systems. The results of this effort were (1) a bibliography concentrating on the more recent research endeavors; (2) abstracts of selected papers; and (3) technical notes on current signal-control projects. All of these appear as appendixes in this report.

Strategic concepts of signal control were thoroughly reviewed, and four methods were applied in various combinations to formulate strategic timing plans for a pilot study arterial system. In addition, a traffic-adjusted control concept, an experimental traffic-responsive control, and a specialized adaptation of a strategic concept were developed for testing in the pilot system. In all, 11 test conditions (described in Chapter Two) were formulated for comprehensive testing and evaluation.

A section of a signalized arterial street in the city of Los Angeles was selected as a pilot system for testing the various alternative concepts of traffic-signal operation. Empirical data were collected in the pilot system for use in conjunction with the signal-timing computer programs and the traffic-simulation program. Most important was the coordinated 5-day effort involving simultaneous acquisition of aerial photographic data, instrumented floating-car recordings of speed and delay characteristics, and manual traffic counts.

The research agency's traffic arterial and network simulation model, TRANS III, was used for conducting comprehensive, controlled tests of the effectiveness of the traffic-signal-control alternatives. In this connection, new subroutines were incorporated in TRANS III to simulate the experimental traffic-responsive control concept. The experimental, basic queue-control mode of signal control was the same one developed and tested for individual intersections in an earlier phase of this project. The simulation results for all alternatives incorporated operating characteristics on the arterial street and the signalized cross-street approaches.

A total of 100 real-time hours of traffic operation were simulated to produce statistically reliable results in conjunction with the effectiveness tests of alternatives. The magnitudes of system utilization, as measured by total vehicle-miles of travel in the system, under which each of the alternatives was tested, were not significantly different. In other words, the alternatives were subjected to closely controlled traffic demands.

With only a single exception, the results indicated that all the signal-control alternatives produced numerical improvements in operating characteristics in comparison to the existing signal-operation plan. However, not all of these differences were statistically significant.

All three offset determination concepts—Yardeni, Little, and delay/difference-of-offsets—yielded statistically significant improvements in performance when integrated with the Webster technique for optimizing cycle and splits. However, with only one exception, the same three offset concepts failed to produce statistically significant improvements when applied independently, i.e., with the existing cycle and splits.

The traffic-adjusted concept of control at least equaled the improvement level achieved with the best of the strategic plans during the offpeak period. However, during the peak period, the numerical improvements resulting from the application of the traffic-adjusted concept were not statistically significant.

The experimental traffic-responsive concept ranked second of all alternatives tested under peak-period traffic conditions. Improvement in operational effectiveness produced by this method during the offpeak period was smaller, however, and was not statistically significantly different from operation under the existing signal-timing plan. An interesting characteristic of behavior under this concept of control was a striking reduction in the variability of average speeds for the individual links in the system.

The mixed-cycle mode of signal operation, wherein a 40-sec cycle length was utilized at five of the intersections and a 60-sec cycle length at the critical intersection, was tested during the peak period only. The resulting operation was the best of any alternative tested.

The best of the traffic-signal-control alternatives tested produced reductions in total delay in the system of 39 percent during the offpeak hour and 24 percent during the peak hour. Assessed in terms of total travel time or average speed in the system, the best control alternative resulted in improvements of approximately 12 percent during the offpeak period and 9 percent in the peak period.

It is believed that the results of this research can be immediately translated into practical traffic-engineering applications.

CHAPTER SIX

SUGGESTED RESEARCH

EXTENSION OF RESEARCH APPROACH

Several advanced concepts of traffic-signal control, both strategic and tactical, are currently in various stages of development and refinement. These developments had not proceeded to the point to warrant comprehensive testing in this project. In the near future, however, it would be appropriate to subject these additional advanced concepts to controlled experimentation by simulation to estimate the effectiveness of each under conditions representing an actual street system. It is also considered fruitful to extend the controlled testing process to more complex, closed network configurations.

CONCEPTUAL DEVELOPMENT OF NEW METHODS

Another important area in need of research is the conceptual development of the type of control methods that can be implemented by digital control computer systems. Here it is believed that special attention should be devoted to the formulation and testing of methods that combine the positive qualities of the strategic and tactical approaches.

FINAL DEVELOPMENT OF DELAY/DIFFERENCE-OF-OFFSET COMPUTER PROGRAM

This research indicated that the delay/difference-of-offset technique of formulating a strategic offset plan was consistently more effective than the other techniques tested. Furthermore, this technique is more flexible in that it can be applied to networks of any configuration. The research agency's current delay/difference-of-offset computer program requires certain off-line processing of program output to produce a final result. Although this can be accomplished easily for the simple arterial situations, the off-line postprocessing is challenging for larger, closed systems. This inadequacy is expected to seriously limit practical application of the current program. It is suggested, therefore, that additional development of the delay/difference-of-offset computer program be undertaken to make the process entirely automatic.

DEVELOPMENT OF COST ANALYSIS METHODS FOR TRAFFIC-CONTROL PROJECTS

A genuine need exists for the development of scientific management techniques to assist traffic administrators in their decisions concerning large capital expenditures for traffic-control improvement projects. The increasing size of investments in traffic-signal systems requires greater attention to determination of the cost and effectiveness parameters of such systems during the planning stage to achieve more efficient allocation of resources. Cost analysis studies, essential to all effective long-range planning, involve not only the costs of hardware and its installation and maintenance, but also a study of the allocation of all critical resources, especially of time and skilled manpower.

APPENDIX A

DERIVATION OF METHOD AND DESCRIPTION OF PROGRAM FOR COMPUTING DELAY/DIFFERENCE-OF-OFFSET RELATIONSHIPS

A computational method has been developed for theoretically estimating delay incurred by traffic because of queuing at the traffic signal on the downstream end of the link (link head); delay is estimated as a function difference-of-offset of the traffic signals at the link head and link tail, given certain unique geometric, signal timing, and traffic characteristics of the link. A computer program to implement this computation has been written, checked out, and used in this project.

DERIVATION

Given Characteristics

Consider a link, i.e., a section of street carrying traffic in one direction between two signalized intersections, as illustrated in Figure A-1.

Suppose the link has the following characteristics:

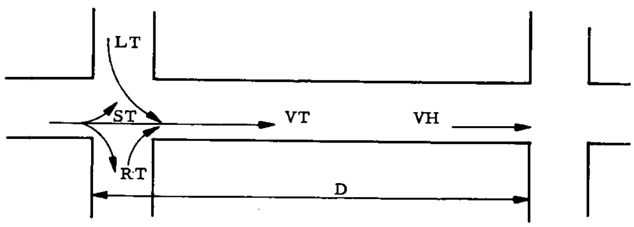

Figure A-1.

1. Geometrics:

D = distance between signalized intersections, stop-line to stop-line, ft (link length).
W = number of moving through-lanes at intersection 2 (link width).

2. Traffic Characteristics:

ST = straight-through movements arriving at the tail of the link, vph.
LT = left turns from the cross street arriving at the tail of the link, vph.
RT = right turns from the cross street arriving at the tail of the link, vph.
VT = total traffic volume arriving at the tail of the link, vph, = ST + LT + RT.
VH = total traffic volume arriving at the head of the link, vph. VH generally does not equal VT because vehicles may be gained or lost between the signalized intersections.
NC = intralink net change rate, vph, = VH − VT. The net gain or loss of traffic volume from the link tail to the link head.
V = average free-flow velocity of the traffic stream along the link, ft per sec.
SAT = saturation flow rate at link head, vehicles per lane per sec. The average rate at which a single-lane queue discharges through intersection 2 during the effective green portion of the signal cycle.

LOST = lost time per green interval at the link head, intersection 2, sec. Lost time results from starting delay at beginning of green and subsidence of traffic flow during the yellow interval.

3. Signal Timing:

C = common signal cycle length, sec.
G1 = green interval for the main street at intersection 1, sec.
A1 = yellow interval for the main street at intersection 1, sec.
R1 = red interval for the main street at intersection 1, sec.
G2 = green interval for the main street at intersection 2, sec.
A2 = yellow interval for the main street at intersection 2, sec.
R2 = red interval for the main street at intersection 2, sec.
GE = the effective green interval for the main street at intersection 2, sec, = G2 + A2 + LOST. During effective green, a lane of queued vehicles may be discharged through the intersection at saturation flow rate.
RE = the effective red interval for the main street at intersection 2, sec, = C − GE. During effective red, no vehicles may be discharged through the intersection.
BG1 = the signal offset for the main street green at intersection 1, sec. The offset is defined as the beginning of interval G1, measured in seconds from a master zero-reference point.
BG2 = the signal offset for the main street green at intersection 2, sec.
ψ = the difference-of-offsets for the link, sec, = [BG2 − BG1] Modulo C; i.e., = [BG2 − BG1] $\pm nC$,

in which n is an integer selected in such a way as to make $0 \leq \psi < C$.

Assumption and Definitions

The principal simplifying assumption for the initial analysis is that no significant dispersion of traffic platoons occurs along the link. This is not unreasonable for relatively short links. Under this assumption, two distinct bands of time contain identifiable components of traffic flowing down the link.

1. Straight-through traffic at the link tail, ST, is assumed to be uniformly distributed over time interval (G1 + A1) at the link tail, and maintains this distribution as it travels down the link, thereby arriving at the link head uniformly distributed over time interval T1. By definition,

$$T1 = G1 + A1 \tag{A-1}$$

2. Left-turn and right-turn traffic arriving at the link tail (LT + RT) is uniformly distributed over time interval R1 at the link tail, and maintains this distribution as it travels down the link, thereby arriving at the link head uniformly distributed over time interval T2. By definition,

$$T2 = R2 \tag{A-2}$$

Gains and losses of traffic along the link occur randomly with respect to time. Therefore, the intralink net change, NC, is uniformly distributed over time. For a given signal cycle, NC is uniformly distributed over C.

Given the preceding assumption, it follows that two rates of arrivals of traffic at the head of the link can be defined.

Q1 = Average arrival rate at the head of the link during time intervals T1

$$= \frac{ST}{\left(\frac{T1}{C}\right)\left(3{,}600\right)} + \frac{NC}{3{,}600}, \text{ vps.}$$

Q2 = Average arrival rate at the head of the link during time intervals T2

$$= \frac{LT + RT}{\left(\frac{T2}{C}\right)\left(3{,}600\right)} + \frac{NC}{3{,}600}, \text{ vps.}$$

It is assumed that the formation of queues and discharge of vehicles is uniformly distributed across the through lanes at intersection 2. Further, turning movements and their associated delay-producing effects at the link head (intersection 2) are ignored. Thus, for a W-lane intersection 2 approach, the rate of discharge of queued vehicles during effective green can be defined.

S = Average discharge rate of queued vehicles on W lanes during effective green intervals at intersection 2.

S = (SAT)(W), vps.

If the queue at intersection 2 is dissipated before the end of the effective green interval, then, for the remainder of the effective green interval, additional arriving vehicles may be discharged as rapidly as they arrive without forming queues.

It is assumed that subsaturation conditions exist. That is, $(Q1)(T1) + (Q2)(T2) < (S)(GE)$. Therefore, the queue is always dissipated at or before the end of the effective green interval.

Let

Q_t = The number of vehicles in queue at the link head at time t.

By definition,

$Q_t = 0$ for t = the end of time interval GE.

In the light of this definition, the end of time interval GE (which corresponds to the beginning of time interval RE) is useful as a time reference point. Therefore, the time differential, τ, is defined:

τ = The time differential between the beginning of time interval T1 (in more common terms the leading edge of the straight-through time band) and the succeeding end of time interval GE at intersection 2.

Figure A-2 shows pertinent time-distance relationships. It can be seen that:

$$\tau = (BG2 - BG1) - D/V - R2 \qquad \text{(A-3)}$$

but the difference of offsets is

$$\varphi = [BG2 - BG1] \text{ Modulo } C \qquad \text{(A-4)}$$

therefore,

$$\tau = [\varphi - D/V - R2] \text{ Modulo } C \qquad \text{(A-5)}$$

The domain of τ is $0 \leq \tau < C$.

Queue-Size Equation

Starting, by definition, with a zero queue at the end of time interval GE, the formation and dissipation of a queue during the subsequent signal cycle can be described. The queue change rate (QCR) is defined as:

QCR = The rate at which queues at the link head (intersection 2) either grow or dissipate with time, vps.

Only four average values of QCR are possible (five, if QCR = 0 is counted).

During time interval RE (i.e., when the signal is effectively red) the queue grows (QCR is positive). Only two values are possible.

1. During time interval T1, QCR = Q1, vps.
2. During time interval T2, QCR = Q2, vps.

During time interval GE (i.e., when the signal is effectively green) the queue dissipates (QCR is negative). Only two values are possible.

3. During time interval T1, QCR = Q1 − S, vps.
4. During time interval T2, QCR = Q2 − S, vps.

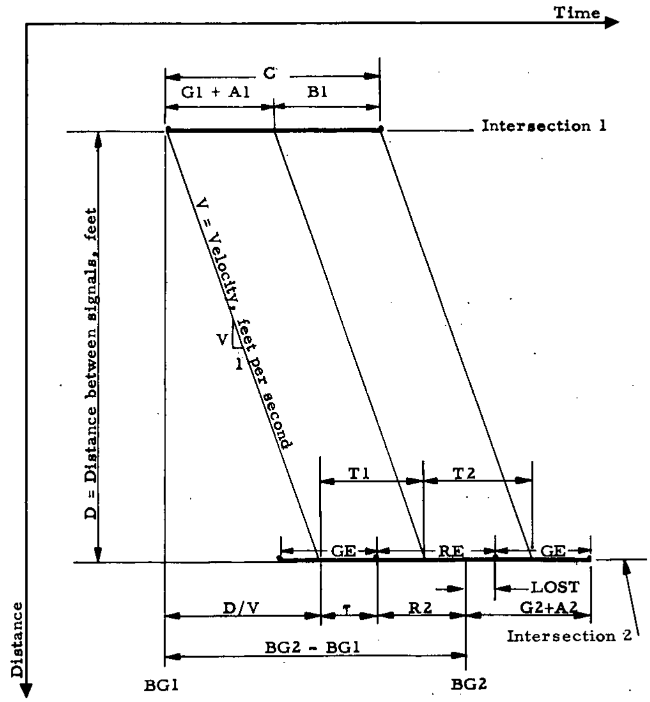

Figure A-2. Time-distance relationships.

The equation of state-of-queue size may be written:

$$Q_t = Q_{(t-\Delta t)} + (QCR)(\Delta t) \qquad \text{(A-6)}$$

in which

Q_t = queue size at time t;
Δ_t = a specified small increment of time, sec;
$Q_{(t-\Delta t)}$ = queue size at time $(t-\Delta t)$; and
QCR = queue change rate, as defined previously.

If $\Delta t = 1$ sec, then,

$$Q_t = Q_{t-1} + QCR \qquad \text{(A-7)}$$

It must also be stated that Q_t has a non-negative domain. $Q_t \geq 0$.

Computation of Delay-in-Queue

Given the equation of state-of-the-queue, queue size may be computed for any point in the signal cycle. Then, by summing queue sizes over one entire signal cycle, the magnitude of delay-in-queue may be obtained. It appears to be convenient to employ a stepwise numerical computation technique. Consider the following example.

Specify the beginning of time interval T1 as the zero time reference point, $t = 0$. By previous definition then, the end of time interval RE occurs at time $t = \tau$. Therefore,

$$Q_t = 0 \text{ for } t = \tau$$

Then it is possible to proceed stepwise computing Q_t for successive 1-sec intervals throughout one signal cycle, from $t = \tau + 1$ to $t = \tau + C$, by applying the equation of state:

$$Q_t = Q_{t-1} + QCR \qquad \text{(A-8)}$$

This results in a polygon-shaped graph of queue size (see Fig. A-3). It can be seen from this example that:

QCR = Q1 for $\tau < t \leq$ T1;
QCR = Q2 for T1 $< t \leq \tau +$ RE;
QCR = Q2—S for $\tau +$ RE $< t \leq$ C; and
QCR = Q1—S for C $< t \leq \tau +$ C.

Similarly shaped queue-size polygons result when τ is located differently with respect to T1 and T2, as shown in Figure A-4. In each case, the applicable QCR's can be quickly found.

In general and formalized terms, QCR is determined as follows:

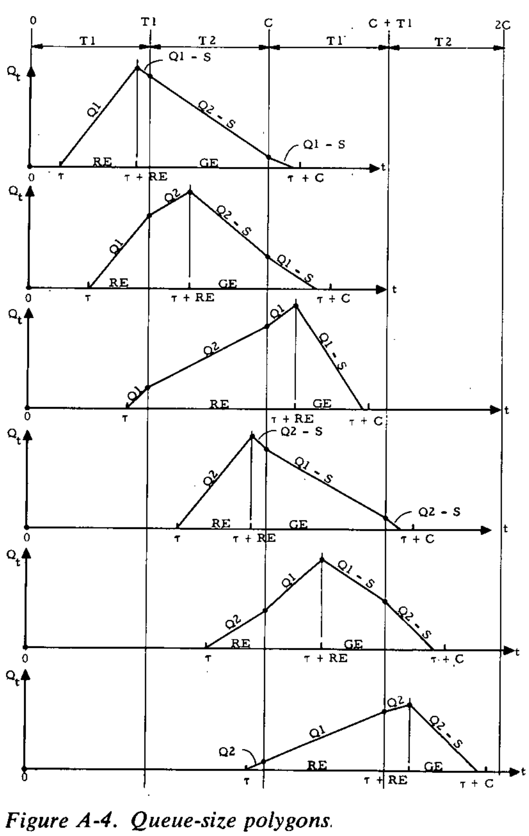

Figure A-4. Queue-size polygons.

