NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE 172

SIGNAL TIMING IMPROVEMENT PRACTICES

TRANSPORTATION RESEARCH BOARD
National Research Council

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TRANSPORTATION RESEARCH BOARD
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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.

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PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire highway community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff
Transportation
Research Board

This synthesis will be of interest to traffic engineers, public officials, and others interested in developing improved traffic signal timing procedures. Information has been assembled on traffic signal timing software, resources required for timing, procedures for single intersections and coordinated systems, pedestrian intervals, and fine-tuning solutions.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated, and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

Traffic engineers need to know the comparative requirements and effectiveness of alternative traffic signal timing techniques. This report of the Transportation Research Board describes these techniques, presents the general principles for application, including source material for more detailed information, and discusses the issues associated

with traffic signal timing alternatives. It should be noted that, while traffic engineers frequently use standards developed by the American Association of State Highway Officials (AASHTO), the Federal Highway Administration, or other agencies in making engineering judgments, they are always well advised to protect themselves by carefully supporting the bases of their decisions with factual findings and documenting the reasons for the decisions.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researcher in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

CONTENTS

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Special appreciation is expressed to Peter S. Parsonson, Georgia Institute of Technology, who was responsible for the collection of the data and the preparation of the report.

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Richard A. Cunard, Engineer of Traffic and Operations, and Dan A. Rosen, Senior Program Officer, Transportation Research Board, assisted the NCHRP Project 20-5 Staff and the Topic Panel.

Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance were most helpful.

SIGNAL TIMING IMPROVEMENT PRACTICES

SUMMARY

The various techniques and the current knowledge of the art and science of traffic signal timing improvement practices are described in this synthesis. The contents have been assembled primarily from published material and responses to questionnaires sent to a number of professionals considered knowledgeable in the field. There is some original material on timing yellow and all-red phase-change intervals and on flashing operation.

Fundamental principles of the application of techniques are presented and the reader is directed to source material where more detailed information may be found. In cases where the literature on a topic is not readily available, more detail is presented in the text, as in the chapter on single intersections, which describes procedures to calculate the timing of green, yellow, and all-red intervals, including the consideration of pedestrian needs. The chapter on coordinated systems develops the various methods to determine cycle lengths and offsets, and includes practical advice on artful fine-tuning of solutions derived manually or by computer.

Much signal timing continues to be done by manual methods rather than by computer models. The synthesis attempts an evenhanded discussion of these two approaches and tries to give some guidance to the user desiring to blend the two to maximum advantage. Many examples are provided and case studies with detailed examples are included in the appendixes.

Poor signal timing results in needless stop and delay, with attendant energy consumption, operating costs, and detrimental air quality. In an effort to improve this situation, California developed the Fuel Efficient Traffic Signal Management (FETSIM) Program in 1983 that was funded by petroleum overcharge rebates. The program has been extremely cost-effective, because the avoided fuel expenditures have amounted to four times program costs. If benefits are broadened to include savings in motorist travel time and vehicle wear and tear, a 16:1 benefit-to-cost ratio has been the experience.

The reported results of a retiming project can sometimes be misleading, because the measured improvement may be the result of the unreported correction of massive equipment failures that had resulted in poor signal timing. The evaluation report may give the credit for the improvement to the "digital system" or to a certain off-line computer program when, in fact, almost as much improvement would have been obtained from rudimentary but reliable hardware and "hand" settings to provide improved signal timing. Fortunately, there are some large-scale, statewide retiming programs, the results of which are not confounded by other factors.

The synthesis identifies a number of problems that remain largely unsolved. These are listed briefly as follows:

- Need for adequate funding for retiming. Retiming an intersection requires about one person-week of effort and is needed every year in high-growth areas. After the state-allocated petroleum rebate funds have been spent, a need will continue for state and local governments to continue to retime their signals regularly. Many government agencies tend to be reactive rather than proactive in this area, responding only to complaints and crises.
- Need for uniform timing practices and procedures. Despite the availability of desktop computers and good programs, signal timing continues to be more of an art than a science. In addition, some areas of timing are controversial or not well understood. Field fine-tuning, based on observation and judgment, is currently considered essential no matter what office procedures were followed.
- Need for enhancements to the TRANSYT-7F program. This program has enormous potential; some of the improvements requested by traffic engineers are included in the 1991 release.
- Need for cost-effective field data-gathering procedures. Turning-movement counts at intersections still must be obtained manually at great expense. High-tech detection methods are needed to automate data collection.
- Need for strategies to handle oversaturation. The selection of cycle length, especially, is poorly understood.
- Need for proper management of the pedestrian situation at signalized intersections. There are conflicting issues between signal timing practices and maintaining pedestrian safety.
- Need for research on the expectancies of the driving public regarding safe timing of the yellow and all-red intervals.
- Need for guidance for setting the timing intervals of "density" phases on actuated controllers.
- Need for adequate dissemination of information to traffic engineers on the "left-turn trap" hazard that accompanies certain lagging left-turn phasings.
- Need for adequate dissemination of information to traffic engineers on the proper use of flashing red/yellow operation at intersections that are busy or lacking in sight distance.
- Need for a uniform public education program related to signalization.

Correction of many of these problems would require a higher level of funding; agencies at all levels need to find more dollars for needed research, technology-transfer programs aimed at traffic engineers, regularly scheduled signal timing, public education programs, and police enforcement of signal-related laws and ordinances.

CHAPTER ONE

INTRODUCTION

This synthesis presents a compilation of the current knowledge of the art and science of traffic signal timing improvement practices. The scope, however, does not include the topics of lane designation and signal phasing, which are interrelated with timing. Contents of this synthesis have been assembled from various sources, including published material and responses from questionnaires sent to a number of professionals considered knowledgeable in this field. The project was guided by a panel of experts who fixed the scope of the work, suggested names of questionnaire recipients, made their own responses to the questionnaire, reviewed drafts, and furnished helpful comments. Appendix A includes the details of the survey questionnaire and tabulations of characteristics of the 34 engineers who responded.

This introductory chapter is written primarily for the executive or administrator. It underscores the impressive cost-effectiveness of signal-timing efforts, spelling out in dollars and person-hours the resources required for the timing task. Techniques for timing are summarized in a broad fashion, and the chapter concludes with the argument that signal timing is worthy of the expenditure of local funds on a scheduled basis, not merely in reaction to complaints and crises. Chapter Two focuses on the details of timing for single intersections and includes pedestrian considerations and the timing of yellow and all-red displays. The third chapter covers the timing of signals in coordinated systems. Chapters Two and Three begin with coverage of the principles involved, followed by a discussion of current good practice. The responses to the survey questionnaire are woven into each chapter. Chapter Four deals with flashing operation and timing for adverse weather. Conclusions and recommendations of best signal timing techniques are in Chapter Five. A list of references is followed by a bibliography of worthwhile documents that were not mentioned in the text.

The functional objectives of traffic signals are realized mainly through proper timing. These objectives have been set forth in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*) as follows:

> Traffic control signals, properly located and operated, usually have one or more of the following advantages:
>
> 1. They can provide for the orderly movement of traffic.
> 2. Where proper physical layouts and control measures are used, they can increase the traffic-handling capacity of the intersection.
> 3. They can reduce the frequency of certain types of accidents, especially the right-angle type.
> 4. Under favorable conditions, they can be coordinated to provide for continuous or nearly continuous movement of traffic at a definite speed along a given route.
> 5. They can be used to interrupt heavy traffic at intervals to permit other traffic, vehicular or pedestrian, to cross.