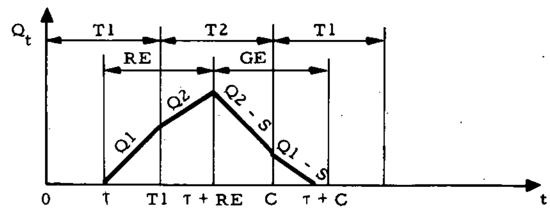

Figure A-3.

1. If $\tau < t \leq \tau +$ RE, then:
   QCR = Q1 if $0 < t \leq$ T1, or C $< t \leq$ C + T1.
   QCR = Q2 if T1 $< t \leq$ C, or C + T1 $< t \leq$ 2C.
2. If $\tau +$ RE $< t \leq \tau +$ C, then:
   QCR = Q1—S if $0 < t \leq$ T1, or C $< t \leq$ C + T1.
   QCR = Q2—S if T1 $< t \leq$ C, or C + T1 $< t \leq$ 2C.

Once the queue size has been computed for successive 1-sec intervals over one signal cycle, delay-in-queue can be determined by summation.

$$\text{Delay-in-Queue} = \sum_{t=\tau}^{\tau+C} Q_t, \text{ veh-sec per signal cycle} \qquad \text{(A-9)}$$

Computation of Delay/Difference-of-Offset Relationship

The delay/difference-of-offset relationship for the link is determined by computing delay-in-queue for all integer values of τ from 0 to C. And since τ is directly related to the offset difference, φ, i.e.,

$$\varphi = [D/V + \tau + R2] \text{ Modulo C} \qquad \text{(A-10)}$$

it should be clear that by computing delay-in-queue for all values of τ, it has in effect been computed for all values of φ, the offset difference.

The foregoing work represents an extension of the ideas that Webster (*28*) used to obtain the first term of his now classic equation:

$$d = \frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2_q(1-x)} + \text{empirical correction} \qquad \text{(A-11)}$$

in which

d = average delay per vehicle on approach;
c = cycle time;
$\lambda = \dfrac{\text{Effective Green}}{\text{Cycle}}$; and
$x = \dfrac{q}{\lambda\ (\text{saturation flow})}$.

COMPUTER PROGRAM

Because the computational method is laborious and must be applied to each network link individually, a computer program was prepared to perform the work. The program was written in FORTRAN IV language for operation under the IBSYS monitor system of the IBM 7094. The program can easily be compiled and used on other hardware. The program has been debugged and is fully operational.

Input Data

All the input data needed by the program are supplied on a single punched card. The program considers only one link at a time, as represented by one data card. However, any number of data cards may be stacked for sequential processing. The content and format of the data card are shown in Figure A-5.

Field Starts in Column		Symbol	Description
1	7 2	RUN	Run number (alphanumeric)
4	060	IC	Signal cycle length, sec
7	26	IG1	Green interval, intersection 1, sec
9	26	IG2	Green interval, intersection 2, sec
11	4	IA	Amber interval, sec
12	.560	SAT	Single-lane saturation flow, vps
16	5	LOST	Lost time per green interval, sec
17	0880	ID	Distance between signals, feet
21	2.	N	Number of through lanes, intersection 2
23	0600.	AST	Straight-through traffic arriving at tail, vph
28	0150.	ALT	Left turns arriving at tail, vph
33	0250.	ART	Right turns arriving at tail, vph
38	1400.	AVH	Total volume at head, vph
43	44	IV	Average free-flow velocity, ft /sec

Figure A-5. Sample data card.

Program Detail

Figure A-6 is a flow chart of the computational logic. A listing of the FORTRAN program code is shown in Figure A-7.

Output

Output data are delivered at the conclusion of processing of each data set (i.e., each data card). Output is in two sections. First, the input data are listed; then the results of the computations are tabulated. For every possible value of offset difference, in 1-sec increments, the following are tabulated:

1. TAU—the time differential between the leading edge of the straight-through time band and the succeeding end of the effective green time interval at intersection 2, sec.
2. PHI—the difference-of-offset of the signals at the link head and link tail, sec.
3. QSUM—the total delay-in-queue during one signal cycle, veh-sec per signal cycle.
4. DPV—the average delay per vehicle, veh-sec per vehicle.
5. QAVE—the average queue length, vehicles. This is synonymous with the "total delay rate"; e.g., in veh-hr per hr, veh-min per min, or veh-sec per sec.

A sample output tabulation is shown in Figure A-8.

PROGRAM RESULTS

The delay/difference-of-offset computer program was used in this project as one method of determining signal offset plans along the Pico arterial. Typical results derived from the program are shown in Figure A-9, where delay is plotted as a function of the offset between each pair of traffic signals on Pico Boulevard. The results shown are for peak-period-operation conditions using a 60-sec cycle length and the existing splits. Delays in the eastbound and westbound directions between each pair of signals have been summed for the illustrations.

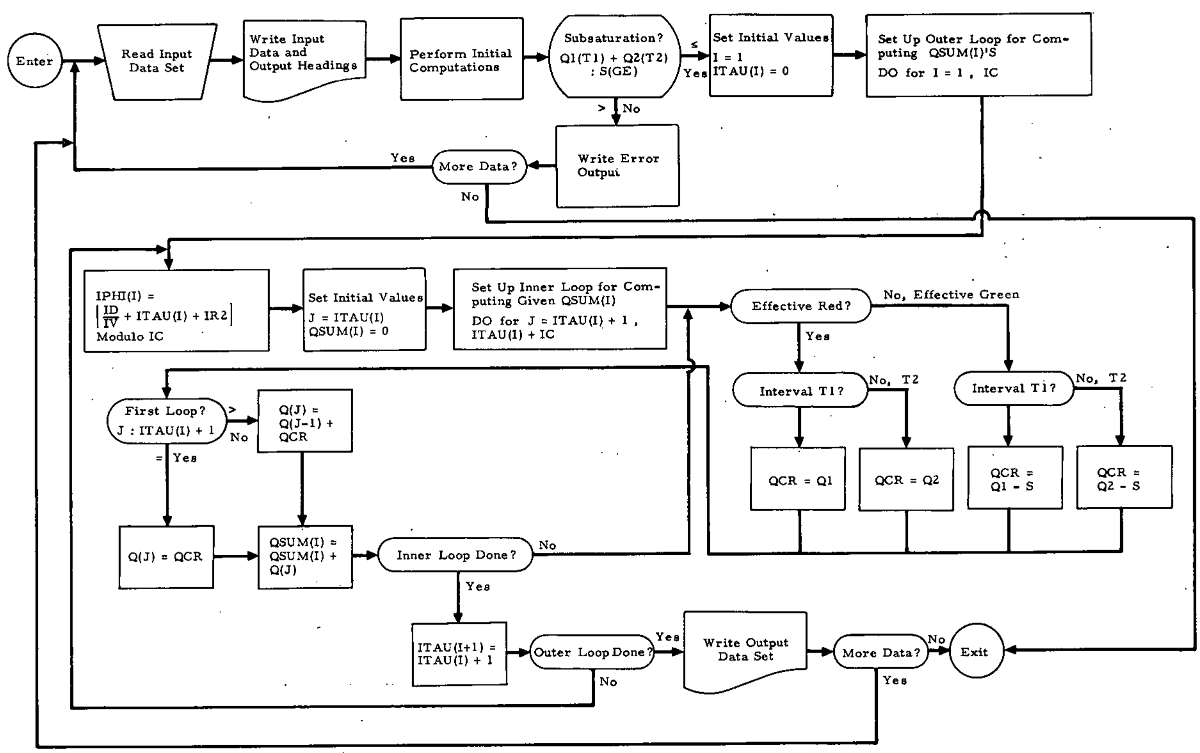

Figure A-6. Flow chart, theoretical delay/difference-of-offset program.

```
C       DELAY-DIFFERENCE OF OFFSET COMPUTATIONS, UNIDIRECTIONAL LINK,
C       TWO UNIFORM ARRIVAL RATES.
        DIMENSION ITAU(151),IPHI(151),QSUM(151),Q(301),DPV(151),QAVE(151)
C       READ INPUT DATA
  100   READ (5,101) RUN,IC,IG1,IG2,IA,SAT,LOST,ID,W,AST,ALT,ART,AVH,IV
  101   FORMAT (A3,I3,I2,I2,I1,F4.3,I1,I4,F2.0,F5.0,F5.0,F5.0,F5.0,I2)
C       WRITE HEADING FOR INPUT DATA
        WRITE (6,102)
  102   FORMAT(1H1, 5HINPUT)
        WRITE (6,103)
  103   FORMAT(1H0, 59HRUN  C  G1 G2 A SAT  LOST  D  W     ST     LT     RT
       1  VH   V)
C       WRITE INPUT DATA
        WRITE (6,104) RUN,IC,IG1,IG2,IA,SAT,LOST,ID,W,AST,ALT,ART,AVH,IV
  104   FORMAT (1H ,A3,1X,I3,1X,I2,1X,I2,1X,I1,1X,F4.3,2X,I1,2X,I4,1X,F2.0
       1,1X,F5.0,1X,F5.0,1X,F5.0,1XF5.0,1X,I2)
C       WRITE HEADING FOR OUTPUT DATA
        WRITE (6,105)
  105   FORMAT (1H0, 6HOUTPUT)
        WRITE (6,106)
  106   FORMAT (1H0,34HTAU   PHI    QSUM      DPV      QAVE)
C       PERFORM INITIAL COMPUTATIONS
  200   IR1=IC-IG1-IA
        IR2=IC-IG2-IA
        IT1=IG1&IA
        T1=IT1
        IT2=IR1
        T2=IT2
        IGE=IG2&IA-LOST
        IRE=IC-IGE
        GE=IGE
        AVT=AST&ALT&ART
        ANC=AVH-AVT
        C=IC
        Q1= AST/((T1/C)*3600.)&ANC/3600.
        Q2= (ALT&ART)/((T2/C)*3600.)&ANC/3600.
        S=W*SAT
C       CHECK SUBSATURATION
  300   IF(S*GE-(Q1*T1&Q2*T2))301,400,400
C       DUMBHEAD OUTPUT
  301   WRITE (6,302)
  302   FORMAT (1H0, 31HDUMBHEAD,THIS IS SUPERSATURATED)
        GO TO 900
C       SET INITIAL TAU
  400   I=1
        ITAU(I)=0
C       SET UP OUTER LOOP TO COMPUTE QSUM FOR ALL TAUS
        DO 706 I=1,IC
C       COMPUTE PHI MODULO C
  401   IPHI(I)=ID/IV&ITAU(I)&IR2
  402   IF(IC-IPHI(I))403,403,404
  403   IPHI(I)=IPHI(I)-IC
        GO TO 402
C       SET INITIAL J AND QSUM
  404   J=ITAU(I)
        QSUM(I)=0
C       SET INNER LOOP TO COMPUTE QSUM FOR GIVEN TAU
  405   ISTA=ITAU(I)&1
        IFIN=ITAU(I)&IC
  406   DO 705 J=ISTA,IFIN
C       IS EFFECTIVE RED OR EFFECTIVE GREEN ON
        IF(J-(ITAU(I)&IRE))500,500,600
C       IS T1 OR T2 ON (EFFECTIVE RED)
  500   IF(J-IT1)510,510,501
  501   IF(J-IC) 520,520,502
  502   IF(J-(IC&IT1))510,510,520
  510   QCR=Q1
        GO TO 700
  520   QCR=Q2
        GO TO 700
C       IS T1 OR T2 ON (EFFECTIVE GREEN)
  600   IF(J-IT1)610,610,601
  601   IF(J-IC) 620,620,602
  602   IF(J-(IC&IT1))610,610,620
  610   QCR=Q1-S
        GO TO 700
  620   QCR=Q2-S
        GO TO 700
C       COMPUTE Q AND ACCUMULATE QSUM
  700   IF(J-(ITAU(I)&1))701,701,702
  701   Q(J)=QCR
        GO TO 703
  702   Q(J)=Q(J-1)&QCR
  703   IF(Q(J))704,705,705
  704   Q(J)=0.
  705   QSUM(I)=QSUM(I)&Q(J)
C       END OF INNER LOOP
        DPV(I)=(QSUM(I)*3600.)/(AVH*C)
        QAVE(I)=QSUM(I)/C
  706   ITAU(I&1)=ITAU(I)&1
C       END OF OUTER LOOP
C       WRITE OUTPUT DATA
  800   DO 802 I=1,IC
        WRITE (6,801)ITAU(I),IPHI(I),QSUM(I),DPV(I),QAVE(I)
  801   FORMAT (1H ,I3,3X,I3,3X,F6.1,3X,F5.1,3X,F5.2)
  802   CONTINUE
C       IS THERE MORE DATA
  900   GO TO 100
        END
```

Figure A-7. FORTRAN code, delay/difference-of-offset program.

INPUT

RUN	C	G1	G2	A	SAT	LOST	D	W	ST	LT	RT	VH	V
T 2	60	26	26	4	.500	5	880	2.	800.	150.	250.	1400.	44

OUTPUT

TAU	PHI	QSUM	DPV	QAVE
0	50	489.5	21.0	8.16
1	51	483.3	20.7	8.05
2	52	476.8	20.4	7.95
3	53	470.3	20.2	7.84
4	54	463.7	19.9	7.73
5	55	457.0	19.6	7.62
6	56	450.3	19.3	7.51
7	57	443.7	19.0	7.39
8	58	437.0	18.7	7.28
9	59	430.3	18.4	7.17
10	0	423.7	18.2	7.06
11	1	417.0	17.9	6.95
12	2	410.3	17.6	6.84
13	3	403.7	17.3	6.73
14	4	397.0	17.0	6.62
15	5	390.3	16.7	6.51
16	6	383.7	16.4	6.39
17	7	377.0	16.2	6.28
18	8	370.3	15.9	6.17
19	9	363.7	15.6	6.06
20	10	357.0	15.3	5.95
21	11	350.3	15.0	5.84
22	12	343.7	14.7	5.73
23	13	337.0	14.4	5.62
24	14	330.3	14.2	5.51
25	15	323.7	13.9	5.39
26	16	317.0	13.6	5.28
27	17	310.3	13.3	5.17
28	18	303.7	13.0	5.06
29	19	297.0	12.7	4.95
30	20	290.3	12.4	4.84
31	21	296.4	12.7	4.94
32	22	302.8	13.0	5.05
33	23	309.5	13.3	5.16
34	24	316.2	13.5	5.27
35	25	322.8	13.8	5.38
36	26	329.5	14.1	5.49
37	27	336.2	14.4	5.60
38	28	342.8	14.7	5.71
39	29	349.5	15.0	5.82
40	30	356.2	15.3	5.94
41	31	362.8	15.5	6.05
42	32	369.5	15.8	6.16
43	33	376.2	16.1	6.27
44	34	382.8	16.4	6.38
45	35	389.5	16.7	6.49
46	36	396.2	17.0	6.60
47	37	402.8	17.3	6.71
48	38	409.5	17.5	6.82
49	39	416.2	17.8	6.94
50	40	422.8	18.1	7.05
51	41	429.5	18.4	7.16
52	42	436.2	18.7	7.27
53	43	442.8	19.0	7.38
54	44	449.5	19.3	7.49
55	45	456.2	19.5	7.60
56	46	462.8	19.8	7.71
57	47	469.5	20.1	7.82
58	48	476.2	20.4	7.94
59	49	482.8	20.7	8.05

Figure A-8. Sample output, theoretical delay/difference-of-offset program.

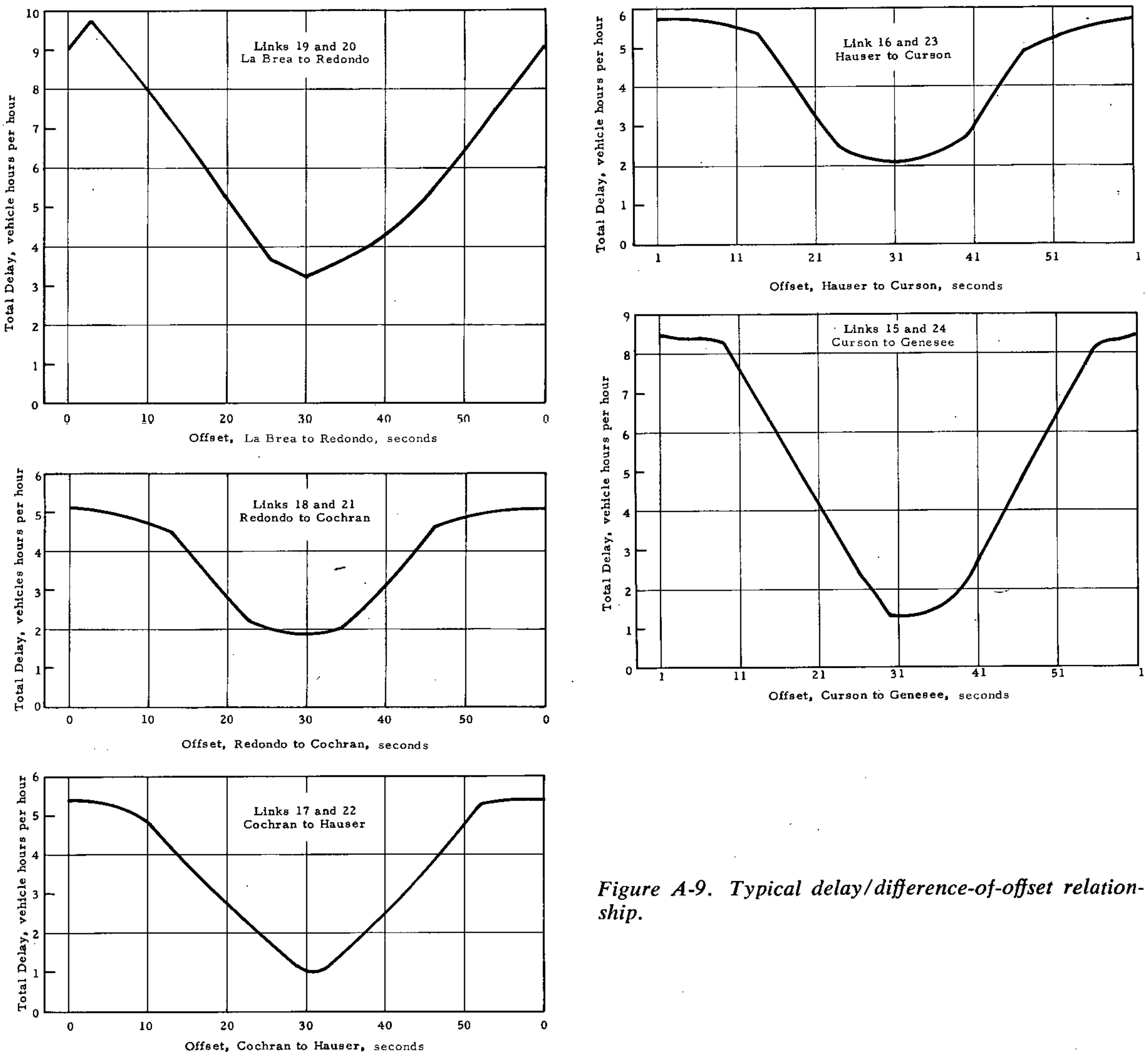

Figure A-9. Typical delay/difference-of-offset relationship.