For purposes of this synthesis, a signal-timing plan at a single intersection is a unique set of timing parameters comprising the cycle length (the length of time for the complete sequence of the signals), the "split" (the division of the cycle length among the various movements or "phases"), pedestrian requirements for timing, and the phase-change intervals (yellow plus all-red clearance where provided). For a coordinated system, an additional timing parameter must be determined: the "offset," or time from the start of green at one intersection to the start at the next, in an attempt to provide vehicles moving at the proper speed with a green indication at each intersection. When such a "green wave" is successful, it is said that these vehicles enjoy "progressive movement."

Several timing plans are normally required for an intersection or a system, to match the various traffic-volume levels encountered during a weekday, weekends, seasonal variations, and special events. A separate timing plan may also provide for flashing operation, instead of the normal green-yellow-red sequence, at certain times of day.

Both pretimed ("fixed-time") controllers and actuated controllers are discussed. A pretimed controller operates on a clockwork basis, timing the same cycle length and split over and over during an entire period such as a two-hour morning rush. Various timing plans can be developed to suit traffic conditions at various times of day. In downtown grids, traffic may be so stable, or predictable by time of day, as to make pretimed control the logical choice. Actuated control makes use of detectors (sensors) buried in the road, or mounted over it, on some or all of the approaches to the intersection. Actuations received by the controller allow it to give the green only to approaches with waiting vehicles, and to change the signal as soon as they have been served. Actuated control is used where traffic volumes are not steady and minute-to-minute timing adjustment is more important than progressive flow, such as at isolated intersections and along arterials. (An "isolated" intersection is one operated independently of adjacent signals, perhaps not close enough to them to make interconnection worthwhile; it may also be referred to as a single intersection or local intersection.)

For several reasons, much signal timing continues to be done by manual methods rather than by computer models. This synthesis attempts an evenhanded discussion of these two approaches and tries to give some guidance to the user desiring to blend the two to maximum advantage.

It was the expressed desire of the panel that the synthesis serve as a handbook or design manual. Thus, specific recommendations are included where appropriate, with the omission of detailed directions that can best be found in primary references [such as the TRANSYT-7F manual (*2*)].

TRAFFIC ENGINEERING SOFTWARE

A number of signal-timing optimization programs, traffic-simulation models, and other programs are referenced in this synthesis. These are briefly described.

Signal Timing Optimization Software

The following programs are the most widely used for signal timing optimization. As described below, each has its own particular area of application and its own signal timing design philosophy.

SOAP

The SOAP (*Signal Operations Analysis Package*) program develops fixed-time signal-timing plans for individual intersections. SOAP can develop timing plans for six design periods in a single run. It can also analyze 15-min volume data for up to 48 continuous time periods, and determine which timing plan is best suited for each 15-min period. A data input manager is included with the program to facilitate data entry.

PASSER II and MAXBAND

PASSER II (*Progression Analysis and Signal System Evaluation Routine*) and MAXBAND are known as bandwidth-optimization programs. They develop timing plans that maximize the through progression band along arterials of up to 20 intersections. Both programs work best in unsaturated traffic conditions and where turning movements onto the arterial are relatively light. PASSER II and MAXBAND can also be used to develop arterial phase sequencing for input into a stop-and-delay optimization model such as TRANSYT-7F.

The latest version of PASSER II features enhanced program output, explicit treatment of permitted left turns, and a menu-driven, graphical input/output processor. The program also comes with a user-friendly input preprocessor.

TRANSYT-7F and SIGOP-III

The *Traffic Signal Network Study Tool* (TRANSYT-7F) and the *Signal Timing Organization Program* (SIGOP-III) develop signal-timing plans for arterials or grid networks. The objective of both programs is to minimize stops and delay for the system as a whole, rather than maximizing arterial bandwidth.

TRANSYT-7F (Release 6, December 1988) was a cooperative effort by the Federal Highway Administration (FHWA), The University of Florida, and the California Department of Transportation (Caltrans). This version of the program features better treatment of actuated control, a bandwidth constraint capability, and several other new features. The program is completely menu driven and comes with a data input manager and a number of other utility programs to assist in creating data files and displaying results.

Arterial Analysis Package

The *Arterial Analysis Package* (AAP) allows the user to easily access PASSER II, and TRANSYT-7F to perform a complete analysis and design of arterial signal timing. The package contains a user-friendly forms display program so that data can be entered interactively on a microcomputer. Through the AAP, the user can generate an input file for any of the two component programs to quickly evaluate various arterial signal-timing designs and strategies. The package also links to the "Wizard of the Helpful Intersection Control Hints" (WHICH), to facilitate detailed design and analysis of the individual intersections. The current program interfaces with TRANSYT-7F, (Release 7), PASSER II(90) and WHICH.

Recently the AAP software was completely rewritten to take advantage of the latest in microcomputer technology. It features pull-down menus, on-line help, and many automated features to make it easier to use TRANSYT and PASSER together to perform a comprehensive signal-timing analysis and design.

Traffic Simulation Models

Simulation models allow the traffic engineer to evaluate a variety of proposed operational improvements before implementing the changes in the field. The models vary in their levels of detail and are classified as either microscopic (simulation of individual vehicles) or macroscopic (simulation of platoons of traffic). The models also apply to different types of facilities (e.g., signalized networks, freeways, or corridors).

TRAF-NETSIM

TRAF-NETSIM is a microscopic simulation model that provides a detailed evaluation of proposed operational improvements in a signalized network. For example, TRAF-NETSIM can evaluate the effects of converting a street to one way, adding lanes or turn pockets, moving the location of a bus stop, or installing a new signal.

CORFLO

The CORFLO model, formerly called TRAFLO, provides a macroscopic simulation of a corridor containing both signalized intersections and freeways. It also contains a traffic assignment model that can redistribute traffic flows in response to control or geometric changes in the corridor. For example, the model provides a powerful tool for analyzing alternative construction by FHWA, and graphics software is under development to display the input data and the results of the simulation. No release date for CORFLO has been scheduled.

Other Traffic Engineering Software

Highway Capacity Software

The Highway Capacity Software (HCS) replicates the procedures described in the 1985 *Highway Capacity Manual* (*3*). It is a tool that greatly increases productivity and accuracy, but it should be used in conjunction with the 1985 *Highway Capacity Manual* not as a replacement for it.

1.5 Generation

Signal timing plans are typically developed using programs such as TRANSYT-7F or PASSER. This approach is known as first-generation control and is based on off-line signal timing plan generation with manual, time-of-day, or traffic-responsive system plan selection. There are alternative control strategies for automatic generation of timing plans while the system is on-line

and operating that are known as second-generation control. These programs automatically collect detector data, perform calculations of signal timing, and immediately implement the plans on the system. These systems require the installation of numerous detectors in order to have the detailed traffic flow data required to calculate effective timing plans. This is an expensive installation and has not been used in the United States.

An alternative strategy, known as the 1.5 generation of control, is being developed in an effort to have a simpler, less expensive way of generating new signal timing plans. It incorporates many of the automatic features of the second-generation programs in terms of the automatic linkage between traffic flow data and the timing programs and between the timing programs and the signal timing data base. In addition to providing this automatic linkage, the 1.5 generation of control provides for a manual review and adjustment of the timing plans. This reduces the need for numerous detectors, because there can be user intervention to compensate for the errors produced because of limited data during the manual review process.

The TRANSYT-7F program is used for the 1.5 generation control because it can be used for all types of street networks.

IMPACTS OF GOOD AND BAD SIGNAL TIMING

Webster and Cobbe (*4*) have shown clearly the impact of cycle length in terms of delay for random arrivals. Figure 1 shows for each of four different volume levels at a single intersection that there is an optimum cycle length, C_o, that minimizes delay. In this example, varying the cycle length from three-quarters to one and one-half the optimum does not increase delay by much, but very short cycle lengths cause delay to skyrocket. Delay in a typical arterial signal system is a much more complex subject than Figure 1 suggests. The Transportation Research Board's *Highway Capacity Manual* (*3*) addresses this complexity in depth; this synthesis addresses the impact of better timing.