APPENDIX B

ALTERNATIVE STRATEGIC TRAFFIC-SIGNAL-TIMING PLANS

Alternative methodologies were used to formulate strategic traffic-signal-timing plans in the six-intersection Pico arterial system. The resulting fixed-time plans appear in this appendix.

Condition 1: Represents the existing 60-sec cycle plan currently in use in the system. It is used throughout the day.

Condition 2a: Uses both the splits and offsets given by implementing Yardeni's time-space design model. Yardeni's program was run with traffic data from the peak period and offpeak period to yield two different plans, although both use a 60-sec system cycle length.

Condition 2b: The existing 60-sec cycle and existing splits are maintained for all intersections, and the offsets are determined from exercising the Yardeni program. Alternative plans are found for peak and offpeak periods.

Condition 2c: Uses Webster's technique for determining optimum cycle length for the system and the splits at each intersection, and the offset plan given by another exercising of the Yardeni program. During the peak period, a 60-sec system cycle length is still required, but during the offpeak period the cycle can be reduced to 40 sec and still accommodate traffic demands.

Condition 3a: The existing 60-sec cycle and the existing splits at all intersections are maintained; the offset plan is formulated by implementing Little's maximal bandwidth computer program. Alternative plans are created for peak period and offpeak period.

Condition 3b: Uses both Webster's optimum cycle and splits as determined previously in Condition 2c, and the offset plan given by a second exercising of the Little program. A 60-sec cycle is used for the peak period and a 40-sec cycle is used for the offpeak period.

Condition 4a: The existing cycle and splits are used in conjunction with results obtained from the delay/difference-of-offsets program, developed by the research agency, to determine an offset plan. Different plans are formulated for peak and offpeak periods.

Condition 4b: Webster's optimum cycle and splits are coupled with offsets given by further exercising of the delay/difference-of-offset program.

Table B-1 is a summary of the strategic signal-timing alternatives developed for the offpeak and peak periods. Figures B-1 through B-8 are time-space diagrams for all the peak-period signal-timing alternatives developed. Figures B-9 through B-15 are the offpeak-period time-space diagrams.

TABLE B-1

SUMMARY OF STRATEGIC SIGNAL-TIMING ALTERNATIVES, OFFPEAK AND PEAK PERIODS

	OFFPEAK PERIOD (SEC)						PEAK PERIOD (SEC)					
	PICO OFFSETS [a]		PICO GREEN		CROSS-STREET GREEN		PICO OFFSETS [a]		PICO GREEN		CROSS-STREET GREEN	
CROSS STREET	WB	EB	REF.[b]	DUR.[c]	REF.[b]	DUR.[d]	WB	EB	REF.[b]	DUR.[c]	REF.[b]	DUR.[d]
(*a*) CONDITION 1: EXISTING TIMING												
La Brea			6	28	34	32			6	28	34	32
	26	34					26	34				
Redondo			32	36	8	24			32	36	8	24
	24	36					24	36				
Cochran			56	38	34	22			56	38	34	22
	42	18					42	18				
Hauser			38	36	14	24			38	36	14	24
	56	4					56	4				
Curson			34	36	10	24			34	36	10	24
	26	34					26	34				
Genesee			0	30	30	30			0	30	30	30
(*b*) CONDITION 2A: YARDENI SPLITS—YARDENI OFFSETS												
La Brea			0	24	24	36			0	24	24	36
	38	22					26	34				
Redondo			38	36	14	24			26	34	0	26
	28	32					32	28				
Cochran			6	36	42	24			58	40	38	20
	34	26					28	32				
Hauser			40	36	16	24			26	34	0	26
	24	36					34	26				
Curson			4	36	40	24			0	38	38	22
	52	8					30	30				
Genesee			56	36	32	24			30	42	12	18
(*c*) CONDITION 2B: EXISTING SPLITS—YARDENI OFFSETS												
La Brea			0	28	28	32			0	28	28	32
	38	22					30	30				
Redondo			38	36	14	24			30	36	6	24
	30	30					32	28				
Cochran			8	38	46	22			2	38	40	22
	32	28					24	36				
Hauser			40	36	16	24			26	36	2	24
	26	34					34	26				
Curson			6	36	42	24			0	36	36	24
	50	10					28	32				
Genesee			56	30	26	30			28	30	58	30

TABLE B-1 (*Continued*)

CROSS STREET	OFFPEAK PERIOD (SEC): PICO OFFSETS[a] WB	PICO OFFSETS[a] EB	PICO GREEN REF.[b]	PICO GREEN DUR.[c]	CROSS-STREET GREEN REF.[b]	CROSS-STREET GREEN DUR.[d]	PEAK PERIOD (SEC): PICO OFFSETS[a] WB	PICO OFFSETS[a] EB	PICO GREEN REF.[b]	PICO GREEN DUR.[c]	CROSS-STREET GREEN REF.[b]	CROSS-STREET GREEN DUR.[d]
(*d*) CONDITION 2C: WEBSTER SPLITS—YARDENI OFFSETS												
La Brea			0	18	18	22			0	28	28	32
	20	20					26	34				
Redondo			20	24	4	16			26	34	0	26
	10	30					32	28				
Cochran			30	24	14	16			58	40	38	20
	36	4					28	32				
Hauser			26	24	10	16			26	36	2	24
	22	18					34	26				
Curson			8	24	32	16			0	40	40	20
	8	32					30	30				
Genesee			16	24	0	16			30	44	14	16
(*e*) CONDITION 3A: EXISTING SPLITS—LITTLE OFFSETS												
La Brea			0	28	28	32			0	26	26	34
	30	30					30	30				
Redondo			30	36	6	24			30	36	6	24
	30	30					30	30				
Cochran			0	38	38	22			0	38	38	22
	30	30					30	30				
Hauser			30	36	6	24			30	36	6	24
	0	0					56	4				
Curson			30	36	6	24			26	36	2	24
	30	30					32	28				
Genesee			0	30	30	30			58	30	28	30
(*f*) CONDITION 3B: WEBSTER SPLITS—LITTLE OFFSETS												
La Brea			0	18	18	22			30	28	58	32
	0	0					30	30				
Redondo			0	24	24	16			0	34	34	26
	20	20					30	30				
Cochran			20	24	4	16			30	40	10	20
	20	20					30	30				
Hauser			0	24	24	16			0	36	36	24
	20	20					30	30				
Curson			20	24	4	16			30	40	10	20
	0	0					30	30				
Genesee			20	24	4	16			0	44	44	16
(*g*) CONDITION 4A: EXISTING SPLITS—DELAY/DIFFERENCE OFFSETS												
La Brea			0	28	28	32			0	28	28	32
	20	40					22	38				
Redondo			20	36	56	24			22	36	58	24
	30	30					30	30				
Cochran			50	38	28	22			52	38	30	22
	32	28					30	30				
Hauser			22	36	58	24			22	36	58	24
	32	28					32	28				
Curson			54	36	30	24			54	36	30	24
	32	28					30	30				
Genesee			26	30	56	30			24	30	54	30
(*h*) CONDITION 4B: WEBSTER SPLITS—DELAY/DIFFERENCE OFFSETS												
La Brea			0	18	18	22			0	28	28	32
	2	38					26	34				
Redondo			2	24	26	16			26	34	0	26
	16	24					28	32				
Cochran			18	24	2	16			54	40	34	20
	28	12					34	26				
Hauser			6	24	30	16			28	36	4	24
	18	22					28	32				
Curson			24	24	8	16			56	40	36	20
	26	14					30	30				
Genesee			10	24	34	16			26	44	10	16

[a] Offset is the difference between references of successive intersections in the direction of travel indicated.
[b] Reference is the time between a common (zero) reference point in the system cycle and the start of the green interval at the individual intersection.
[c] Duration of green-plus-yellow intervals on Pico Boulevard.
[d] Duration of green-plus-yellow intervals on the cross streets.

Note: System cycle length = 40 sec for Conditions 2c, 3b, and 4b of offpeak period; for all other conditions (offpeak and peak), system cycle length = 60 sec.

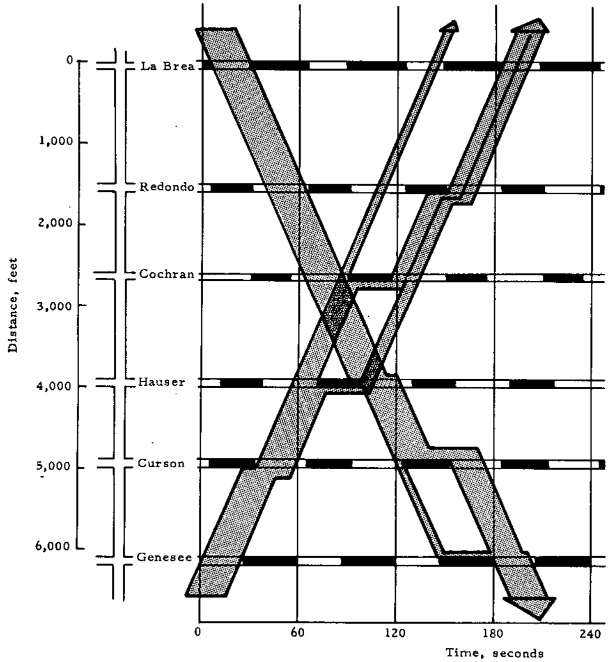

Figure B-1. Condition 1: Existing timing (peak period).

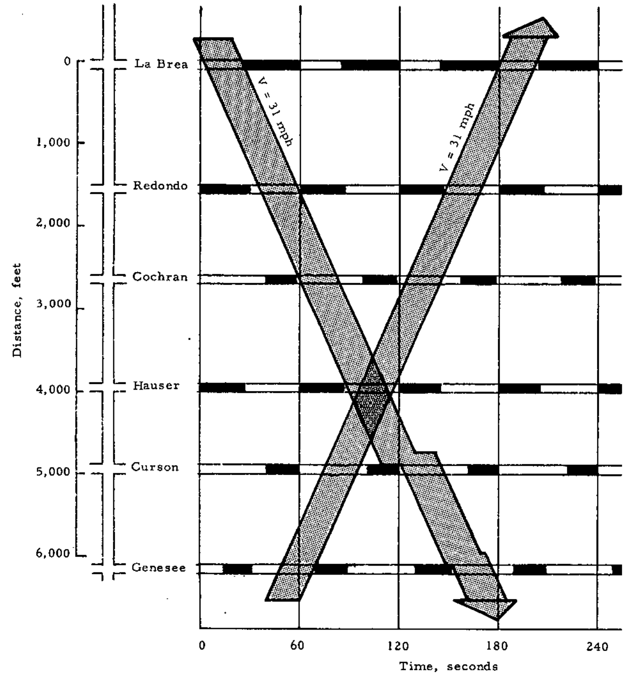

Figure B-2. Condition 2a: Yardeni splits—Yardeni offsets (peak period).

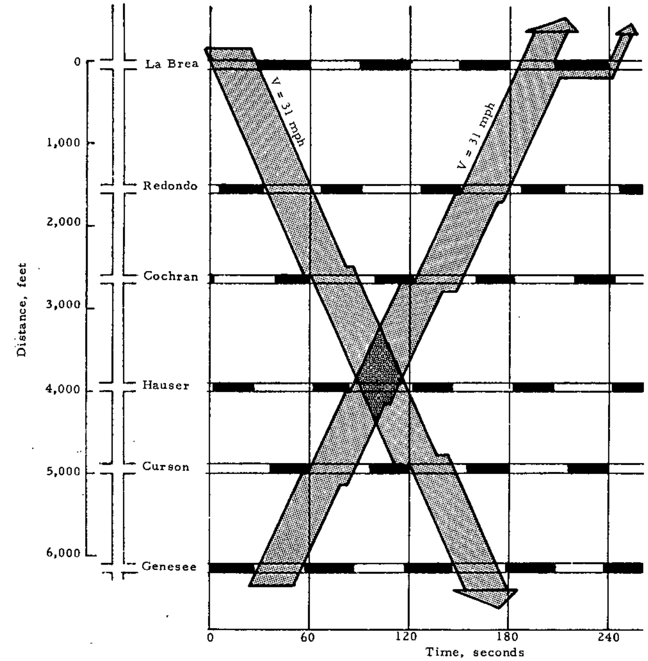

Figure B-3. Condition 2b: Existing splits—Yardeni offsets (peak period).

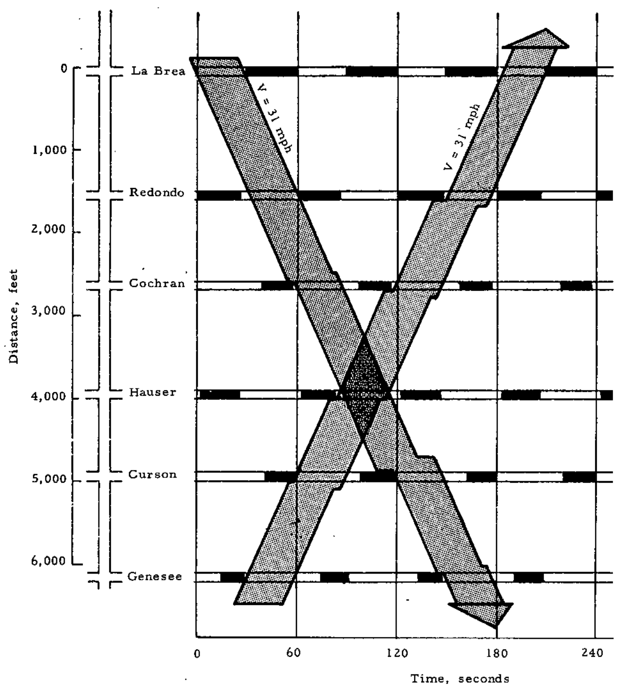

Figure B-4. Condition 2c: Webster splits—Yardeni offsets (peak period).

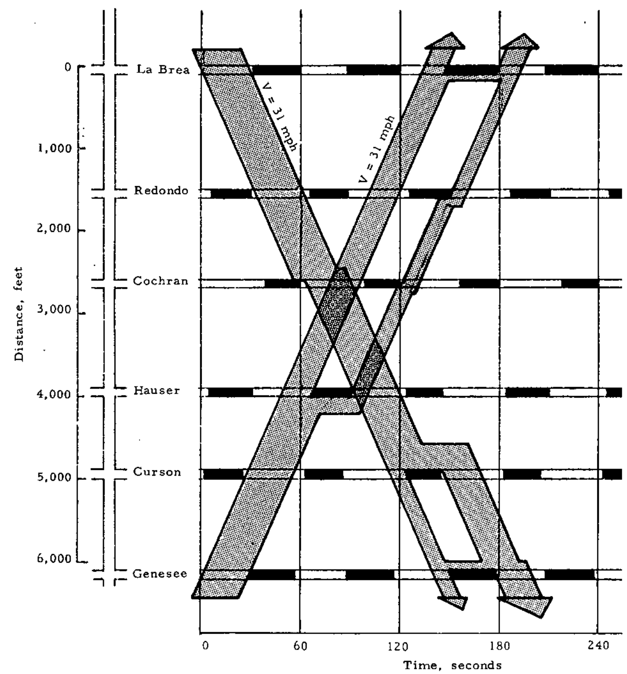

Figure B-5. Condition 3a: Existing splits—Little offsets (peak period).

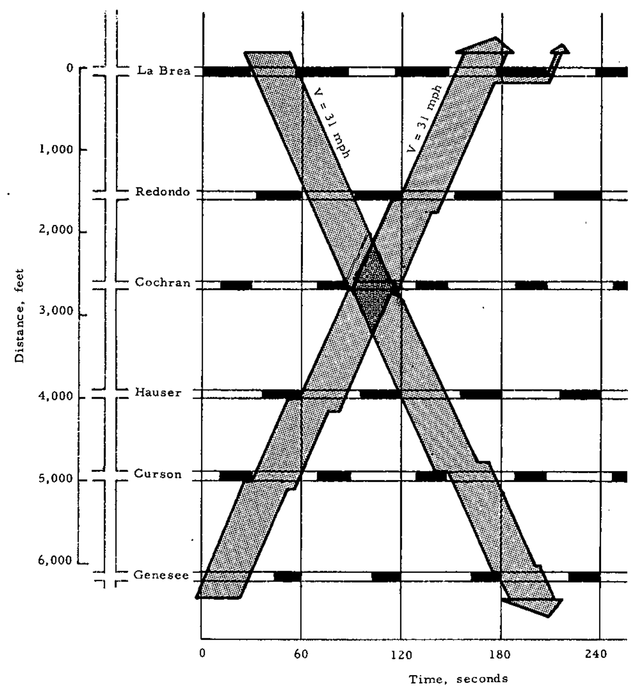

Figure B-6. Condition 3b: Webster splits—Little offsets (peak period).

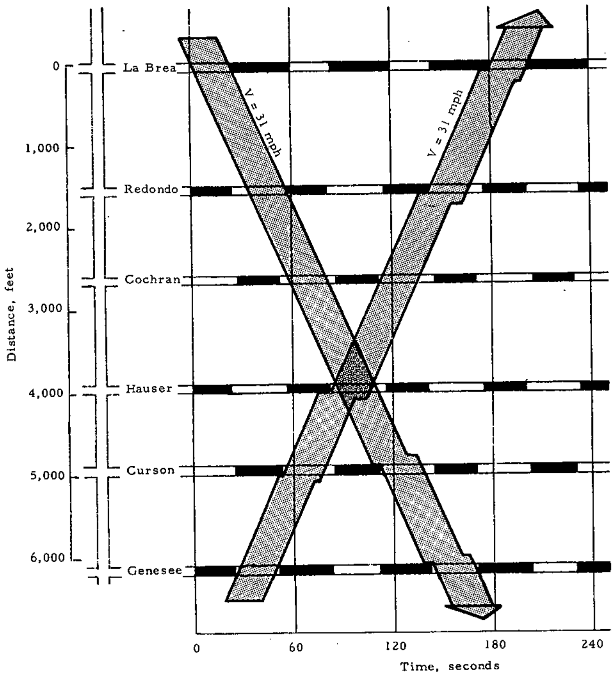

Figure B-7. Condition 4a: Existing splits—delay/difference offsets (peak period).

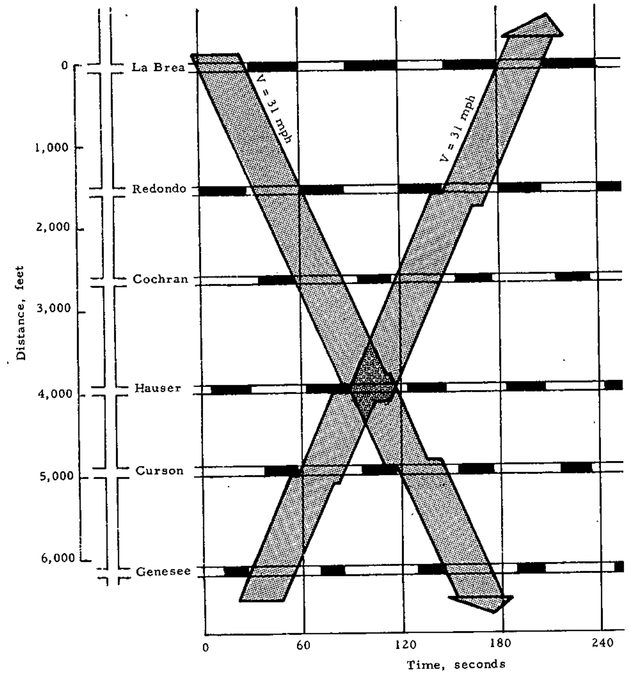

Figure B-8. Condition 4b: Webster splits—delay/difference offsets (peak period).

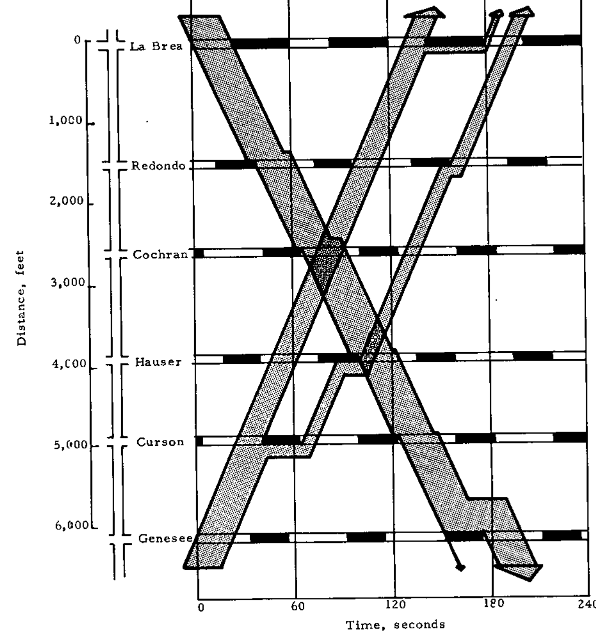

Figure B-9. Condition 2a: Yardeni splits—Yardeni offsets (offpeak period).

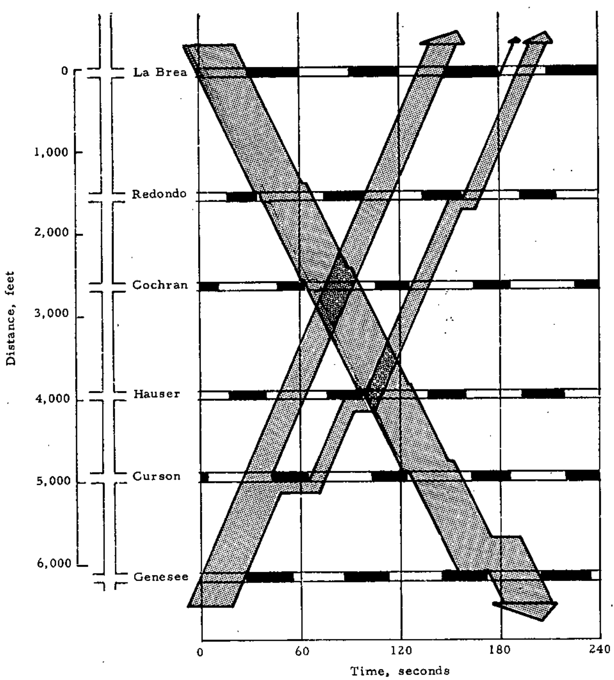

Figure B-10. Condition 2b: Existing splits—Yardeni offsets (offpeak period).

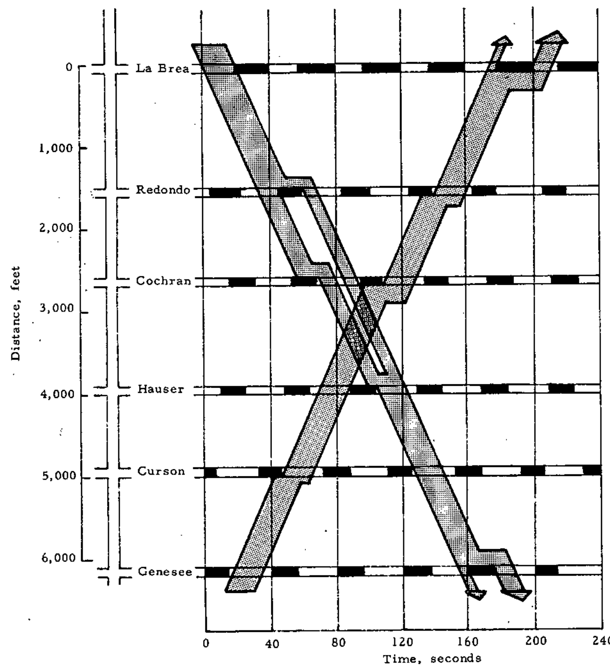

Figure B-11. Condition 2c: Webster splits—Yardeni offsets (offpeak period), 40-sec cycle.

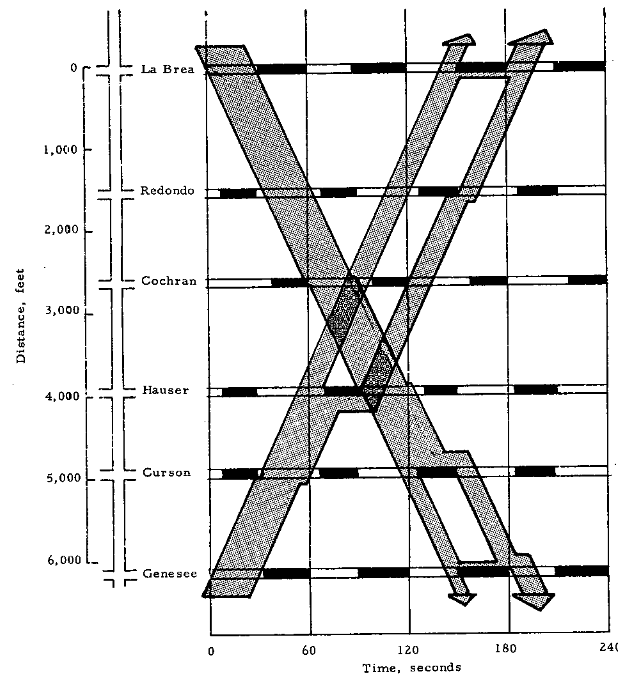

Figure B-12. Condition 3a: Existing splits—Little offsets (offpeak period).

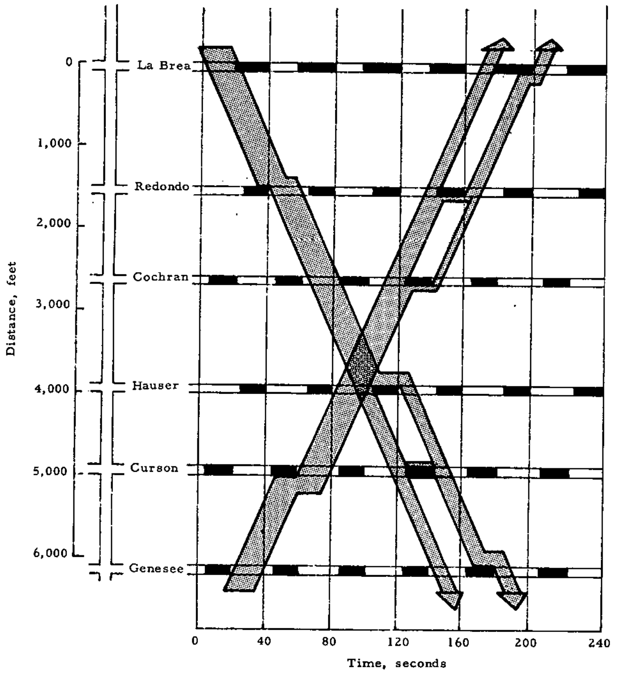

Figure B-13. Condition 3b: Webster splits—Little offsets (offpeak period), 40-sec cycle.

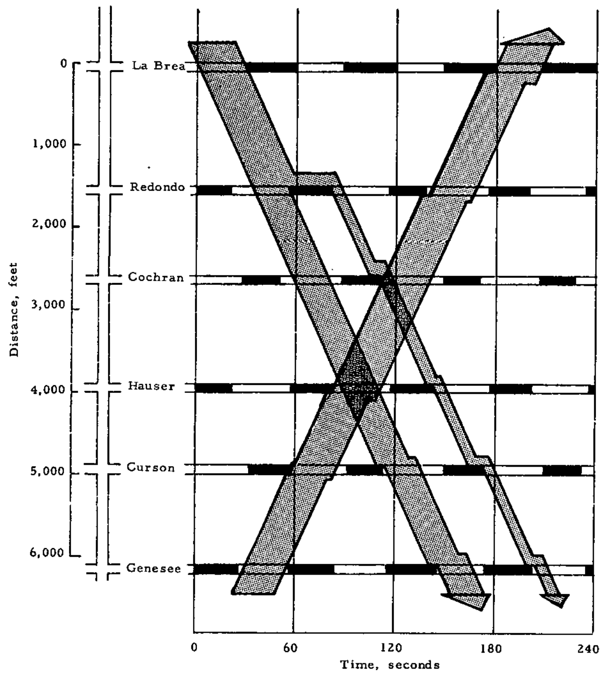

Figure B-14. Condition 4a: Existing splits—delay/difference offsets (offpeak period).

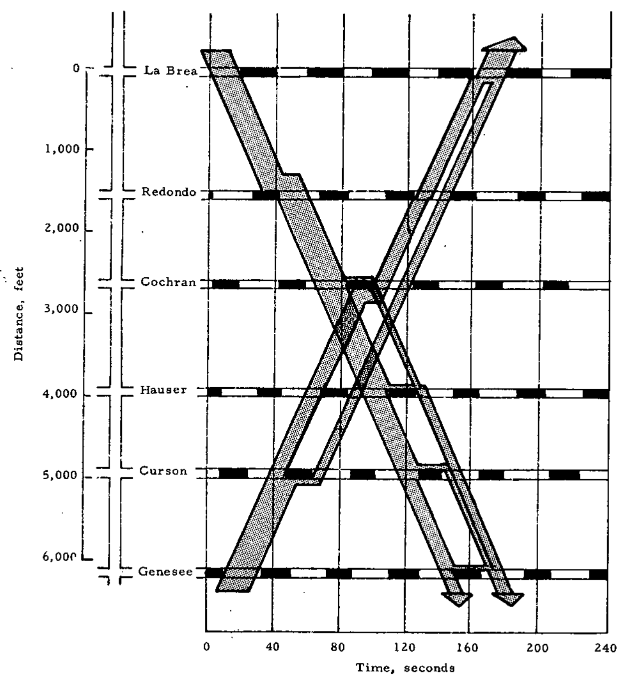

Figure B-15. Condition 4b: Webster splits—delay/difference offsets (offpeak period), 40-sec cycle.

APPENDIX C

DETAILED RESULTS OF SIMULATION TESTS

Extensive simulation exercises were performed with the research agency's traffic network simulation model (TRANS III) for the purpose of testing the effectiveness of alternative traffic-signal operation plans. For each traffic-signal system alternative, including existing settings, simulations were run to estimate effectiveness during an afternoon offpeak period (2:30-3:30 PM) and an afternoon peak period (4:30-5:30 PM) in the Pico arterial system. Five replications were executed under each condition to enable determination of confidence intervals of traffic operation characteristics.

Detailed tabulations of the simulation results appear in this appendix. Table C-1 gives offpeak-period and peak-period results. Total travel time, total vehicle-miles traveled, and average speed values shown are for the entire arterial system; i.e., they include all of the Pico Boulevard links and the signalized side-street approach links.

Table C-1 also gives the mean of each set of five replications and the confidence interval estimates of the means with confidence coefficient, $1 - \alpha = 0.95$. For small samples, confidence intervals for the true mean, μ, with confidence coefficient, $1 - \alpha$, are given by

$$\bar{x} \pm \frac{t_{(n-1,\ 1-1/2\alpha)}s}{\sqrt{n-1}} \qquad \text{(C-1)}$$

in which $\bar{x}$ is the sample mean, n the sample size, s the sample standard deviation, and $t_{n-1,\ 1-1/2\alpha}$ is that point on the students t-distribution with $(n - 1)$ degrees of freedom having probability $\alpha/2$ of being exceeded.