Some respondents to the survey questionnaire routinely document the effectiveness of improved timing by means of before/after studies and simulation comparisons showing increased speed and decreased delay even with increased traffic volumes. Summary results of a few before/after studies were reported. The benefits were measured by travel time studies and appeared be attributable to any one of a combination of factors including improved timing and/or phasing, reliable coordination where that had been lacking, changing from fixed-time to actuated equipment, new detectors and loops, repair of malfunctioning controllers, etc. The effects of signal timing alone were not well-isolated. It appears that estimates of the impacts of improved signal timing alone are best obtained from the large-scale retiming projects described in the next sections.

The questionnaire uncovered one before/after study that seemed to be clearly focused on "improved coordination" achieved only by running a computer program oriented to arterials. The routes were located in Toronto and included a downtown one-way pair, a suburban arterial commuter route, and two commercial arterials. The quality of the "before" coordination, if any, was not described. Annual fuel savings of almost $1,000,000 allowed the project cost of less than $30,000 to be paid back in less than two weeks.

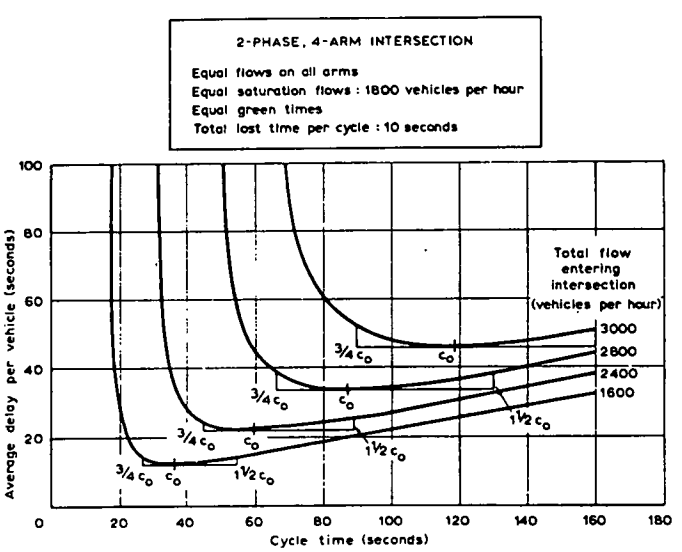


FIGURE 1 Effect on delay of variation in cycle length.

National Signal Timing Optimization Project

The National Signal Timing Optimization Project was initiated by FHWA in 1980 as a fuel-conservation effort in response to the high cost of imported oil. The project report made clear the opportunity at hand (*5*):

> It is estimated that approximately one-fifth of the total daily U.S. oil consumption is used by vehicles traveling in urban areas through signalized intersections. A significant portion of this is wasted due to poor traffic signal timing. In street networks with poorly timed traffic signals, the fuel consumed by vehicles stopping and idling at traffic signals accounts for approximately 40 percent of network-wide vehicular fuel consumption. Improving traffic signal timing will improve the quality of traffic flow 24 hours per day, 7 days per week with no sacrifice required on the part of the individual. Driving is made faster and easier for all cars, trucks, and buses using the street system.

The project established credible data on the effectiveness of signal-timing optimization, made optimization easier to do through the development of a computer program called TRANSYT-7F, and assisted the budgeting process by determining the resources required (such as person-hours of various levels of staff) for a timing project.

Personnel from 11 cities around the United States were trained to use TRANSYT-7F to optimize the timing of a portion of the street network. They implemented the new timings, evaluated their effectiveness, and kept close records of labor and other resources.

The project results showed that, assuming gasoline costs of $1.35 a gallon, a benefit/cost ratio of at least 10 to 1 conservatively can be expected for first-time projects. With an experienced staff, retiming costs per intersection were estimated to drop significantly, resulting in a 15 to 1 ratio. When the evaluation also included the cost of time saved and savings from the elimination of non-fuel vehicle-operating costs, the benefit/cost ratio jumped to 20 to 1 for first-time projects and 30 to 1 for projects by experienced staff. These latter results were obtained using an ultraconservative value of time of $0.50/vehicle/hr; values of

time 10-fold greater could have been justified by the standards of the American Association of State Highway and Transportation Officials (AASHTO) (*6*), resulting in even higher benefit/cost ratios.

State Programs for Signal-Timing Optimization

A 1988 Virginia report by Arnold (*7*) summarized signal-timing optimization studies in California, Florida, Illinois, Maryland, Michigan, Missouri, North Carolina, and Wisconsin. As of late 1989, similar projects were also under way in Iowa, New Hampshire, New York, Nevada, Pennsylvania, Texas, and the state of Washington. Most of these programs have been funded totally or in part from oil overcharge money allocated to the state.

Arnold found five categories of program costs, as follows:

- Services: Promotion, training, and technical assistance.
- Retiming: Data collection, timing-plan development, and plan implementation.
- Retiming-related equipment: Computer software, data-collection equipment, computers, etc.
- Minor equipment: Time-based coordinators, additional controller phases and detectors, etc.
- Major equipment: Controllers, hard-wire interconnect, system masters, etc.

He found that the states varied widely in their policies for eligibility of these costs, as shown in Table 1.

Arnold reported that most states evaluated the effectiveness of their retiming projects in saving fuel and improving operations. Some programs relied solely on computer-derived evaluations, whereas others required field studies. California "concluded from a controlled experiment using an instrumented vehicle that the TRANSYT software produces reasonably accurate estimates of savings" (*7*). The next two sections summarize the excellent results of the programs in California and North Carolina. Comparable results were obtained from similar programs in Florida and Missouri (*7*).

California's FETSIM Program

California's Fuel Efficient Traffic Signal Management (FETSIM) Program began in 1983. Funded through Caltrans, grants are available to local governments. Through 1991, 263 grants had been awarded to 142 separate local public agencies. These grants, totaling $11,000,000, were used to retime more than 9,000 signalized intersections, nearly one-third of the state's total.

The program is limited to coordinated systems with eight or more signals. For the retiming calculations (*8*), "TRANSYT-7F was selected because it is capable of handling complicated networks, because it has been thoroughly field tested, and because it directly produces estimates of delay, stops, and fuel consumption."

Initially, most local agencies employed consultants to complete retiming in the FETSIM projects. This situation changed as the tool (TRANSYT) became easier to use. For the 1990 FETSIM grant cycle, 18 of 30 projects were conducted by local public agency engineering staff. In 1991, FETSIM expected local staff to do the analysis of all projects with fewer than 15 intersections.

The state cost per signal, including retiming, training, and technical assistance, for the most recently completed cycle (1988) was approximately $1,500. Expenditures are allowed for all aspects of timing: data collection, data processing, timing-plan development, implementation, and field evaluation. The benefits from the program through 1988 were substantial, with average first-year reductions of 14 percent in stops and delay, 7.5 percent in travel time, and 8.1 percent in fuel use. Avoided fuel expenditures in the first year are four times program costs. Using a broader but widely accepted estimate of benefits that includes travel time and vehicle wear and tear savings, a 16:1 first-year benefit-to-cost ratio results.

TABLE 1
ELIGIBLE PROGRAM COSTS IN VARIOUS STATES (*7*)

			Equipment		
Program	Services	Retiming	Retiming-Related	Minor	Major
California FETSIM	X	X	X	X[a]	
Florida GASCAP	X	X[b]			
Florida STSRP		X			
Illinois SCAT		X		X	
Maryland STSSP		X	X	X	X
Michigan TSOP		X		X	
Michigan TSMP				X	X
Missouri TRANSYT-7F	X		X[c]		
North Carolina TSMP		X	X		
Wisconsin FET	X	X	X	X	X

[a]Special demonstration projects.
[b]Special consultant subcontract with the Transportation Research Center.
[c]Participants given electronic turning movement counter.

California's experience provides excellent guidance to others considering developing their own programs.

North Carolina's Traffic Signal Timing Optimization Program

Another example is the program conducted in North Carolina through the University of North Carolina Institute for Transportation Research and Education (ITRE), in cooperation with the Energy Division of the North Carolina Department of Commerce (*9*). Like California and many other states, North Carolina used petroleum account monies to optimize traffic-signal timing. ITRE conducted a program from May 1985 through November 1987 to retime and perform minor repairs on 980 of the 4,600 traffic signals on the state highway system. The locations were primarily isolated and had semi- or fully actuated controllers. The retimings were calculated in the field by the critical-lane method and in the office using TRANSYT-7F (Release 4). This computer program was also used to estimate the annual energy (fuel) savings and total annual savings in costs of fuel, delays, and stops.

The 30-month program cost $470 per intersection. The TRANSYT-7F program estimated an annual saving of 12.2 million gallons of fuel. This amounted to an average savings per intersection of 13,900 gallons of fuel and $51,815 in operating costs. The ratio of fuel-cost savings to direct program costs, assuming fuel at $1 per gallon, was 29 to 1. The ratio of total operating cost savings to direct program costs was 108 to 1. (Some of these huge benefits may be because TRANSYT-7F may overestimate benefits if the "before" condition was oversaturated and the "after" was not. This is particularly true when the "before" oversaturation lasted less than an hour.) In any event, these results were impressive, and the program is continuing with the goal of eventually optimizing the timing of all 4,600 signals on the state highway system.

The North Carolina experience reinforced the evidence from around the United States and Canada that signal retiming is extremely effective in reducing fuel consumption and the cost of fuel, stops, and delay. The bibliography lists many publications and project reports that further document this effectiveness.

Interpretation of Retiming Results

Sometimes the results of a retiming project are not what they appear on the surface and require interpretation. Two examples are offered:

First, the measured improvement may be caused less by better timing than by the unreported correction of massive equipment failures. The "before" condition may have been close to chaos, with frequent controller malfunctions, many failed detectors, and the loss of coordination resulting from faulty interconnect. The evaluation study may give the credit for the improvement to the "digital system" or a certain off-line computer program, when, in fact, almost as much improvement would have been obtained from rudimentary but reliable hardware and "hand" timings.

Second, the success of a retiming project often depends on the measures of effectiveness chosen, particularly on arterial projects. A computer program may use measures of effectiveness to minimize stops and delay, on main line and side street alike, without preference given to one traffic stream or another. How successful is a computer-derived timing plan that sacrifices arterial progression in order to benefit side-street flow? On the other hand, when the arterial carries heavy volumes of out-of-jurisdiction vehicles, the point of view can be just the reverse. How successful is a project that minimizes delay to predominantly through traffic at the expense of the local drivers who feel the loss of freedom of movement?

RESOURCES REQUIRED FOR TIMING

Unless an area is experiencing no growth or other change, a pretimed traffic signal controller in place for more than a year or so probably needs retiming. Actuated control has more flexibility and adaptability to changing volumes, but still demands periodic retiming and optimization. This section summarizes the resources in personnel and dollars required. Because the frequency of retiming is central to the determination of resources, it is discussed first.

The Decision to Retime

The survey questionnaire asked, "How do you decide when a signal or system needs retiming?" Most respondents answered that complaints from motorists provide the primary motivation. Also mentioned were input from the agency's own personnel as they drive around the area and from police and other local authorities; field observations of longer queues, wasted green time, or other form of degraded operation; judgment; changed traffic patterns; change in speed zone; change in accident frequency or a specific accident pattern; geometric changes; significant roadway closures and detours; additional signals within the control system; and development. From a Caltrans engineer:

> In some cases, the addition of a new signal on an arterial may influence timing at an existing intersection. One example might be a decrease in left turns at the existing signal or a decrease in side-street volumes if more than one signal is available to provide access to the arterial.

However, availability of staff time is the critical deciding factor. The comments from the state of Delaware expressed the frustration indicated by many respondents:

> Unless an area has zero growth, any system in place for more than a year needs retiming. The question is, How does one get the resources to do the job to the top level of his ability? I don't have an answer. Perhaps, one of the reasons I select actuated equipment and highly flexible system operation is to allow the equipment to compensate to the extent possible for the lack of engineering attention which I know it will get.

A West Coast consultant explained that the decision to retime is made "when citizen complaint reaches the point of forcing something be done."

A yearly review seems to be widely recognized as a reasonable standard for retiming locations where travel patterns and volumes are not static. "Over the past five to six years our traffic volumes have increased anywhere from 10 to 40 percent on some of our arterials, so signal re-timing becomes an annual program,"

said a city engineer. Another response estimated two to three years. A 10 percent improvement is all that is needed to make retiming "very worthwhile."

Resources Required by Various Programs

The survey questionnaire asked for any data on the resources in personnel and dollars needed for retiming or detection of malfunctions or equipment upgrading.

It appears to be a rule of thumb that one person-week is required to retime an intersection; it follows that 50 intersections can be retimed by the allocation of one person-year of time. (Of course, in practice a team of persons with various skills is used.) A substantial amount of data must be collected if a signal is to be retimed by means of the most comprehensive computer programs. The *TRANSYT-7F User's Manual (2)* devotes a chapter to data-collection requirements and summarizes them, as shown in Table 2..

Required computer resources include machine time, and in the past often included the services of a computer specialist (systems analyst). Computer-use data from the early 1980s, such as the National Signal Timing Optimization Program and the early FETSIM studies, should be interpreted with caution, because mainframes were used to run TRANSYT-7F. By the late 1980s this program was being run routinely on desktop equipment. This development greatly reduced computer charges and training; moreover, the ease of use is conducive to performing many more iterations than before. The engineer now has time to explore a number of "what if" scenarios in order to arrive at the best solution. The TRANSYT-7F program is becoming easier to use, thanks to several data input managers that have been developed. For example, in 1991 Caltrans fully tested the Quick-7F preprocessor in six cities in Northern California. Based on successful results, Quick-7F will be implemented statewide in 1992. The preprocessor is designed to ease data entry and testing of various options.

The city of Stockton, California, reported its two FETSIM projects cost approximately $700 to $950 per intersection, including consultant. A consulting firm in Atlanta stated that it normally estimates $1,000 per intersection to obtain peak-hour counts and develop four or five timing plans.

A Southeast consultant estimated $1,800 per intersection for services including equipment inventory, traffic counts, timing plans, and equipment recommendations. Another in the same area charges $1,000 per intersection for 8-hr turning-movement counts and the development of five timing plans. A Mississippi consulting firm estimates $500 per intersection, based on 10 hr labor, 6-hr turning-movement counts, various 24-hr counts, and use of TRANSYT-7F.

Lakewood, Colorado, estimates 40 hr/intersection to fully retime a signal. "We contract with consultants to provide many of our TRANSYT runs as well as the travel time and delay studies. . . ." The city of Huntsville, Alabama, has 210 signalized intersections and budgets 1.5 personnel positions, at a total cost (including benefits) of more than $40,000, for detecting malfunctions and retiming traffic signal equipment. This city also has an aggressive program to convert to Type 170 controllers at a rate of 25 per year.