TABLE C-1

SIMULATION RESULTS, OFFPEAK AND PEAK PERIODS

RUN NUMBER	OFFPEAK PERIOD TOTAL TRAVEL TIME (VEH-SEC)	OFFPEAK PERIOD TOTAL VEH-MILES	OFFPEAK PERIOD AVERAGE SPEED (MPH)	PEAK PERIOD TOTAL TRAVEL TIME (VEH-SEC)	PEAK PERIOD TOTAL VEH-MILES	PEAK PERIOD AVERAGE SPEED (MPH)
(a) Condition 1: Existing Timing						
1	335,826	2,063	22.11	523,726	2,956	20.31
2	353,262	2,164	22.05	517,106	2,920	20.32
3	349,514	2,133	21.97	513,950	2,905	20.34
4	346,270	2,122	22.06	525,074	2,942	20.17
5	332,608	2,043	22.11	523,884	2,923	20.08
Average	343,500	2,105	22.06	520,750	2,929	20.24
95% Confidence interval	331,150–355,850	2,035–2,175	21.98–22.14	513,920–527,580	2,901–2,957	20.08–20.40
(b) Condition 2a: Yardeni Splits—Yardeni Offsets						
1	328,880	2,055	22.49	495,212	2,922	21.24
2	355,056	2,186	22.16	478,356	2,821	21.23
3	331,390	2,078	22.57	515,160	3,041	21.25
4	331,478	2,077	22.55	495,650	2,981	21.65
5	336,810	2,108	22.53	511,432	2,884	20.30
Average	336,720	2,101	22.46	499,162	2,930	21.13
95% Confidence interval	321,940–351,500	2,030–2,172	22.29–22.63	478,720–519,600	2,812–3,048	20.44–21.82
(c) Condition 2b: Existing Splits—Yardeni Offsets						
1	340,404	2,097	22.17	506,212	3,000	21.34
2	346,128	2,111	21.95	487,496	2,863	21.14
3	335,380	2,032	21.81	493,668	2,868	20.91
4	346,638	2,111	21.92	504,918	2,979	21.23
5	343,686	2,103	22.03	500,384	2,918	21.00
Average	342,450	2,091	21.98	498,535	2,926	21.12
95% Confidence interval	335,985–348,915	2,045–2,137	21.79–22.17	487,590–509,480	2,839–3,013	20.88–21.36

TABLE C-1 (*Continued*)

RUN NUMBER	OFFPEAK PERIOD TOTAL TRAVEL TIME (VEH-SEC)	OFFPEAK PERIOD TOTAL VEH-MILES	OFFPEAK PERIOD AVERAGE SPEED (MPH)	PEAK PERIOD TOTAL TRAVEL TIME (VEH-SEC)	PEAK PERIOD TOTAL VEH-MILES	PEAK PERIOD AVERAGE SPEED (MPH)
(*d*) Condition 2c: Webster Splits—Yardeni Offsets						
1	309,494	2,055	23.90	493,398	2,918	21.29
2	302,918	2,023	24.04	484,046	2,869	21.33
3	315,784	2,102	23.97	491,674	2,885	21.12
4	316,902	2,104	23.90	506,104	2,978	21.18
5	312,972	2,085	23.99	494,408	2,910	21.18
Average	311,615	2,074	23.96	493,930	2,912	21.22
95% Confidence interval	303,785–319,445	2,026–2,122	23.88–24.04	481,930–504,930	2,854–2,970	21.10–21.34
(*e*) Condition 3a: Existing Splits—Little Offsets						
1	337,530	2,163	23.07	499,488	2,925	21.08
2	310,752	1,993	23.09	515,518	3,034	21.18
3	315,862	2,035	23.19	514,986	3,044	21.27
4	329,496	2,111	23.07	521,630	3,048	21.03
5	321,094	2,058	23.07	527,416	3,060	20.88
Average	322,950	2,072	23.10	515,800	3,022	21.09
95% Confidence interval	308,100–337,800	1,979–2,165	23.03–23.17	503,060–528,540	2,046–3,098	20.88–21.30
(*f*) Condition 3b: Webster Splits—Little Offsets						
1	315,690	2,140	24.40	491,128	2,920	21.40
2	301,526	2,049	24.46	502,382	2,970	21.28
3	309,900	2,087	24.25	499,250	2,997	21.61
4	315,242	2,126	24.28	506,586	3,018	21.44
5	308,076	2,088	24.40	477,344	2,851	21.50
Average	310,090	2,098	24.36	495,340	2,951	21.44
95% Confidence interval	302,010–318,170	2,048–2,148	24.24–24.48	479,320–511,360	2,859–3,043	21.27–21.61
(*g*) Condition 4a: Existing Splits—Delay/Difference Offsets						
1	330,466	2,085	22.71	522,876	3,033	20.88
2	331,070	2,112	22.97	515,146	3,002	20.97
3	313,618	1,990	22.84	501,386	2,955	21.21
4	320,650	2,019	22.66	500,542	2,958	21.27
5	329,226	2,087	22.82	510,350	2,996	21.13
Average	325,000	2,059	22.80	510,060	2,989	21.09
95% Confidence interval	314,410–335,590	1,988–2,130	22.63–22.97	500,620–519,500	2,942–3,036	20.96–21.22
(*h*) Condition 4b: Webster Splits—Delay/Difference Offsets						
1	308,784	2,135	24.89	491,430	2,944	21.56
2	299,190	2,062	24.81	487,466	2,925	21.60
3	306,228	2,106	24.76	511,206	3,065	21.58
4	300,612	2,079	24.90	481,168	2,863	21.42
5	302,232	2,097	24.97	477,484	2,867	21.61
Average	303,410	2,096	24.87	489,750	2,933	21.55
95% Confidence interval	297,865–308,955	2,058–2,134	24.76–24.98	471,360–508,140	2,851–3,015	21.44–21.66
(*i*) Condition 5: Traffic-Adjusted Concept, Cycle and Offset Selection Mode						
1				511,500	2,967	20.94
2				511,944	3,011	21.17
3				488,072	2,903	21.41
4				504,292	2,971	21.21
5				497,126	2,975	21.54
Average				502,590	2,965	21.25
95% Confidence interval				488,530–516,650	2,911–3,019	20.93–21.57

TABLE C-1 (*Continued*)

RUN NUMBER	OFFPEAK PERIOD TOTAL TRAVEL TIME (VEH-SEC)	OFFPEAK PERIOD TOTAL VEH-MILES	OFFPEAK PERIOD AVERAGE SPEED (MPH)	PEAK PERIOD TOTAL TRAVEL TIME (VEH-SEC)	PEAK PERIOD TOTAL VEH-MILES	PEAK PERIOD AVERAGE SPEED (MPH)
(*j*) Condition 6: Experimental Traffic-Responsive Concept, Basic Queue-Control Mode						
1	316,806	2,074	23.57	471,232	2,865	21.89
2	332,604	2,160	23.38	477,050	2,912	21.97
3	322,022	2,082	23.28	477,178	2,928	22.09
4	319,274	2,080	23.45	485,646	2,925	21.68
5	325,128	2,107	23.33	482,962	2,964	22.10
Average	323,167	2,101	23.40	478,814	2,919	21.95
95% Confidence interval	314,672–331,662	2,052–2,150	23.24–23.56	470,984–486,644	2,869–2,969	21.71–22.19
(*k*) Condition 7: Special Strategic Concept—Mixed-Cycle Length						
1				483,100	2,973	22.16
2				479,034	2,929	22.01
3				495,538	3,003	21.81
4				466,716	2,902	22.38
5				474,832	2,938	22.28
Average				479,844	2,949	22.13
95% Confidence interval				465,048–494,640	2,894–3,004	21.82–22.44

Note: System cycle length = 40 sec for Conditions 2c, 3b, and 4b of offpeak period; for all other Conditions 1 through 4b (offpeak and peak), system cycle length = 60 sec.

APPENDIX D

TRAFFIC-ADJUSTED CONTROL SYSTEMS

Analog computer systems for the centralized traffic-adjusted control of traffic on thoroughfares and in networks are available off-the-shelf. The principal manufacturers of this equipment are the Automatic Signal Division (ASD) of Laboratory for Electronics, Inc., the Crouse Hinds Company, and the Eagle Signal Division of the E. W. Bliss Company.

The Automatic Signal equipment is discussed as an example. This equipment provides the capability of selecting one of six preset traffic-signal cycle lengths based on counts of traffic actually present in the system. It also provides four offset preferences, as well as three cycle splits, at each intersection. Semiactuation on minor side streets can be provided where desired.

SYSTEM SENSING

Although any type of detector can be used for sensing traffic in the system, the usual Automatic Signal installations employ either pressure detectors or radar detectors. There is a decided advantage to sensing lanes separately. This is recommended by the equipment manufacturers. The computing circuitry can be arranged to accept inputs from several locations within the traffic network.

MASTER CONTROLLER (COMPUTER)

In some literature the entire central control device is referred to as the master controller; in other literature this assemblage is referred to as the "computer." This equipment may be installed either in a remote central control room or in a curbside cabinet. (ASD equipment is used as an example throughout this discussion.)

The basic computing element is an averaging circuit that receives detector impulses from one set of system detectors and computes the average traffic flow over a period that is generally set to between 4 and 9 min. The length of this period is chosen by the traffic engineer. The average of traffic counts over the sampling period is compared with preset reference levels to determine the appropriate traffic-signal cycle length for the traffic currently present in the street network. Inasmuch as six cycle lengths are available for selection, there are five count-level settings that serve to determine the changeover points between various cycles during periods of increasing traffic volume, and another five settings to determine the changeover points between various cycles for decreasing traffic volume.

The equipment is constructed in modular form so that the various capabilities can be multiplied as required. In the normal installation, there will be one cycle computer for each set of system detectors, where a set is a group of detectors at one location sensing the traffic going in one direction. Reversible lane operation can be provided for through the use of directional detectors. Thus, if there are to be separate measurements of northbound traffic, southbound traffic, eastbound traffic, and westbound traffic, four separate cycle computer modules will be required.

Where a grid system is being controlled, it is possible to provide north-south offsets at one time of day and east-west offsets during another time of day. When traffic is being controlled on a thoroughfare, an inbound offset can be selected in the morning and an outbound offset can be selected in the afternoon. During times of day when there is no preference of one direction over another, an average offset can be selected. Although the traffic conditions that cause a particular offset plan to be selected are chosen by the traffic engineer, the actual activation of this offset plan is dependent on the levels of traffic sensed by the system detectors.

A system selector determines which of the various cycle computers is calling for the longest cycle length. It causes this cycle length to be put into effect throughout the system. The system selector also determines the offset plan that is to be used. When the traffic demands determined by the cycle computers are equal in two directions, an average offset is used. When two different cycle computers are calling for cycles two or more steps apart, the one having the longer cycle length is considered to be the preferred direction, and the offset plan providing preference in this direction is selected. Simultaneous operation is also possible if the traffic engineer so decides.

The system selector indicates to the cycle generator which cycle length is to be in effect at any given time. The cycle lengths are adjusted by the traffic engineer in advance so that the shortest length for a cycle is not less than 40 sec and the longest length for a cycle is not greater than 120 sec.

There is provision for graphic recording of traffic volumes as a function of time.

LOCAL CONTROLLERS

The master controller sends information to local controllers, commanding them to operate on a specified cycle length and with a specified direction of offset. The circuitry at the local controller causes it to select the appropriate split for the given cycle and offset, based on settings previously made in the local controller by the traffic engineer. Three different cycle split settings can be made on each of the local controllers, and each can be used with one or more offsets by means of adjustments within the local controller.

Cycle and split changes affect only the green intervals; pedestrian intervals, yellow, and all-red periods are timed by the local controller and remain constant regardless of the total cycle length. Offset changes are made by slewing processes that spread the effect over several cycles, with not more than 17 percent of the cycle length taking place in any one cycle. This eliminates abrupt changes and dwelling of the controllers, thereby providing for a minimum of disruption of progressive traffic movement.

Where vehicle actuation on minor approaches is needed, this feature is provided in the local controller.

COMMUNICATION

When mounted in curbside cabinets, the master controller is linked to intersection controllers either by city-owned cables or by leased telephone lines. When the master controller is located in a central control room, some installations make use of radio communication for interconnection purposes.

EQUIPMENT CAPABILITY

Although it is possible in theory to have an infinite number of local controllers supplied by one master controller, there is a circuit restriction which requires that amplifiers be provided for approximately each 40 controllers added to the system. More important, however, is the matter of traffic restrictions. Traffic engineers have found that different areas or sectors of a city have different traffic characteristics. For example, the morning peak period may be from 7:00 to 8:00 AM in one sector, and from 8:00 to 9:00 AM in another. With these sectors tied to the same master controller, it will be necessary for them to operate with the same cycle length and the same offset at any given time. Thus, as the size of the city increases, it becomes desirable to provide each sector with its own traffic control system. Using the analog computing equipment described herein, this implies a separate computer installation for each sector. While hard and fast rules cannot be given, it appears that 100 intersections perhaps represent a good overall average for the size of the system that the traffic engineer might wish to operate in any given sector.

The analog equipment now available has no ability to accept inputs of speed, delay, platoon behavior, or roadway occupancy.

The following sections describe the way in which traffic-adjusted control systems can be implemented on a digital computer for experimental purposes.

AVERAGING OF VOLUME MEASUREMENTS

The following procedure may be used to simulate the volume averaging properties of the traffic-adjusted control system.*

* In commercial electronic traffic-adjusted systems the averaging is performed by analog methods, and follows the relationship:

$$\bar{y}(t) = \sum_i y_i \exp[-k(t - t_i)] \quad \text{(D-2)}$$

Although this method is simple using resistance-capacitance circuits, its use for digital systems would lead to excessive computation and storage. The averaging resulting from Eq. D-1 is approximately equivalent to that obtained in Eq. D-2.

1. Sample volume at 1-min intervals; that is, count for 1 min, clear, and repeat.
2. Perform averaging by expression:

$$\overline{V}_i = (1-\beta)\overline{V}_{i-1} + \beta V_i \qquad \text{(D-1)}$$

in which

$i =$ the identification of the particular observation period (minute);
$V_i =$ the count during the period i;
$\overline{V}_i =$ the moving average at the end of period i;
$\overline{V}_{i-1} =$ the moving average at the start of period i (at the end of period $i-1$); and
$\beta =$ a fraction (equal to the reciprocal of the nominal number of minutes in the averaging period).

SELECTION OF CYCLE LENGTH

The traffic engineer specifies the cycle lengths to be used, together with the volume levels at which each cycle length is activated. For a change of cycle length from a given cycle length to the next longer cycle, there will be a specified critical volume level. For changing from the longer cycle to the next shorter cycle length, there will be a different (lower) critical volume level. This is necessary to prevent hunting of the system.

Let

$V_{ku} =$ volume level at which cycle length will change from level k to level $(k+1)$ when volume is increasing;
$V_{kd} =$ volume level at which cycle length will change from level k to level $(k-1)$ when volume is decreasing;
$\overline{V}_j =$ average volume at jth review time; and
$\overline{V}_{j-1} =$ average volume at previous review time.

Where an arterial is controlled, separate checks are made for each direction of traffic. Where a network is controlled, there may be multiple checks for cycle length selection. For example, consider an east-west arterial. Separate checks will be made for eastbound and westbound traffic.

1. For eastbound traffic:

Current preferred cycle from previous review $= C_{e_k}$ $(j-1)$. If $\overline{V}_{je} > V_{ku}$, change preferred cycle to $C_{e_{k+1}}$ (j) (i.e., level $k+1$). If $\overline{V}_{je} < V_{kd}$, change preferred cycle to $C_{e_{k-1}}$ (j) (i.e., level $k-1$).

2. For westbound traffic:

Current preferred cycle from previous review $= C_{w_k}$ $(j-1)$. If $\overline{V}_{je} > V_{ku}$, change preferred cycle to $C_{e_{k+1}}$ (j) (i.e., level $k+1$). If $\overline{V}_{jw} < V_{kd}$, change preferred cycle to $C_{e_{k-1}}$ (j) (i.e., level $k-1$).

System Cycle Selection $S(j) = \text{Max}\,[C_w(j), C_e(j)]$

If $S(j) - S(j-1) \geq 1$ (i.e., one level), increment S by 1 (level) with the sign of the difference.

OFFSET SELECTION

Provision is made herein for three offsets: "inbound," average, and "outbound." In the following statements, it is assumed that offset is expressed as a fraction of cycle length. For purposes of illustration, inbound will be taken as eastbound, and outbound will be taken as westbound.

Selection

Let

$k_e =$ level of cycle length preferred for eastbound traffic; and
$k_w =$ level of cycle length preferred for westbound traffic.
$(k = 1, 2, \ldots 6)$

If $k_e = k_w$, use average offset. If $(k_e - k_w) \geq 2$, use eastbound offset. If $(k_e - k_w) \leq -2$, use westbound offset.

Slewing

To minimize the transient conditions occasioned by changing the offset, which would disrupt progressive flow of traffic, the new offset is introduced gradually by a process that has been termed slewing because of its likeness to the slewing of a radar antenna. This slewing is accomplished at the local controller. The following paragraph develops a method for simulating this effect for experimental purposes.

Recall that if the offset is expressed as a fraction (either simple or improper), any integer value is the equivalent situation to zero offset. Consider the region in which

$$-1 \leq \frac{(N - \emptyset)}{\text{Cycle}} \leq +1$$

in which

$N =$ new offset; and
$\emptyset =$ old offset.

The objective is to slew in such a way that the difference of offset reaches a null in the shortest time (i.e., by the shortest route). The shift thus should be toward -1, 0, or $+1$, depending on which is the closest. It should further be remembered that at the end of the slewing, the difference of offset should be equivalent to zero and the new offset should be in effect. The shift is accomplished by incrementing the old offset. The sign of the increment should be:

Range of Difference of Offset	Sign of Increment to Old Offset
$-1 < \frac{(N-\emptyset)}{\text{Cycle}} \leq -½$	+
$-½ < \frac{(N-\emptyset)}{\text{Cycle}} < 0$	−
$0 < \frac{(N-\emptyset)}{\text{Cycle}} \leq ½$	+
$½ < \frac{(N-\emptyset)}{\text{Cycle}} < 1$	−

The magnitude of the increment should not be greater than 0.17 x (cycle length) during any signal cycle. This will cause the slewing to be completed in no more than three signal cycles.

APPENDIX E

REVIEW OF SPECIAL SIGNAL SYSTEMS RESEARCH AND APPLICATIONS PROJECTS

NOTES ON VISIT TO U.K. ROAD RESEARCH LABORATORY

On September 26 and 28, 1966, visits were made to the Road Research Laboratory (RRL) in England. Discussions were conducted at the Traffic and Safety Laboratory located at Langley. Persons contacted included: D. J. Lyons, Director; John A. Hillier, Acting Head, Traffic Control Section; John G. Wardrop; Dr. F. V. Webster; Dr. Joyce Almond; and P. D. Whiting.

Glasgow Project

Description

Glasgow, Scotland, has a population of approximately 1,036,000. There are 112 signalized intersections, of which 80 are involved in a large-scale traffic-signal experiment conducted by RRL. A central digital computer will be used to control the signals. Several strategic and tactical control techniques will be studied. Considerable effort will be devoted to "assessment," which will be done largely by floating-car methods.

Status of the Project

A building housing the computer, a small instrumentation laboratory, and a small staff office has been completed. Some equipment has been ordered. However, there is going to be a problem with fiscal matters. The fiscal year ends in March, and governmental regulations require that, in order not to lose an appropriation, equipment must be delivered and the bills actually paid before the end of the fiscal year. At the moment, there are £215,000 available for capital equipment, of which £120,000 are for a computer. There are an additional £60,000 for general expenditures. However, all of these must be completed with the bills paid by the end of March.

Loop Detectors

The principal inputs to the computer will be derived from loop detectors.

For individual-lane service, 12- by 6-ft loops appear to be best. Where wide coverage is necessary, the scheme shown in Figure E-1 is used to provide additional widths in 8-ft modules.