Montgomery County, Maryland, has 564 signalized intersections, of which 542 are controlled by a central computerized system and 22 are isolated. The engineering staff has six full-time positions for signal design, construction, coordination, oversight, timing, system parameters, and routine surveillance. About 60 percent of staff time is devoted to timing, detection of malfunctions, and other maintenance-oriented activities. Said the respondent, "We could do a better job if we had more." The signal shop staff has 23 full-time positions for installation, construction, cabinet setup and testing, modifications, repairs, communications cable maintenance, etc. Approximately 60 percent of the shop staff's time is devoted to maintenance activities, including trouble calls, replacing loop detectors, bench repair work, and troubleshooting problems in systems-communications cables. Montgomery County's maintenance budget for the 1989–1990 year is shown as follows:

Salaries and Fringe Benefits (32%):		
Signal Engineering: $320,000 × 60% time		$192,000
Signal Shop: $968,000 × 60% time		581,000
Overtime and Standby Pay for Signal Maintenance		30,000
		$803,000
Motor Pool Charges for Signal Surveillance and Maintenance Vehicles		$130,000
Supplies, Uniforms, etc.		$34,000
Repairs of Equipment by Vendors, including Maintenance of Central Computer Equipment		$16,000
Signal Parts and Components for Maintenance		
Funded from Operating Budget		$188,000
Charged to Capital Improvement Funds (knockdowns, replacements of controllers and detectors)	est.	$500,000
	Total	$1,671,000

Assuming that the $192,000 for signal engineering salary and benefits is substantially for timing, the cost amounts to $340 per intersection per year, or about $1,000 to retime each intersection every three years.

Retiming can be made easier. One consultant said that "consultants should be made to deliver their data files on magnetic media, along with link-node diagrams, etc., to the client, so that updates can be made with a minimal amount of recoding effort!"

SUMMARY OF TECHNIQUES AVAILABLE

A signal-timing plan can be developed by "feel" (experience or judgment), by manual calculations, by computer program, or by a judicious blend of the three. All methods require "artful" fine-tuning (i.e.,optimization) based on field observation, and all require updating every few years because of changing conditions.

This section briefly summarizes the techniques available for determining and updating the following elements of a timing plan:

- Cycle length and its "split" among the movements
- Pedestrian intervals
- Offsets in a coordinated system
- Phase-change intervals
- Flashing operation

TABLE 2
SUMMARY OF TRANSYT-7F DATA REQUIREMENTS (*2*)

Major Category	Data Types		Data Source	
Network Data	1.	Nodes (intersections)	1.	Maps, drawings, aerial photographs
	2.	Links (streets)	2.	Maps, drawings, aerial photographs
	3.	Link distances (stopline to stopline)	3.	Maps or field measures
	4.	Parking/turn restructions	4.	Maps or field inventory
	5.	Bus routes	5.	Bus company
Timing Data	1.	Existing cycle lengths	1.	Timing plans
	2.	Existing offsets	2.	Timing plans
	3.	Existing interval durations and phase lengths	3.	Timing plans (except for semi-actuated, which normally require field studies)
	4.	Phase sequences	4.	Timing plans
	5.	Minimum greens	5.	Calculated by user
Saturation Flow, Lost Time, Green Extension, and Sneakers	1.	Saturation flow	1.	Field studies (guidelines available for estimates)
	2.	Start-up lost time	2.	Guidelines provided or field measure for special cases
	3.	Green extension time	3.	Guidelines provided or field measure for special cases
	4.	Sneakers	4.	Guidelines provided or field observations
Speed Data	1.	Cruise speed on the links	1.	Field studies
	2.	Bus dwell times	2.	Field studies or bus company data
Volume Data	1.	Total volumes on a sample of blocks	1.	Day-long field studies
	2.	Turning movement counts	2.	Manual field studies
	3.	Link-to-link movements	3.	Sampling studies (or estimated)
	4.	Classification studies	4.	Sampling studies
Control Data	Program controls and parameters		User determined	

A pretimed controller usually is applied in a coordinated system such as a downtown grid. The cycle length is normally the same at each intersection in the system, although a lightly traveled intersection may be operated at one-half the system cycle length. Longer cycle lengths provide greater capacity, at least up to a point of diminishing returns, because the fewer cycles per hour the less time lost as queues get into motion at the start of the green. Also, there are fewer losses caused by stopping the stream at the end of the yellow.

The cycle length needed to provide sufficient capacity is determined for each intersection, either by hand calculation or by a computer program such as SOAP84 (*10*). Manual calculations commonly make use of queue-discharge observations performed by Greenshields in the 1940s that are still essentially valid today (*11*). With this formula, the green time required for a phase is determined for the number of vehicles expected in the heaviest-traveled ("critical") lane. The greens for each phase are summed along with the phase-change intervals to give the required cycle length.

If the controller is of the actuated type, the "maximum interval" for a green phase is selected to be somewhat longer than the calculated time, to allow for brief surges or peaks in arriving volumes.

If pedestrians are present at this location, the sum of the green and phase-change times is checked for adequate duration to allow safe crossing. If it is insufficient, the traffic engineer must either increase the green time or specify an actuated controller with pedestrian push buttons to extend the green time long

enough for pedestrians. The engineer may agonize over this decision, because merely increasing the green will cause unnecessary delay on the busy artery during the many cycles when no pedestrians are present. On the other hand, if the green remains short and efficient except when the push button is used, a pedestrian not bothering to push it would be endangered by too short a green.

An interconnected system requires, in addition, the determination of the offset in the start of green from one intersection to another, to promote nonstop (progressive) movement in one, or possibly both, directions for vehicles traveling at the designed speed. Progressive movement greatly improves capacity. The most common practice in the United States is to prepare a "time-space diagram," usually by hand but increasingly by computer program such as PASSER II (*12*) or TRANSYT-7F. The diagram is a graphic indication of the extent to which selected offsets produce a "green wave" or "cascade effect" along the arterial. Movable paper strips colored with the reds and greens of the cycles can be artfully adjusted at each intersection to optimize the progression. The computer programs may add further graphics, such as a "platoon progression diagram" or a histogram of vehicle arrivals and departures during the cycle, that assist in the fine-tuning of the progression. These programs may provide calculated values of stops, delay, fuel consumption, etc., and compute signal timings that will minimize these values for all traffic or for selected movements (such as through traffic on the arterial).

Calculation of the timing of the yellow change interval is performed manually according to established formulas. The all-red clearance interval is not as well-defined and is commonly determined by local policy and intersection-specific conditions. The prudent traffic engineer is especially careful with the yellow, all-red, and pedestrian intervals because of the potential for related accidents causing damage, injury, and litigation.

Flashing operation is the backup mode of operation when normal control is malfunctioning, and may be used for times of day with very light traffic volumes. Through movements on the major route are normally shown flashing yellow, and all other movements are shown flashing red. The intersection then operates on a see-and-be-seen basis, as under STOP sign control. If that should be dangerous because of heavy volumes or because stopped drivers have an obstructed view, the plan for flashing operation should be designed to assure appropriate safety.

Much more detail on the techniques of timing is provided in Chapters Two, Three, and Four.

CHOICE OF TWO MANAGEMENT PHILOSOPHIES

The National Signal Timing Optimization Project and those in the states show what can be done when funds earmarked for better timing are made available. What, then, should be the policies of local governments to retime regularly if there is no external funding? Experience has shown that cities, counties, and states tend to be reactive rather than proactive, with the result that retiming projects often receive low priority. This section suggests that road users educated in the availability, characteristics, and advantages of efficiently timed signals could demand that local funds be made available for scheduled retiming.

Small fuel savings at prices of only $1 per gallon may be of little interest to the individual motorist. However, stops and delay are another matter. Traffic engineers know well that there are many motorists who become quite vocal when they experience stops and delay that seem to be avoidable. Safety, too, is seen locally as an important goal, particularly for pedestrians. It appears that good signal timing will flourish only to the extent that it can generate a strong constituency, and signals will be retimed in response to complaints and crises.

Respond to Complaints and Crises

One large city in the Southeast is known to have an informal policy by which the signals are to be operated and maintained to be safe for motorists and pedestrians, but will not necessarily be timed to minimize stops, delay, fuel, etc. In another instance, the traffic engineer of a large county once said that he was resisting the installation of signal systems that would monitor equipment performance and automatically report malfunctions. He already had a long list of failed detectors and saw no point in learning of more. In both examples, the goal seems to be to minimize expense and trouble to the agency and its overburdened employees rather than to maximize service to the taxpayer or the road user. What counts is the agency budget rather than the overall cost to the community. In this context, a malfunctioning detector really has not failed until a complaining phone call is received, and the timing of a signal or system is by definition fine until a protesting voice is heard.

Under these circumstances, an educated road user, skilled in recognizing stops and delay that are unnecessary, and effective in voicing complaint to traffic engineers and elected officials, would be needed.

Educate the Public and Public Officials

Management by complaints and crises puts quite a burden on the motoring public to know when a complaint is in order. Drivers seem to have varying degrees of awareness of malfunctions and poor timing, and differing levels of tolerance to them. Traffic engineers and administrators may be aware of widespread detector failures in their jurisdiction, with hundreds of actuated phases operating "on recall," like a pretimed controller. Seldom, if ever, does that information make its way into the newspapers. The fact seems to be that most motorists have only a vague and imprecise impression of the quality of signal timing and maintenance. Short of crisis, they do not know enough to complain; stopping for a red signal may or may not be appropriate at any given intersection and time. In general, the motoring public is not in a position to hold the traffic engineering agency accountable for excessive stops and delay. This fact is probably an impediment to local agencies' ability to secure local funds to time signals. Educating the public and officials to the availability and characteristics of traffic-responsive signalization could help to create an "informed consumer." Increased complaints may be matched by higher allocations for better roads and streets, and gradually agencies may find local dollars for regularly scheduled signal timing.

CHAPTER TWO

SINGLE INTERSECTIONS

Whether signal timing is done by manual methods or by computer modeling, the traffic engineer needs to know the underlying principles. Lacking that knowledge, the engineer is in a poor position to interpret the results properly and fine-tune them to actual field conditions. Computer-modeling, in particular, becomes merely an exercise in coding—an act of blind faith. Engineering for public safety and convenience requires more.

Chapters Two, Three, and Four include special mention of principles critical to the safety of road users and to the liability of the engineer or the agency. Advice on the extent of the engineer's duty in applying these principles is offered, because the courts are free to look beyond professional standards, such as the MUTCD, for example, and to require the engineer to apply principles of traffic flow and road-user characteristics to account for intersection-specific conditions.

With an understanding of the principles of vehicle and pedestrian flow and a good idea of what is desired as the outcome, the traffic engineer can make an art of tailoring manual and computer-based timings to the unique combination of characteristics at a particular intersection or in a coordinated system. Artful fine-tuning will continue to be needed until the eventual development of an all-encompassing, computer-based expert system (and will still be needed as long as timings are based on ever changing traffic counts and conditions).

This chapter discusses cycle length and split, pedestrian intervals, and phase-change intervals. For each subject, an explanation of the principles is followed by pertinent responses from the survey questionnaire.

CYCLE LENGTH AND SPLIT

This section explains the traffic-flow principles underlying the selection of cycle length and its division ("split") into phases and intervals at single intersections.

Chapter One explained that longer cycle lengths provide greater capacity, at least up to a point of diminishing returns. Figure 1 showed that higher volumes require longer cycle lengths and that for any volume level there is an optimum cycle length that minimizes delay.

The Transportation and Traffic Engineering Handbook (*13*) states:

> The objective of signal timing is to alternate the right of way between traffic streams so that the average delay to all vehicles and pedestrians, the total delay to any single group of vehicles and pedestrians, and the possibility of accident-producing conflicts are minimized. These criteria frequently conflict and require compromise based on engineering judgment.

In practice, it may be the goal to minimize the delay to selected traffic. For example, motorists on the major route may be given priority over those on the intersecting minor road. Calculated signal timing may require adjustment so that critical queues will not become so long that they overfill left-turn bays or threaten the operation of intersections upstream.

Short cycle lengths provide "snappy" traffic operation resulting in low average delay, provided the capacity of the cycle to service vehicles is not exceeded. Short cycles have relatively low capacity because over any time period their larger number of starting delays and yellow intervals will produce more lost time and thus accommodate fewer total vehicles. In some cases, this may be offset somewhat if the short cycle enables better progression with attendant greater capacity. However, the larger number of cycles per hour will allow more vehicles to "sneak" a permitted left turn at the end of the yellow, thus providing greater left-turn capacity. (Although they are termed "sneakers," these vehicles have entered the intersection legally on the green or yellow, and complete their turns legally on the yellow or red.)

Longer cycle lengths will accommodate more vehicles per hour, but only if demand continues throughout each green. Cycles longer than necessary to serve the demand present will produce higher average delays. As the cycle and green intervals lengthen, there is a lower probability that demand will be present during the latter portions of the green phase, whereas a higher average delay must occur for all vehicles waiting on the corresponding red phase. This produces a higher total delay.

These principles suggest the following axiom with regard to cycle length: Consider the shortest cycle that will accommodate the demand present and thus will produce the lowest average delay. (If pedestrians are present, check to ensure that they will have sufficient time to cross, as discussed later.)

Often the cycle length is determined by finding each green time needed to serve the demand, and adding these times with the yellow and all-red times. These principles begin with a discussion of the capacity of a green interval.

Basics of Green Interval Capacity

The capacity of a green interval is related to the rate at which a queue of waiting vehicles will enter the intersection after the signal goes green. The General Electric Co.'s Illuminating Laboratory made such measurements of automobile discharge in the early 1940s. Although they did not express their results as an equation, their unpublished graphs showed the average amount of green time required was 3 sec/car when 4 cars were waiting, and 2.5 sec/car when the queue was 10 to 12 cars long. In 1947, Greenshields published similar data in forms that included an equation still used widely today (*11*). The field results for passenger cars on a level approach are shown in Table 3. These are the times required for the vehicle to react to the signal or the car

TABLE 3
GREENSHIELDS' QUEUE-DISCHARGE DATA (*11*)

Car in Line (number)	Green Time Consumed (median value) (seconds)	Cumulative Times (seconds)
1	3.8	3.8
2	3.1	6.9
3	2.7	9.6
4	2.4	12.0
5	2.2	14.2
6 and over	2.1	16.3

ahead, get into motion, and reach the stop line or enter the intersection. A graph of Greenshields' cumulative times (Figure 2) shows that the time t for the nth car to reach the stopline or enter the intersection is

$$t = 3.7 + 2.1n \text{ or, more commonly, } t = 4 + 2n \quad (1)$$

The 3.7 sec is the start-up "lost time" per phase and the 2.1 sec is the equilibrium headway once the first five or six cars have discharged.