For analog outputs of long loops, an analog-to-digital converter (built by Marconi) is used. The analog is produced as a current. The converter handles 100 analog signals of 0 to 10 milliamps, providing a 7-bit output at a scanning speed of 1,000 channels per second.

Communication System

The Glasgow project will use its own direct wire as the primary means of communication. However, some leased (post office) telephone lines will be used in order to gain experience in this mode of communication.

The cables for direct-wire communication will be pulled through existing ducts originally used by the streetcar system. At each intersection there will be terminals to 20 voice-grade pairs. Fourteen of these pairs will be for the exclusive use of that intersection. One pair will be shared with five other intersections and will be used for voice communication. Five pairs will be shared with other intersections in a manner such that there are five common pairs to each two controllers. Thus, there will be 100 pairs for each six intersections. The cables will use polythene insulation around the individual conductors with a PVC outside cover. The outside cover will be surrounded by a steel armor to provide protection during installation.

Where leased lines are used, tone channels will be employed for multiplexing.

Experiments on Platoons

A joint experiment has been carried out by the Road Research Laboratory and the General Motors Research Laboratory. Specifically, Rothery of GM spent some time at RRL and conducted the following experiment. (A second phase of this cooperation will involve a representative from RRL spending some time at GM to work on another project.)

Field data were obtained on the platoon flow. The data were from four sites, where there were from 700 to 1,300 vph, nominally, in two lanes. The data were then used as input to a simulation to study the effect of signal offset on delay. It appears that for the sites examined, the delay is independent of flow but dependent on the characteristic speeds from one site to another. The results of this research work will be described in a paper by Rothery and Hillier, hopefully during 1967. The paper will appear first as a Road Research Laboratory Note, and may be published later by the Transportation Science Section of the Operations Research Society of America. The results were summarized in a paper by Almond and Lott, entitled "Glasgow: Implementation and Assessment," presented at the Symposium on Area Traffic Control sponsored by the Institution of Civil Engineers, February 20 and 21, 1967.

University of Birmingham Project

The University of Birmingham for several years has been interested in area traffic control, and has sought funding from the Road Research Laboratory, the Ministry of Transport, etc. They now have started a project in the northern part of London, including five signals. Their approach is simple but not much has been done. They are cooperating with the London police.

Simple Simulation

In a paper presented in Boston, Massachusetts (*13*), Hillier referred to a simple simulation for determining delay/difference-of-offset relationships. The details of this simulation were as follows.

An entry pattern was defined as consisting of certain straight-through, right, and left movements. These were then assumed to define the arrival pattern farther downstream. Two cycles of green for each signal were then observed. During the first scan, vehicles queued on red were discharged on green. On the second scan, delay was computed. The cycle was divided into 50 steps, and delays associated with each of 50 possible difference-of-offset values were determined.

NOTES ON VISIT TO U.K. MINISTRY OF TRANSPORT, TRAFFIC CONTROL DEVELOPMENT DIVISION

London Experiment Project

The Traffic Control Development Division of the Ministry of Transport is conducting a large-scale traffic-signal experiment in a section of west London. The area selected has a wide range of traffic conditions, including two important shopping areas, three major commuter routes, two major football grounds, and two major exhibition halls. A digital computer installation will be provided as the central control. Approximately 70 signals are involved.

On September 29, 1966, a visit was made to Mr. B. M. Cobbe, who is in charge of the experiment.

Status of the Project *

Detectors are presently being installed. Factory acceptance tests of the computer were expected during October 1966, and site tests of the computer are expected in January 1967. The project will move into its quarters in the new police building on Victoria Street about January 1, 1967.

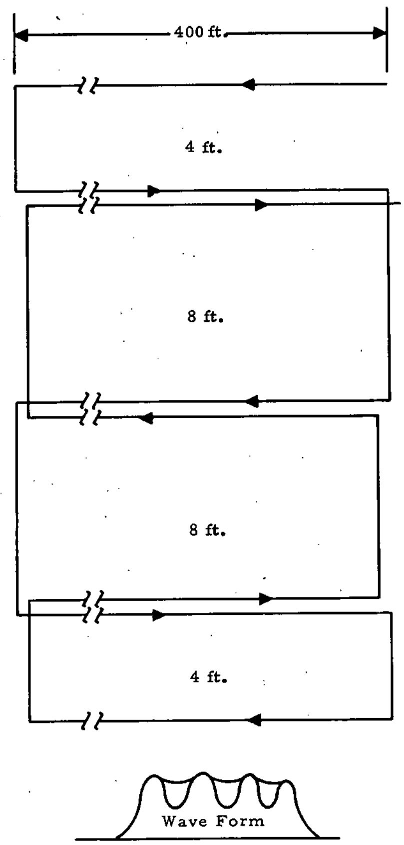

Figure E-1. Loop detector installation practice for Glasgow project.

Data transmission will begin with a few intersections on January 1, and with all intersections by the end of February. Closed-circuit television will be operating at some locations in January, with limited system operation by February 1, 1967.

Detectors

Several detection objectives are provided within the London Experiment. On the average there will be six detectors per intersection. Some pneumatic detector locations include two tubes about 6 in. apart to give measure of speed. In the present installation, speeds from the pneumatic detectors are not being transmitted to the computer, but are being used by local controllers.

On every approach to each signalized intersection there will be a pneumatic detector about 130 ft upstream from the stop line to sense traffic carried (not traffic demand). These pneumatic detectors are of the type used in many European cities in which a rubber tube of rectangular cross section is enclosed in a housing below the roadway surface, with only a small portion of the rubber tubing protruding. This tubing is then connected to a pneumatic diaphragm contactor located in a cabinet at the side of the road.

Loop detectors are being placed at locations that are representative of free-running traffic to give a sample of

* Tentative scheduling as of September 1966. As of July 1967, implementation was running behind schedule.

traffic offered, which is used to produce an index representing current traffic-demand conditions.

Detectors not yet developed will be used to sample platoon speed. With present techniques, it is possible to get platoon speeds within ± 10 percent. Cobbe would like to achieve accuracies of ± 5 percent.

To get a quick look at speed, two loops are used in one lane (usually the center lane of three, but each site is given separate consideration). The first loop triggers a timing gate and the second loop closes the gate. There is a 5-sec hold in the logic to prevent ambiguities in recognizing the two pulses from one car, as opposed to pulses from two successive cars (see Fig. E-2).

For these purposes, a platoon is defined as a certain number of vehicles passing along the road within a certain time. Two loops are used in a particular lane. The output goes to an analog circuit which will produce a moving average of the rate of arrival. If the rate of arrival is low, there will be no computation of speed. If the pulse rate from the detector is greater than some preset figure the equipment will start to measure speed. They may use eight measurements and obtain a true average.

An important part of the system is the use of queue detectors. There will be approximately two queue detectors per signalized intersection. The first form of queue detection will be a point detector to measure whether a queue has reached a certain point. A queued vehicle will be defined as one which is stationary or one which is traveling at less than 8 mph.

A short-queue detector will be a queue detector at a point a little farther upstream than the point of natural fluctuation in storage at the intersection. Short-queue detectors may be approximately 400 ft from the stop line.

A long-queue detector will be placed approximately 1,000 ft from the intersection.

Exit-queue detectors will be placed at 130 ft beyond the curb line on the exit side of the intersection to provide information to the computer as to when the output of any intersection is blocked.

Local Controllers

The local controllers are essentially vehicle-actuated controllers of the type generally used in Great Britain, but with special features to permit them to operate in the digital computer system. The computer emits two kinds of pulses, designated as a p and a q pulse. When the local controller receives a p pulse, this will cause a forced change of phase, provided there has been local demand

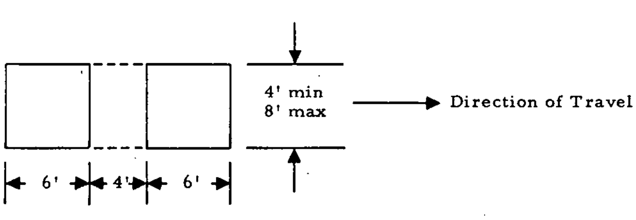

Figure E-2.

for a change. A q pulse can provide an artificial demand to the local controller, and a continuous chain of p and q pulses gives the computer complete control.

Communication System

The communication system will make use of post-office lines (i.e., leased voice pairs). Each pair will carry 12 channels of output, 24 channels of input, and 5 channels of analog-derived information. These channels will be handled by time-division multiplexing. The message will be in a 50-band format. There will be one pair per intersection carrying data (counts, speeds, queue information), control pulses for the vehicular crossing, and up to three midblock pedestrian crossings. Outstations will be locked to mains (power line) frequency and synchronized from the computer at 50 cps. Up to eight outputs will be required for four-phase intersections.

Control Center

The control center (at the police building) will contain two computers, one of which will serve as a data scanner to format and distribute the data as needed. The input to the computer will be 72 bits, and the output from the computer will be 24 bits.

It will be possible for the operator to alter constants in the program during operation. The operator can also use a manual control console.

Computer Programming

A contract has been let to the Plessey Automation Group to provide computer programmers in the amount of 15 man-yr (five men for 3 yr). At present, there are three priorities for the programming effort:

1. Priority 1: Development of detector checking programs.
2. Priority 2: A series of 10 fixed-time programs.
3. Priority 3: Use of exit-queue detectors to split greens at a particular intersection.

To check detectors, it will be possible to put in historical program tapes of time-of-day and day-of-week for up to 12 weeks. Standard deviations, etc., will then be computed, and an index for all traffic in the area will be used to develop a relative value to be expected from a particular detector. This relative value then is compared with the historical data, and when the detector is found to be outside expected limits, an alarm message will be given.

The detector check program will be followed by a pattern recognition routine used to select the appropriate strategic program from a library of 10 strategic programs. This pattern recognition program will look at approximately 10 signal cycles, adjusting traffic levels by means of a prediction curve. It will then select cycle times by subareas. Pattern recognition will then give a selection between inbound, outbound, etc. Of these 10 fixed-time (strategic) programs, 1 or more can be used at any time. These are written in assembly language. The strategic timing plan will be modified by tactical measures based on detection of queues, speeds, etc. Tactical changes will be a common-sense approach to local situations. For instance, if both phases have queues, the tactic may be to take that intersection out of system control or to change the system timing.

The tactical program will create the biggest load on the computer. Each intersection will be scanned once per second. Initially, there will be 70 intersections in the system, but there will be capacity for 150 intersections.

Counting of Traffic

Counters will be scanned in discrete time units with time-division multiplexing. There will be two channels from each intersection approach to the computer. Nowhere is it expected there will be more than seven axle impulses per second, and thus binary counters 0 to 7 will be used. These will be divided by two by dropping the least significant bit.

Closed-Circuit Television

Closed-circuit television is being installed on a trial basis at certain critical intersections (eight cameras). Transmission will be by means of special balanced pairs consisting of four wires. This is a replacement for coaxial cables. Transmission is not in the slowed-down-video mode.

Delay Assessments

The basic technique of delay assessment will be by means of moving cars, but there is some question as to the accuracy of this method. Cobbe would like to obtain a 5-percent accuracy. John Hillier (of RRL) believes that this may be possible. It is expected to have a computer program that will estimate delay, and the Greater London Council staff is looking at methods for such computer estimation of delay in the Greater London Council area.

Capital Costs

The capital costs quoted include all new purchases. (Local controllers are all in service now and are not included.) The total capital cost is expected to be £550,000, plus an annual expenditure of £37,000 for 10 yr. Using a cash discount method of computation at 8 percent interest, the present value is computed as £794,529.

Benefits

In computing benefits it has been assumed that it will be possible to detect a benefit of 5 percent in reduced delay. In 1961 the Road Research Laboratory found that a 10-percent reduction in delay would justify capital costs of £1,000 per stop line.

The value of delay to the public traveling through this area is £10,000 per year, including working time and vehicle costs, but excluding nonworking time. In these computations, traffic growth has been assumed at the rate of 3 percent per year, and delay growth has been assumed at three times the growth of traffic. Under these assumptions, benefits have been estimated as £259,000 for 1967 and £358,000 for 1976. The present value of the total savings through 1967 has been estimated at £1,951,115. This gives a benefit-cost ratio of 2.461. It has been found that a reduction of 2.03 percent in delay would be adequate to cover the costs. A 5-percent reduction in delay would lead to a return of 19 percent on the investment.

Project Management

Although this is strictly a Ministry of Transport project, there is an advisory committee that includes representatives of the RRL, the police, Greater London Council, the Ministry of Transport, and the London Transport Executive (i.e., transit).

Cobbe is chairman of the working party, which includes representatives of the Ministry of Transport, the RRL, the Vehicle Actuated Traffic Signal Manufacturers Association, and the Greater London Council.

Operation of the system will be jointly manned by police and by engineers. The relationship with the police is not yet formally established.

NOTES ON VISIT TO METROPOLITAN TORONTO, DEPARTMENT OF TRAFFIC

A visit was made to Toronto, Canada, on August 8, 1966. Discussions were held with S. Cass and J. T. Hewton.

The Toronto installation consists of two computers—a UNIVAC 1107 performs the main computation and control functions, and a UNIVAC 418 prepares the input format. There are 6 tape units in operation, and 45 million characters stored on disc or drum.

As of early August 1966, the Toronto system included 415 signalized intersections and 400 detectors; there was no description of the program.

Detectors

Detectors are of the loop type, operating at approximately 100 khz. Each detector is scanned 32 times per second, and each intersection is completely scanned at least once per second. In transmitting detector information to the computer, tone-coding is used, employing 10 frequencies between 340 and 2,600 cps. The signal is an on-off type, where the normal condition is on. It is not certain where detectors should be located; however, Hewton favors individual lane detectors.

In the program, each detector input is assigned to a specific intersection and cannot be changed. With multilane detection, the computer divides by 2 and assumes two lanes are flowing (which may not be the case). The equipment is limited as to the number of detectors it can handle.

With regard to loops, the problem is to recognize gaps between vehicles.

Program

As of August 1966, 85 percent of the program cycle time and 95 percent of the memory space were used up.

Normal operation is in the fixed-time mode, but there are two responsive modes of operation: one which operates individual intersections on a volume-density basis, and another which is a predetermined cycle with variable split.

Most of the operations are performed by table lookup.

Much staff time is spent in manual computations and table preparation. Hewton favors the use of some simple formula for calculation of timing. He feels that Webster's formula is all right if saturation flow does not change with time of day, weather, etc.

A fixed-time program is used until it breaks down, and is then switched to a volume-density mode of operation. If volume alone is used, it is possible to get a false response.

Over the past year, program breakdown has occurred with the addition of more intersections. Every intersection is handled on an individual basis—none are grouped. The subroutine for timing of the yellow is entered every cycle.

Effectiveness

Before the computer was installed, 100 intersections were under police control. Now there are 12 special odd movements. Routes are generally working better.

Hewton feels that congestion cannot be eliminated, but can be reduced.

One question Hewton has is, "What is the volume-density relation on surface streets?" He feels that it is necessary to know more about the volume-density relationship and speed-offset relationships. Delay computations made by the computer are not calibrated. Occupancy may be a good measure of effectiveness. Hewton feels that it is necessary for the computer to be programmed to look at trouble spots.

NOTES ON SAN JOSE PROJECT

These notes are based on visits prior to the start of NCHRP Project 3-5/1, telephone conversations, and published information.

History and Status

The project in San Jose, California, is an experiment in the use of a digital computer to control a system of traffic signals. The project officially started in July 1964 as a cooperative effort between the city and IBM. Many delays were encountered in getting the equipment installed and operating; principal delays were caused by loop detectors.

As of June 1966, 397 detectors were installed and operating. They were placed 300 to 700 ft back (upstream) from the stop line. Detectors are sampled every 5 sec. In June 1966, there were 60 intersections under control.

Vehicle Counts

Count data from detectors are analyzed every 5 min, and once a day the traffic engineer receives a printout of 5-min counts for each detector in use. Using arbitrary limits for high and low counts, the computer has been programmed to type out an alarm when detectors are providing either high or low counts. The criterion for a high count has been set at 100. Low counts usually indicate some failure within the detector, while high counts indicate a need to tune the inductive circuit. The detector information is used to estimate the number of vehicles stopped. Total delay is used as a measure of effectiveness.

Analysis and Programming

Dr. Albert Chang of IBM has provided a large part of the analysis for this project. He believes that in San Jose the time of day is more important than traffic measurements in the selection of the control procedure. Current control procedure consists of table lookup, with the tables being selected on the basis of time of day or on the basis of measurements of current traffic. The tables were established by means of a simple simulation based on some theoretical assumptions stated in mathematical terms.

Dr. Chang believes that control may be similar to the algorithm proposed by Potts. He believes that it may be equivalent to an instantaneous change in split. If the split is changed instantaneously, Dr. Chang believes that it should be based only on current data—not on historical data. They are not using the work of J. D. C. Little. Speed is estimated by means of a measurement of occupancy. In one location, however, they have two loops placed 10 ft apart by which to measure speed directly.

The 5-sec scan is a machine limitation. Off-line, there is an analysis to evaluate the operation. This analysis consists of comparing the number of stopped vehicles as observed in the field and the stops provided by the computer. This comparison uses the product moment statistic.

San Jose has an on-line progression program developed by IBM mathematicians and programmers. It utilizes a routine that gives a maximum bandwidth for progressive arterial flow. The way it is used in real time is to recalculate the results and install a new progression every 15 min. This program is also "tuned" at the central location by observing the number of cars stopping at key intersections, and manually inserting parameter changes to minimize this number.

At critical intersections the timing is controlled by one or more "micro loops." A typical micro loop is described as follows (*61*):

> This version holds the offset on the major street and adjusts the split only. It maintains the minimum "Walk" and "Don't Walk" timings.
>
> Calling a certain level of demand on the "A" phase "X," and a level of delay on the cross street "Y," this algorithm works as follows:
>
> If the demand on "A" phase is more than "X" and the delay on the cross street is less than "Y," "A" phase is expanded to the maximum.
>
> If the demand on "A" phase is less than "X" and the delay on cross street is more than "Y," then "A" phase is cut down proportionally to serve the cross street sooner.
>
> If there is no cross street requirement or if the demand on "A" exceeds another fixed value, then the cross street phase will be cut sooner—to get to "A" phase before its normal offset.
>
> If both "X" and "Y" are exceeded, the background cycle timing will stay in effect.
>
> If both "X" and "Y" are below set values, the background cycle timings stay in effect.

By applying micro loops to key intersections, it was found that the benefits of a good background cycle can be enhanced further (*61*):

> Probably one of the most significant results of this system operation so far has been the demonstrated ability to change decision making techniques without physical changes and with a minimum of programming disturbance to the great bulk of the programming system. This, in a time of steady technical growth, is extremely important.
>
> One of the most significant constraints upon the system is that disturbances must be corrected smoothly and accurately. This is accomplished by computer analysis of the absolute time deviation from normal of each intersection for each cycle, and the application of a correction factor for the next cycle. A side benefit of this automatic, constant smoothing and correcting techniques is that the gentle transition from one set of timing tables to another is accomplished by merely replacing old tables with new ones.

APPENDIX F

ABSTRACTS OF SELECTED PAPERS *

ALMOND, J., and LOTT, R. S., "The Glasgow Experiment in Area Traffic Control: Implementation and Assessment." (*1*)

Presents a narrative of the Glasgow Experiment, giving special attention to the actual computer (hardware) operation and to the proposed measurements of operational effectiveness. An earlier paper by Hillier (*13*) describes the control principles to be tried.

The computer to be used has a 16K-24-bit word-core capacity. An additional 50K drum is provided. Priorities are assigned for the interruption by various peripheral components. Obviously, the control inputs and outputs to the intersections are the highest priority. A monitor program allocates machine time for the various functions according to these priorities.

Detectors are scanned 40 times per second. Similarly, each intersection signal is considered for a change of phase once per second. The phase change routine has been especially designed to be compatible with the several control principles to be tested. A set of standard error routines has been provided to check system components for malfunction.

Considerable on-line data collection is provided for. This consists primarily of data from the various vehicle detectors. A unique feature is the proposed radio telemetry link to a vehicle making "floating-car" travel time measurements in the system. The various control strategies require differing amounts of on-line data.

In the evaluation of operational effectiveness, travel time has been chosen as the primary criterion. Number of stops was considered to be second in importance. Several methods of combining observed trip times and traffic volumes were considered. The sum of the travel time on each of the links weighted by the traffic volume on that link appears to be the most satisfactory method of obtaining total travel time.

Much effort is being expended to see that test and control conditions are as nearly alike as possible. Tests will be run at the same time of day and on the same day of the week as far as practicable. Results may be discarded if unusual variations in traffic flow entering the area are observed.

BROOKS, W. D., *Designing Arterial Progressions Using a Digital Computer*. (*2*)

A two-part article. The first section describes a mathematical technique for the construction of a time-space diagram. The second part, "Traffic Progression Program" by Robert N. Chamberlain, describes the computer program to accomplish the method described in the first part.

The technique described constructs a bidirectional time-space diagram using the intersection having the minimum bandwidth (minimum artery green) as the origin. The necessary input data include signal spacing, assumed percent split at each intersection, and the ranges of speeds and cycle lengths desired. The initial solution provides for equal speeds and bandwidths in both directions.

Utilizing the minimum bandwidth of the origin intersection, offsets at succeeding intersections are chosen to minimize the interference with this bandwidth at the given speed (slope). The interferences are then tabulated in two columns, one for each side of the band, intersection by intersection. Data are arranged in descending rank order for one of the columns, with the corresponding figures for the opposite column being kept together with those of the proper intersection (i.e., if a street is the third entry in the rank-ordered column, it is also the third entry in the opposite column). These sets of interferences are examined, in order, in an attempt to find the minimum total reduction of bandwidth.

Once the bandwidth is determined, offsets are calculated such that the middle of the through (green) band is offset either by 0 percent or 50 percent from the origin intersection.

* Numbers indicate references in Appendix G.

The initial solution is then generalized to permit different speed and bandwidths in the two directions as well as a range of cycle lengths. This may give offsets that are different from the half-cycle multiples normally expected with uniform speeds and bandwidths in both directions.

The second part of the article describes the computer program to carry out the computations previously described. This program is based on uniform cycle lengths and uniform speeds in both directions on the street being examined. The required input includes the following items: (1) Intersection spacing; (2) green time at each intersection; (3) range of cycle lengths; (4) range of speeds; (5) increments in speed to be considered; and (6) traffic volumes in both directions.

It is possible to use a single value for cycle length and speed. The program is limited to 24 intersections. The output includes the input data, minimum interferences, optimum speed, cycle length, and bandwidth, as well as the individual offsets for each intersection. For unequal (directional) flows the apportioned offsets and directional flows are also given.

CHANG, A., *Synchronization of Traffic Signals in Grid Networks.* (*4*)

Presents a method of timing traffic signals in a grid network based on the minimization of total delay to traffic in the system. Discrete vehicles are not considered; rather, traffic is treated as a continuous flow. The main variables considered are vehicle flow rates and queues. Both of these variables are defined at the intersections but not on the street in-between.

Several simplifying assumptions are made. These include uniform speeds on each link, constant queue discharge rates with no lost time, uniform periodic platoons with uniform headways arriving at the system boundaries, and flow rates less than saturation (i.e., queues cannot extend from one intersection into another). Expressions are developed for (1) flow rates, at the approach to an intersection, (2) queue length and (3) discharge rate from an intersection. It is shown that (2) and (3) are periodic. The periodic solution is assumed to be the steady-state solution.

The total steady-state delay is expressed as a function of the cycle length and (2) and (3), as mentioned. The delay function is discontinuous. It is approximated by a piecewise quadratic curve. Procedures for a two-stage search for local minima of delay are presented. Two examples—one for an arterial, the other for a network—are shown.

DESROSIERS, R. D., and LEIGHTY, C. H., "Traffic Flow Responses to Unannounced Increases in Progression Speeds of Signal Systems." *Pub. Roads.* (*57*)

Describes an experiment carried out on a portion of 13th Street, N.W., Washington, D.C., to determine the time required for drivers to respond to changes in progression speeds along signalized traffic arteries. The existing progression speed was raised from 27 to 33 mph. Two months later it was raised again to 40 mph. It was concluded that a considerable length of time is required for drivers to adapt to progression speed changes when they have no knowledge of such changes. Further research is recommended, with consideration being given to providing information of the progression speed change to the motorist by the use of signs or other media.

DICK, A. C., "A Method for Calculating Vehicular Delay at Linked Traffic Signals." *Traffic Eng. and Control.* (*9*)

Presents a method for determining vehicular delay on an arterial from a time-space diagram by dividing the flow of traffic into saturated and free-flowing portions. A discrete boundary between the saturated and free-flowing portions is assumed, as is a uniform free-flow speed and uniform vehicle-arrival rate.

Arriving and leaving traffic is classified into either saturated or free-flowing bands or combinations thereof related to the timing of the traffic signal. Differences in traffic volumes between adjacent intersections are accounted for by a correction factor. It is shown that the travel time within a set of bands may be represented by the total moment of the bands about some point divided by the total traffic content of the bands. It is further shown that the delay in a system of signals may be represented by the quantity, the difference between the total moment of the leaving bands and the total moment of the arriving bands (about the same point) divided by the traffic content of the system; minus the travel time for the system. A brief example with graphics is shown. The process of optimization by other than trial and error is still under investigation.

DUNNE, M. C., and POTTS, R. B., "Algorithm for Traffic Control." *Oper. Res.* (*10*)

States that because of rapid technical development, Webster's work is no longer an adequate guide to the traffic engineer who wishes to make the best use of modern equipment available. Describes a signal-control technique wherein the switching of lights never occurs if phase duration is less than a preset minimum, or always occurs if the phase is greater than or equal to a preset maximum. When the duration of time on a phase is greater than the minimum and less than the maximum, lights are switched if (1) the queue being favored is emptied, or (2) there are vehicles in the queue and the applicable control function is negative or zero.

In this context, the control function is the value, computed as follows:

$n_i(t)$ = number of vehicles queued in arm i, $(i = 1, 2)$

$$\left.\begin{array}{l} f_1(t) = \alpha_1 n_1(t) + \beta_1 - n_2(t), \\ f_2(t) = \alpha_2 n_2(t) + \beta_2 - n_1(t), \end{array}\right\} \text{Control functions}$$

in which

α_1 and β_1 are control parameters selected by the engineer, subject to the conditions $\alpha_1 > 1$ and $\beta_1 > 0$.

Figure F-1 illustrates the changes in queue state as a random walk which, when it reaches boundaries defined by the control functions, causes switching of the lights.

This is a tactical algorithm and is suitable for use on a digital computer. It is primarily oriented at individual intersections not having a strategic background cycle.

Figure F-1, n_2 equals number of vehicles queued in approach 2 and n_1 equals number of vehicles queued in approach 1 for a typical case when approach 1 has the green light. The representative point performs a walk on the lattice and $0A_1B_1$ (and similarly $0A_2B_2$) form reflecting barriers at which the light is switched in accord with the control algorithm. The three steps SS' are the possible last steps before reflection.

GAZIS, D. C., and POTTS, R. B., "Route Control at Critical Intersections." *(74)*

Presents a method of relieving traffic congestion at critical intersections by means of route control. By splitting the flows of traffic into two streams for each approach, it is shown that theoretically 100 percent of green time may be achieved for all approaches. This requires that the split streams for each approach be channeled through two intersections properly spaced to utilize the gaps in the cross traffic. Precise platooning and transition roadways of a specified length are required to carry out this plan. In the case of 100-percent flow in all directions, a relatively complex system of eight precisely spaced intersections is needed to replace the original single intersection. Discusses simplifications to the system as well as the practical problems that would arise if the plan were put to use.

JOHNSON, J., *Optimum Control of an Unsaturated Artery. (15)*

Presents techniques whereby aggregate trip time for vehicles traveling along an artery during some period of time is minimized. Describes time-space diagrams with equal speed and equal bandwidths in both directions. It is found that the best offsets are multiples of the half-cycle time. The maximum width bands are usually defined by three points at which the red period is touched. Transformations are then developed to permit unequal speeds and bandwidths.

Develops procedure for generating bands located by these points within given limits of speed and cycle length. The basic assumption that green intervals and bandwidth are proportional to cycle length is then modified to allow for fixed minimum cross-street green times, all red periods, and time lost as a result of acceleration and deceleration. A term is added to account for the loss in bandwidth that occurs at the beginning of green and yellow periods resulting from decisions made by the leading and trailing drivers. Notes the possible occurrence of more than one band per cycle in a given direction.

In the optimization process, an expression for the negative of aggregate trip time is maximized within the constraints of the acceptable range of speeds and cycle lengths as well as the maximum difference in speed between the two directions of travel. Similarly, design flows cannot exceed the maximum progressive flow. Bandwidth is shifted between opposing directions of flow, when possible, to handle unbalanced flows. The optimization problem is formulated as a nonlinear program with nonlinear constraints.

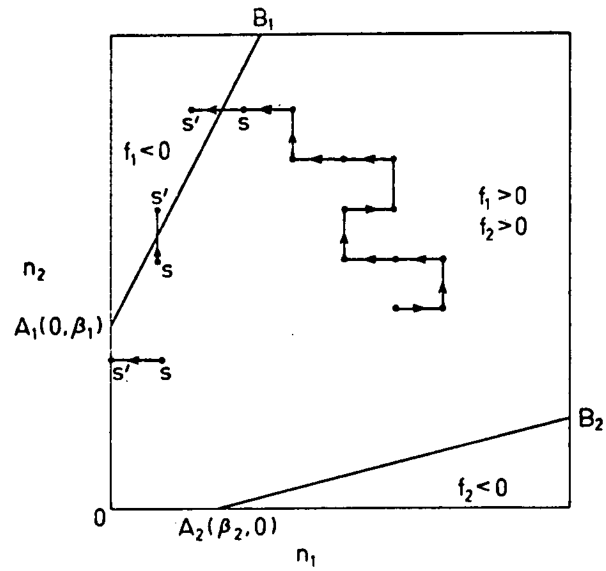

Figure F-1.

Details of the computer program are not given. It appears equally applicable to real-time control operation or to off-line computation. Extensions and improvements are suggested.

LITTLE, J. D. C., "The Synchronization of Traffic Signals by Mixed-Integer Linear Programming." *Oper. Res. (18)*

Gives a method for synchronizing traffic signals for both arteries and networks. The formulation consists of a mixed-integer linear program (LP), i.e., a standard linear program where some of the variables are constrained to be integers.

In a preceding paper, the author developed a computational algorithm (not a linear program) that solves the artery problem where the travel times between signals must be prespecified. Two solutions are possible: the first results in equal bandwidths in each direction; the second allows one bandwidth to increase to a specified feasible value that maintains the other bandwidth as large as possible.

In the present linear programming formulation, additional variables and restraints can be introduced. The driving speed between each pair of signals can be made variable and can be restrained between some given lower and upper limits. Also, the change in speed between adjacent signal pairs can be restrained if desired. The common signal period can be a variable, with an upper and lower bound specified. Also, the red-green split (in percentage) can be made a variable.

Three linear programs (LP) are displayed as representative of the many specific programs that can be specified.

In the first LP, there are two types of restraints. The bandwidths cannot exceed the green period, and the bandwidths on two adjacent signalized intersections must be within the required speed limits. The number of LP restraints for this formulation is about $3n$, in which n is the number of signalized intersections.

In the second LP, the common signal period (cycle length) and speed are allowed to be variables. This larger LP contains about $11n$ restraints. If the speed constraints are the same in each direction, and the bandwidths are the same in each direction, then the third LP results, with approximately $6n$ restraints.

For the network case, additional restraints must be added. Sufficient restraints must be introduced so that every one of the loops will be included. No general rule is given on the number of loop restraints that are required. The number will depend on the number of network modes, areas, and average connectivity. Otherwise, the resultant LP is similar to the artery formulation.

The branch-and-bound algorithm is used as a general solution procedure. In this approach, no integer variables are specified in the LP itself; rather, each such variable is either left unrestricted or set to a constant. The particular constant values are chosen based on the answers obtained from some previously set constant value. Each change in the constant values requires that another standard linear program be solved. This standard LP can be solved using any LP routine available on any digital computer.

A 10-signal artery example is fully worked out. The LP contains 55 restraints and requires 49 separate LP solutions. No timing estimates are given in the paper, but each LP problem on an IBM 7090 should require only about 0.5 min. Thus, all the solutions would use about 25 min of computer time. The cost can be lowered in several ways. First, each LP need not be solved independently; rather, the solution to a previous LP can be used as a starting point for the next. Second, the optimal solution need not be required. In the 10-signal example, an answer only a few percent lower is found solving only 19 LP problems.

A seven-signal network problem is also completely worked out. The solution is obtained by solving 15 LP problems.

In summary, the LP approach appears to offer a comprehensive method for solving a variety of synchronization problems. The basic solution procedure depends on standard LP routines that are available on almost every digital computer. The branch-and-bound algorithm is easily programmed if desired.

MARCONI, W., "Multiphase Signals and Traffic Congestion." (*20*)

Relates experience in the San Francisco area with the reduction of traffic congestion by the elimination of multiphase traffic signals. In particular, the traditional approach to high-volume intersections involving complicated signal phasing is shown to be a contributor to excessive vehicle delay.

From a history of delay studies, a family of curves relating average delay per vehicle to critical approach lane volume is developed. As the number of signal phases is increased, the average delay per vehicle increases significantly. An envelope of curves for two-, three-, and four-phase signals shows this clearly. A similar family of curves is developed for various percentages of comparative main-street/side-street traffic volumes. Again, it is shown that there is significantly less delay with the two-phase signal operation. A series of case histories showing intersection geometrics and signal phasing is presented.

In conclusion, several novel systems of intersection control are shown.

RAUS, J., "A Method of Computing Offset Patterns for Multi-Cycle Signal Systems." *Traffic Eng.* (*75*)

The title is explicit. Inasmuch as the author was employed by Automatic Signal Division at the time the method was developed, it appears intended for use with the traffic-adjusted "PR" system.

First, a bidirectional "ideal" time-space diagram is constructed for a given travel speed and various cycle lengths for the roadway segment under consideration. A "mode of fit" is calculated for each intersection and cycle length. This is actually a measure of the difference between the "ideal" offsets for the two directions of travel and the average offset that would be used at the intersection.

It is then shown that the method can be simplified to eliminate drawing of the time-space diagram. The result is that the average offsets are either 0 percent or 50 percent of the cycle length (i.e., simultaneous or alternate timings).

For each cycle length, the critical values of the "modes of fit" (poorest fit) are located along with the locations of the critical split (shortest artery green). These locations are examined for the minimum through (green) band for each cycle length. It is then possible to compare "cycle efficiencies" for the various cycle lengths.

Traffic Research Corporation, *SIGOP: Traffic Signal Optimization Program.* (*71*)

Prepared under contract with the Bureau of Public Roads, this report describes a computer program for finding optimum traffic-signal timing for street networks. The SIGOP system consists of six program blocks written in FORTRAN IV language. Communication from one block to the next is via files written in machine format. Normally, the output tape from one block is input for the succeeding block.