The capacity of a green interval, or of a cycle length, can now be calculated. For example, the capacity of an 80-sec cycle, split into two phases, is derived as follows:

Cycle Length	Minus Yellows and	All Reds	Total Green	50/50 Split	Green Phase
80 sec	−5	−5	=70	/2	=35

Greenshields	n/ Phase	Two Phases	Veh/ Cycle	No. of Cycles/hr	Veh/ hr
4 + 2n = 35	15.5	×2	=31	×3600/80	=1400

Note that Greenshields' formula is applied to each phase, not to the total green of the cycle. Figure 3 is a graph of the results of similar calculations for other cycle lengths. This figure shows that, at a two-phase intersection, increasing the cycle length above roughly 80 seconds yields diminishing returns in capacity. As mentioned earlier, longer cycle lengths also reduce the capacity to sneak the left turn at the end of the yellow.

Table 3 shows that, once the first five cars have discharged, flow continues at an equilibrium headway of 2.1 sec. If a stream of cars were to flow for an entire hour (3,600 sec) at that headway, the throughput would be 3,600/2.1 = 1,714 cars/hr of green. This value is an example of a "saturation flow rate."

Adjustment for Trucks and Turning Vehicles

Greenshields obtained his data for cars discharging straight through the intersection. Adjustments for trucks, buses, and

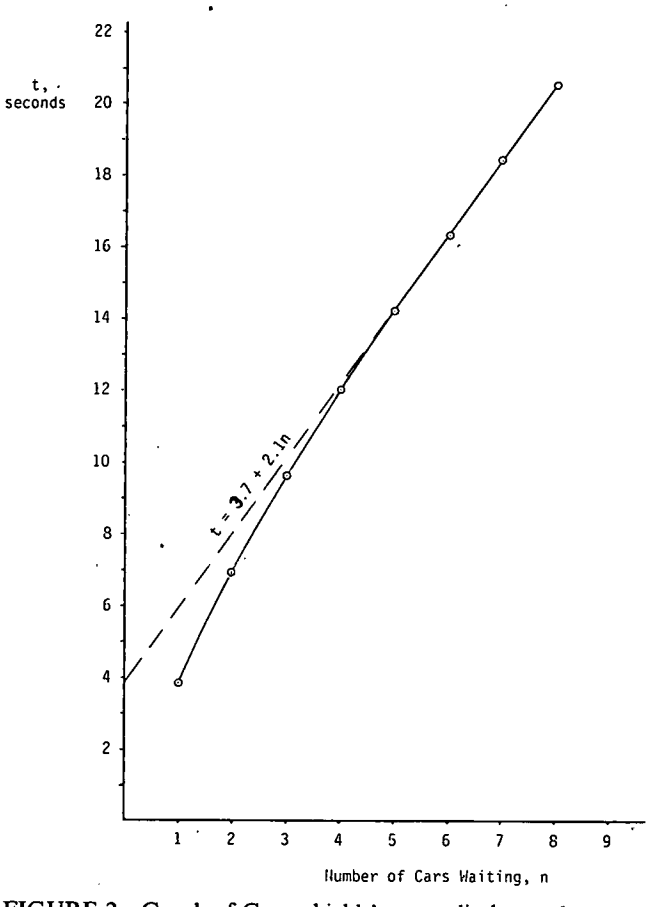


FIGURE 2 Graph of Greenshields' queue-discharge data (*11*).

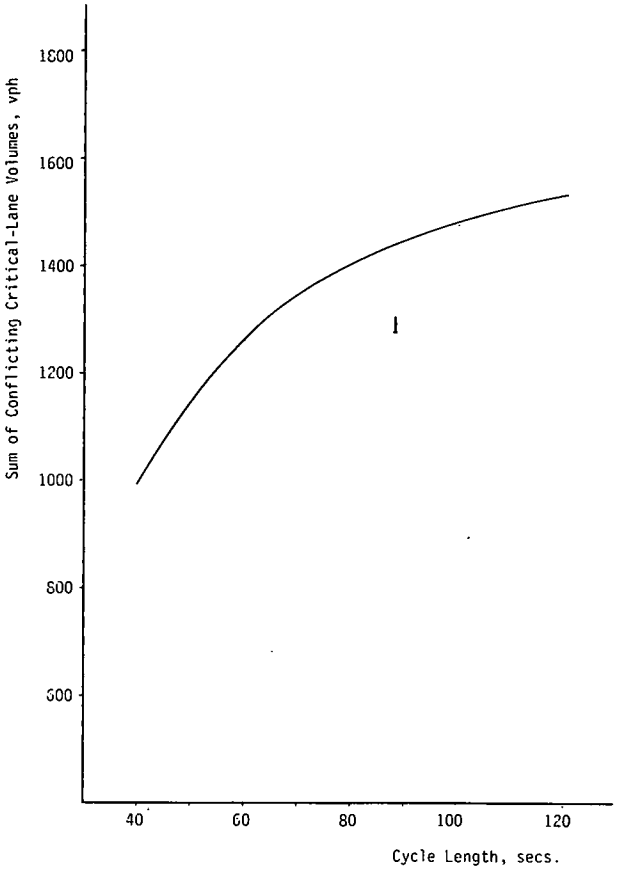


FIGURE 3 Capacities of cycle lengths for two-phase signalization.

turning vehicles are often made using the principle of "passenger car equivalents" (PCEs). For example, a truck may be assigned a PCE of 2 if its start-up time is twice that of passenger cars. Hourly volumes of mixed traffic (cars, trucks, buses) are converted to an equivalent number of straight-through passenger cars per hour by use of these multipliers.

Green-Interval Capacity at Specific Approaches

The procedures of the *Highway Capacity Manual* (*3*) can be used to obtain the capacity of a green interval for a specific location with greater accuracy than is possible with Greenshields' typical values. This reference recommends that, for ideal conditions, the start-up lost time be taken as 2 sec, lower than Greenshields' 3.7, because trends to urbanization over the decades quicken the pace of life and tend to produce drivers "itchy" to get away when the signal changes. Capelle and Pinnell (*14*) are examples of observers who have reported values of about 2 sec. For the clearance lost time, associated with the yellow and any all-red, the *Highway Capacity Manual* recommends 2 sec. (Overlap movements or phases have no lost time because of clearance.) The sum of the two lost times is 4 sec, and tends to range generally between 3 and 5. The manual suggests using the change interval (including any all-red) as an estimate of this sum. For any given lane or movement, then, the manual considers that vehicles use the intersection at the saturation flow rate for a period called the effective green time. This is calculated as the actual green time plus the change interval(s) minus the start-up and clearance lost times. The effective green time may be approximated as the actual green time.

Saturation flow rates can be estimated for a specific approach using the *Highway Capacity Manual* by starting with an ideal value of 1,800 cars/hr of green and adjusting it for several aspects including lane width, trucks, grades, parking, bus blockage, turning movements, and location. (For example, in a central business district, capacity tends to be reduced by construction, double parking, etc.)

Critical approaches may require field measurement of lost times and saturation flow rates. The *TRANSYT-7F User's Manual* (*2*) provides detailed instructions and sample forms for collecting these.

Before the cycle length can be calculated, demand volumes and the critical-lane concept must be discussed.