Briefly, the functions of the various program blocks are as follows:

1. INPUTS Program—reads and checks card input data, and computes critical flow and total flow at each intersection. This is done for each Optimization Time Period under consideration.
2. PHASES Program—computes phase splits for each intersection in proportion to a combination of total flow and critical flow as specified by the traffic engineer. This is done for each cycle length required.
3. OFFSET Program—computes ideal offset differences and weights for each network link.
4. OPTIMIZ Program—computes optimal offsets for each intersection given the ideal offset differences and weights.

5. VALUAT Program—evaluates each set of optimal offsets, or the original offsets, by calculating delay, number of stops, and cost.

6. OUTPUT Program—prints new traffic controller dial settings in terms of splits and offsets. Also prints time-space charts.

For the purpose of analysis, intermediate printed output is available from the INPUTS, PHASES, OFFSET, and OPTIMIZ program blocks.

Input data for the intersections may be specified in either of two ways. Data for systemwide parameters may be specified on the MACRO cards, while data for a specific intersection may be specified on the MICRO cards. If a parameter has a uniform value over most of the system (i.e., free-flow speed), it may be specified in the MACRO card and omitted from the MICRO card. An entry on the MICRO card would be made only where a value different from that on the MACRO card is needed.

Similarly, certain of the parameters may be calculated by the program or may be inserted directly by the traffic engineer. For example, a fixed split may be specified for one intersection, while the remainder of intersection splits is calculated by the program.

At several places in the program, the traffic engineer may choose which variable or combination of variables is used in a particular calculation. For instance, in the split computation, either the total flow (both directions), the critical lane flow (one approach), or a weighted combination of both may be used.

Similarly, there are several "importance" or "weighting" factors that may be applied to each link for various calculations. This allows a great amount of flexibility in the program operation. However, it appears that considerable experience with the program would be necessary before the traffic engineer would be able to choose appropriate values for these factors with ease.

Certain basic assumptions are used in the SIGOP system. Generally, no probabilistic features are incorporated in the program. Traffic is assumed to arrive at a uniform, nonrandom rate in discrete platoons. Uniform headways are assumed although a "platoon coherence factor" (measure of dispersion) is used. Turning movements and large vehicles are converted to equivalent through traffic for each intersection. No provision is made for vehicle-pedestrian conflict or for turning movement conflicts.

SIGOP appears applicable only to stable flow conditions below the level of congestion. Traffic behavior is assumed to be periodic, repeating each signal cycle. No cycle-to-cycle accumulation of queued vehicles is permitted. An average time headway for vehicles discharged from signals is used at all intersections in the network under study.

At the time of this project, SIGOP was undergoing a refinement process and was not yet available for practical applications work.

APPENDIX G

BIBLIOGRAPHY

STRATEGIC AND TACTICAL TECHNIQUES

*1. ALMOND, J., and LOTT, R. S., "The Glasgow Experiment in Area Traffic Control: Implementation and Assessment." Paper prepared for Symposium on Area Control of Traffic, sponsored by Inst. of Civil Engineers, London (Feb. 1967).

*2. BROOKS, W. D., *Designing Arterial Progressions Using a Digital Computer.* IBM Corp. (n.d.).

3. BUCKLEY, D. J., HACKETT, L. G., KEUNEMAN, D. J., and BERANEK, L., "Optimum Timing for Co-ordinated Traffic Signals." Paper presented at Third Conf., Australian Road Research Board (Sept. 1966).

*4. CHANG, A., *Synchronization of Traffic Signals in Grid Networks.* IBM Corp., San Jose Res. Lab. (n.d.).

5. COBBE, B. M., "Development of Traffic Signals for Area Traffic Control." *Traffic Eng. and Control,* Vol. 5, No. 9, pp. 543-546 (1964).

6. COBBE, B. M., "Questions and Answers on West London Traffic Scheme." *Traffic Eng. and Control,* Vol. 7, No. 9, pp. 562-565 (1966).

7. COBBE, B. M., "Area Control of Traffic, West London Experiment." Paper presented at Eighth Internat. Study Week in Traffic Eng., Barcelona (Sept. 1966).

8. DAVIDSON, B. M., "Design of Signal Systems by Graphical Solutions." *Traffic Eng.,* Vol. 31, No. 2, pp. 32–35, 38 (1960).

*9. DICK, A. C., "A Method for Calculating Vehicular Delay at Linked Traffic Signals." *Traffic Eng. and Control,* Vol. 7, No. 3, pp. 224-229 (1965).

*10. DUNNE, M. C., and POTTS, R. B., "Algorithm for Traffic Control." *Oper. Res.,* Vol. 12, No. 6, pp. 870-881 (1964).

11. DUNNE, M. C., and POTTS, R. B., "Analysis of a Computer Control of an Isolated Intersection." *Vehicular Traffic Science, Proc. Third Internat. Symposium on Theory of Traffic Flow,* pp. 258-266 (1967).

12. GAZIS, D. C., "Optimum Control of a System of Oversaturated Intersections." *Oper. Res.,* Vol. 12, No. 6, pp. 815-831 (1964).

13. HILLIER, J. A., "Glasgow's Experiment in Area Traffic Control." *Traffic Eng. and Control,* Vol. 7, No. 8, pp. 502-509 (1965). Appendix, Vol. 7, No. 9, pp. 569-571 (1966).

* Papers abstracted in Appendix F.

14. HILLIER, J. A., and LOTT, R. S. Paper presented at Eighth Internat. Study Week in Traffic Eng., Barcelona (Sept. 1966).
*15. JOHNSON, J., *Optimum Control of an Unsaturated Artery.* Data Processing Div., IBM Corp. (Aug. 1964).
16. LA VALLEE, R. S., "Scheduling of Traffic Signals by Linear Programming." *Proc. HRB,* Vol. 35 (1956) pp. 534-542.
17. LAPIERRE, R., and STEIERWALD, G., "Possibilities for Traffic-Guided Signal Control of Road Networks." Paper presented at Eighth Internat. Study Week in Traffic Eng., Barcelona (Sept. 1966).
*18. LITTLE, J. D. C., "The Synchronization of Traffic Signals by Mixed-Integer Linear Programming." *Oper. Res.,* Vol. 14, No. 4, pp. 568-594 (1966).
19. LITTLE, J. D. C., MARTIN, B. V., and MORGAN, J. T., "Synchronizing Signals for Maximal Bandwidth." *Hwy Res. Record No. 118* (1966) pp. 21-45.
*20. MARCONI, W., "Multiphase Signals and Traffic Congestion." *Proc. 19th Ann. Meeting,* Western Section, Inst. of Traffic Engineers (1966).
21. MILLER, A. J., "Settings for Fixed Cycle Traffic Signals." *Operational Res. Quarterly,* Vol. 14, No. 4, pp. 373-386 (1963).
22. MILLER, A. J., "A Computer Control System for Traffic Networks." *Proc. Second Internat. Symposium on Theory of Traffic Flow,* London, 1963. O.E.C.D., Paris, pp. 200-220 (1965).
23. MORGAN, J. T., and LITTLE, J. D. C., "Synchronizing Traffic Signals for Maximal Bandwidth." *Oper. Res.,* Vol. 12, No. 6, pp. 896-912 (1964).
24. MORRIS, R. W. J., and PAK-POY, P. G., "Intersection Control by Actuated Signals." Presented at Third Conf., Australian Road Research Board (Sept. 1966).
25. MURPHY, C. J., "Development of a Signal Timing Plan for a Network of Signal Systems." *Proc. Institute of Traffic Engineers,* Vol. 19, pp. 147-160 (1948).
26. NEWELL, G. F., "The Flow of Highway Traffic Through a Sequence of Synchronized Traffic Signals." *Oper. Res.,* Vol. 8, No. 3, pp. 390-405 (1960).
27. NEWELL, G. F., "Synchronization of Traffic Lights for High Flow." *Quarterly of Applied Mathematics,* Vol. 21, No. 4, pp. 315-324 (1964).
28. WEBSTER, F. V., "Traffic Signal Settings." *Road Research Tech. Paper No. 39,* HMSO (1958).
29. YARDENI, L. A., "Vehicular Traffic Control: A Time-Space Design Model." *Proc. Institute of Traffic Engineers,* Vol. 34, pp. 59-74 (1964).
30. YARDENI, L. A., "Algorithms for Traffic-Signal Control." *IBM Systems Journal,* Vol. 4, No. 2 (1965).

PLATOON BEHAVIOR

31. BLUNDEN, W. R., and PRETTY, R. L., "On the Theory of Deterministic Cyclic Traffic Flows in Networks." *Vehicular Traffic Science, Proc. Third Internat. Symposium on Theory of Road Traffic Flow,* pp. 287-299 (1967).
32. BUCKLEY, D. J., TOMLIN, J. A., and MINSON, W. G. á B., "Delays to Traffic Platoons." Paper presented to Third Biennial Meeting, Australian Road Research Board, pp. 1-12 (1966).
33. DAOU, A., "On Flow Within Platoons." *Australian Road Research,* Vol. 2, No. 7, pp. 4-13 (1966).
34. EDIE, L. C., FOOTE, R. S., HERMAN, R., and ROTHERY, R. W., "Analysis of Single Lane Traffic Flow." *Traffic Eng.,* Vol. 34, No. 4, pp. 21-27 (1963).
35. GERLOUGH, D. L., "Some Problems of Intersection Traffic Control." *Proc. Symposium on Theory of Traffic Flow,* GM Res. Lab., Dec. 1959, pp. 10-27 (1961).
36. GRACE, M. J., and POTTS, R. B., "Diffusion of Traffic Platoons." *Proc. Australian Road Research Board,* Vol. I, Part 1, pp. 260-267 (1962).
37. GRACE, M. J., and POTTS, R. B., "A Theory of the Diffusion of Traffic Platoons." *Oper. Res.,* Vol. 12, No. 2, pp. 255-275 (1964).
38. GRAHAM, E. F., and CHENU, D. C., "A Study of Unrestricted Platoon Movement of Traffic." *Traffic Eng.,* Vol. 32, No. 7, pp. 11-13 (1962).
39. HERMAN, R., POTTS, R. B., and ROTHERY, R. W., "Behavior of Traffic Leaving a Signalized Intersection." *Traffic Eng. and Control,* Vol. 5, No. 9, pp. 529-533 (1964).
40. LEWIS, B. J., "Platoon Movement of Traffic From an Isolated Signalized Intersection." *HRB Bull. 178* (1958) pp. 1-11.
41. PACEY, G. M., "The Progress of a Bunch of Vehicles Released From a Traffic Signal." *Res. Note No. RN/2665/GMP,* Road Res. Lab. (6 pp.+14 illus. mimeo.) (Jan. 1956).
42. UNDERWOOD, R. T., "Traffic Flow and Bunching." *Australian Road Research,* Vol. 1, No. 6, pp. 8-25 (1963).

USE OF FUNNELS AND PRE-SIGNALS

43. BIERLEY, R. L., and PARKINSON, J., "The Traffic Pacer System." *Hwy. Res. Record No. 49* (1964) pp. 107-126.
44. ELLSON, P. B., "The Pre-Signal." *Traffic Eng. and Control,* Vol. 6, No. 9, pp. 559-561 (1965).
45. MORRISON, H. M., UNDERWOOD, A. F., and BIERLEY, R. L., "Traffic Pacer." *HRB Bull. 338* (1962) pp. 40-68.
46. VON STEIN, W., "From the Laws of Traffic Flow to Traffic Cybernetics." *Proc. Second Internat. Symposium on Theory of Traffic Flow,* London, 1963. O.E.C.D., pp. 241-248 (1965).
47. VON STEIN, W., "Traffic Flow With Pre-Signals and the Signal Funnel." *Proc. Symposium on Theory of Traffic Flow,* GM Res. Lab., Dec. 1959, pp. 28-56 (1961).

TRAFFIC FLOW CHARACTERISTICS AT INTERSECTIONS

48. ARCHER, R. J. G., HALL, R. I., and EILON, S., "Effect of Turning Vehicles on Traffic Flow Through a Signal Controlled Junction." *Traffic Eng. and Control,* Vol. 5, No. 5, pp. 295-297 (1963).
49. DICK, A. C., "Effect of Gradients on Saturation Flow at Traffic Signals." *Traffic Eng. and Control,* Vol. 5, No. 5, pp. 293-294 (1963).
50. LEONG, H. J. W., "Some Aspects of Urban Intersection Capacity." *Proc. Australian Road Research Board,* Vol. 2, pp. 305-338 (1964).
51. NEWELL, G. F., "The Effect of Left Turns on the Capacity of an Intersection." *Quarterly of Applied Mathematics,* Vol. 17, No. 1, pp. 67-76 (1959).
52. PRETTY, R. L., "The Effect of Right-Turning Vehicles on Saturation Flow Through a Signalized Intersection." Paper presented at Third Conf., Australian Road Research Board (1966).
53. WEBSTER, F. V., "Experiment on Saturation Flow of Right Turning Vehicles at Traffic Signals." *Traffic Eng. and Control,* Vol. 6, No. 7, pp. 427-430, 434 (1964).

GENERAL AND MISCELLANEOUS

54. AVERY, E. V., "Effect of Raising Speed Limits on Urban Arterial Streets." *HRB Bull. 244* (1960) pp. 88-97.
55. BLUNDEN, W. R., and PRETTY, R. L., "On the Theory of Deterministic Cyclic Traffic Flows in Networks." *Vehicular Traffic Science, Proc. Third Internat. Symposium on Theory of Road Traffic Flow,* pp. 287-299 (1967).
56. CULSHAW, T. A., MORRIS, R. W. J., and PAK-POY, P. G., "Comparison of Traffic Signal Controllers Using Computer Simulation—Some Preliminary Results." Paper presented at Third Conf., Australian Road Research Board (Sept. 1966).
*57. DESROSIERS, R. D., and LEIGHTY, C. H., "Traffic Flow Responses to Unannounced Increases in Progression Speeds of Signal Systems." *Pub. Roads,* Vol. 34, No. 1, pp. 1-4 (1966).

58. Fielding, R. H., and Young, T. E., "Analysis of Flow on an Urban Thorofare." *HRB Bull. 107* (1965) pp. 35-48.
59. French, R. A., "Co-ordinated Traffic Signal System—Sydney, Australia." *Traffic Quarterly,* Vol. 19, No. 1, pp. 76-88 (1965).
60. Hewton, J. T., "Toronto Installs All Purpose Electronic Computer for Traffic Signal Control." *Traffic Eng. and Control,* Vol. 5, No. 10, pp. 589-594 (1964).
61. International Business Machines Corporation, Data Processing Division, *San Jose Traffic Control Project—Progress Report* (May 1966).
62. International Business Machines Corporation, *San Jose Traffic Control Project—Final Report* (circa 1967).
63. Marrus, B. S., and Main, M. F., "New Method Improves Traffic Signal Timing." *Traffic Eng.,* Vol. 34, No. 9, pp. 23-26 (1964).
64. Mayr, R., "Cologne's Automatic Traffic Control System." *Traffic Eng. and Control,* Vol. 4, No. 5, pp. 256-259 (1962).
65. Miller, A. J., "Computers in the Analysis and Control of Traffic." *Traffic Quarterly,* Vol. 19, pp. 556-572 (1965).
66. Miller, V. E., "Area Control by Digital Computer." *Traffic Eng. and Control,* Vol. 5, No. 5, pp. 359-365 (1963).
67. Olsen, H. P., and Rothery, R., "The Problem of the Amber Signal Light." *Traffic Eng. and Control,* Vol. 5, No. 5, pp. 298-304 (1963).
68. Pak-Poy, P. G., "Traffic Theory and Practice." *Traffic Eng.,* Vol. 34, No. 3, pp. 25-32, 35 (1963).
69. Pinnell, C., "The Value of Signal Phase Overlap in Signalized Intersection Capacity." *Traffic Eng.,* Vol. 33, No. 3, pp. 17-20 (1962).
70. Thorne, S. M., "Relative Effect of Buses and Cars on the Speed of Traffic." *Traffic Eng. and Control,* Vol. 7, No. 2, pp. 140-141 (1965).
*71. Traffic Research Corporation, *SIGOP: Traffic Signal Optimization Program,* New York (1966).
72. Watjen, W. D., "Signalisierte Strassenkreuzungen." Thesis, Technical Univ. of Denmark, Copenhagen (1965).
73. Wardrop, J. G., "The Capacity of a Network of Streets: Definition and Method of Measurement." Unpublished Lab. Note No. LN/204/JGW, Road Res. Lab., U.K. (Oct. 1962).
*74. Gazis, D. C., and Potts, R. B., "Route Control at Critical Intersections." Research Paper RC-1560, IBM Corp., Watson Research Center, N.Y. (Sept. 1966).
*75. Raus, J., "A Method of Computing Offset Patterns for Multi-Cycle Signal Systems." *Traffic Eng.,* Vol. 33, No. 8, pp. 25-28 (1963).
76. Wagner, F. A., Barnes, F. C., Stirling, D. P., and Gerlough, D. L., "Urban Arterial and Network Simulation." *PRC Report R-926,* prepared for U.S. Dept. of Commerce, BPR, Office of Res. and Development (PB-174629).
77. Little, J. D. C., Martin, B. V., and Morgan, J. T., "Synchronizing Traffic Signals for Maximal Bandwidth." *Report R64-08,* Dept. of Civil Engineering, MIT (Mar. 1964).
78. Yardeni, L. A., *Time-Space Diagram (Progression) Design Program.* SHARE General Program Library, Vehicular Traffic Control, 7040 T4IBM 0005, IBM Corp., Data Processing Div., White Plains, N.Y. (Apr. 1964).
79. Webster, F. V., and Cobbe, B. M., "Traffic Signals." *Road Research Tech. Paper No. 56,* HMSO, London, pp. 57-70 (1966).

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