Demand Volumes and the Critical Lane

Figure 3 explained the basics of capacity of green intervals and cycle lengths. A capacity of 1,400 vph was derived from a cycle length of 80 sec at a two-phase signal. This cycle has the capacity to pass an average arrival rate of 1,400 vph. If arrivals fluctuate from minute to minute above and below the average, then about half the time the cycle will not have enough capacity to pass all the vehicles arriving in that cycle. The cycle is said to have a 50 percent probability of "performance" or clearance. This is the same as a 50 percent rate of cycle failure. It corresponds to flow at "capacity" or level of service (LOS) = E. Now, if a cycle is determined using a peak-hourly volume from 4:30 to 5:30 p.m., for example, and there is a 15-min surge at 5:00 p.m., it is clear that the cycle length will be inadequate during the peak 15 min. Traffic engineers attempt to design signal timing so that the desired level of service will occur during the peak 15 minutes of the peak hour. Because traffic engineers normally record volume counts by 15-min intervals, the procedure is to multiply the peak 15-min volume by 4 to give an equivalent hourly "flow rate." It will be somewhat larger than the peak-hour volume (unless the approach is so congested that it flows full for the whole hour, with no peaking possible). If only an hourly volume is available, the procedure is to divide it by a "peak-hour factor" (PHF), determined at a similar location by applying the equation defining PHF, as follows (*4*):

$$\text{PHF} = \frac{\text{Hourly Volume}}{\text{4 times the peak 15-min vol.}} \qquad (2)$$

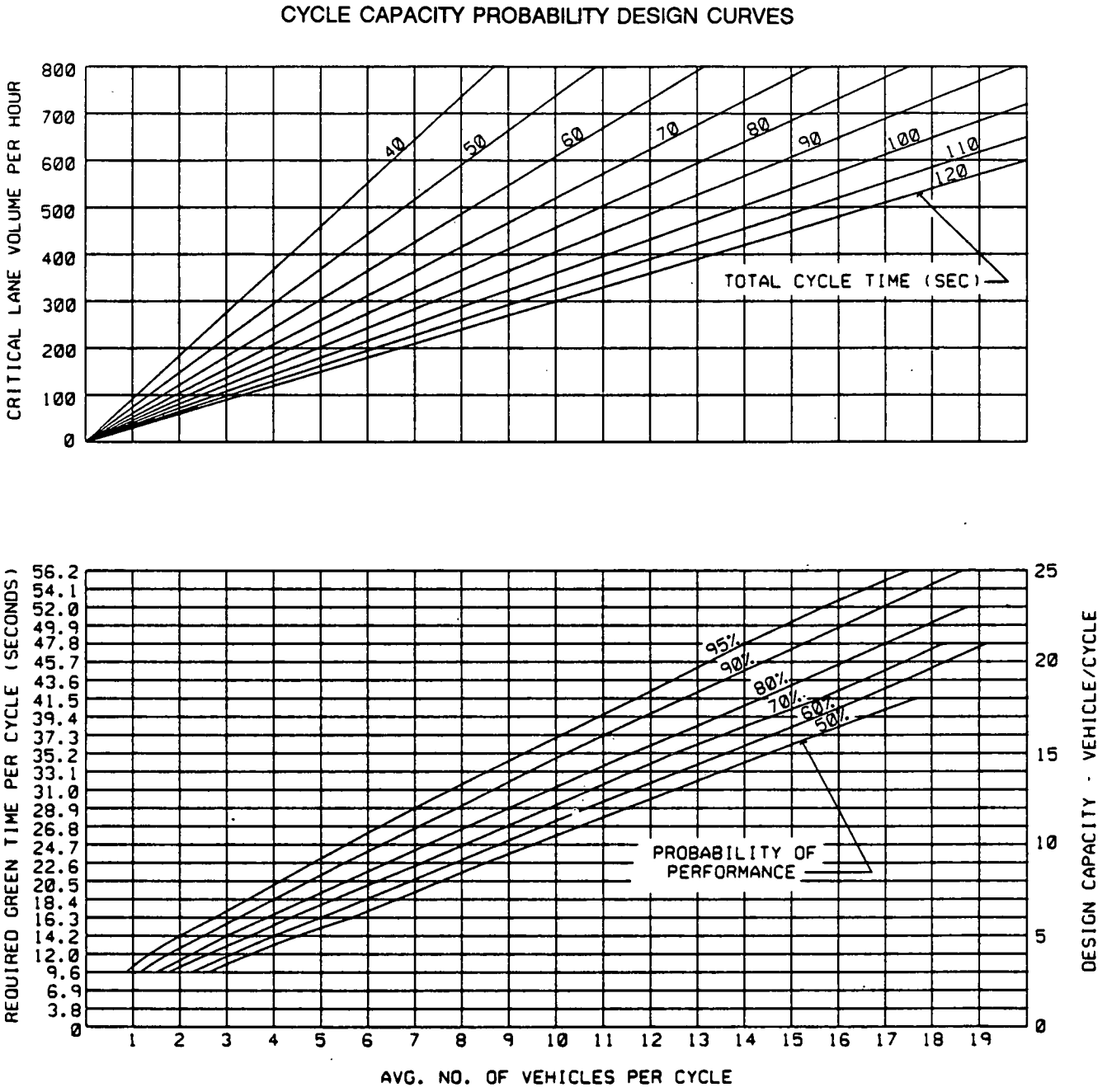
A cycle length designed for the peak 15 min will service the average arrival rate successfully. If arrivals fluctuate about the average, then all the vehicles arriving during the 15 min will also pass during that time, but not necessarily in the cycle in which they arrived. This is "capacity" flow at LOS = E.

An alternative to the procedure just described was proposed by Davidson in 1961 (*15*) and was widely quoted in the 1960s and 1970s (*16, 17*). Hourly volumes unadjusted for PHF can be used with a graph based on the Poisson statistical distribution. (Poisson theory accounts for random fluctuations about the mean of a distribution.) Figure 4, taken from a New York State DOT timing manual (*18*), shows Davidson's family of curves for various probabilities of performance from 50 to 95 percent, derived from the Poisson equation. It can be shown that selecting a 75 percent probability for the entire hour approximates a 50 percent probability for the 15-min peak within the hour, thereby producing results similar to those described in the last paragraph. Davidson used Greenshields' equation to convert the number of arriving vehicles per cycle to the needed green time, shown on the vertical axis of the lower graph in Figure 4. Later in this chapter it will be explained how the New York State DOT uses the 95 percent probability of performance to select the "maximum interval" setting for an actuated controller.

Volumes used to determine green time by the methods of Greenshields or Davidson must be for the "critical lane." For example, consider an east-west street with single-lane approaches. Left-turning volumes are light, and each approach discharges freely. Both approaches are controlled by a single phase of the controller unit. The heavier of the eastbound and westbound volumes is said to be the "critical" lane volume as it controls the green time needed for that phase. In Figure 5a, for example, the 500 exceeds the 200. The critical volumes are summed as 500 + 600 = 1,100 vph.

If an approach has more than one through lane, volume counts are normally recorded for the combination rather than lane by lane. To determine critical lane volumes, it is common to assume that the more heavily traveled lane in a group of two serves 52.5 percent of the total flow, whereas the most heavily used lane in a group of three serves 36.7 percent of the total (*4*). It is the volume of the most heavily used lanes that is used in the calculation of critical lane volumes.

Left turns can complicate the determination of critical lane volumes. In Figure 5b, the 50 vehicles on the east-west street wanting to turn left from westbound to southbound must wait until the oncoming (opposing) through volume of 460 has passed.



1. Enter monograph at upper left with critical lane volume, for one approach or phase.
2. Draw horizontal line to intersect selected cycle line. Make a right-angle turn and draw a vertical line to intersect the probability of performance lines (95 percent probability).
3. Draw a horizontal line to the left and determine allotted green time for the phase. Extend the horizontal line to the right to determine the number of vehicles per phase.

FIGURE 4 New York State Department of Transportation's adaptation of Davidson's curves (*15,18*).

The two movements are sequential, so their volumes are added to give 510 vph as the critical volume for that street. (The fact that the 50 left-turning vehicles could sneak their turn at the end of the yellow is accounted for in the capacity analysis. They are not subtracted from the demand volume.)

On the north-south street in Figure 5b, the 100 vehicles turning left from northbound to westbound are added to the oncoming 100 to give 200. However, this is less than the 400 moving northbound, so the 400 controls. The sum of the critical volumes for this intersection is 510 + 400 = 910 vph.

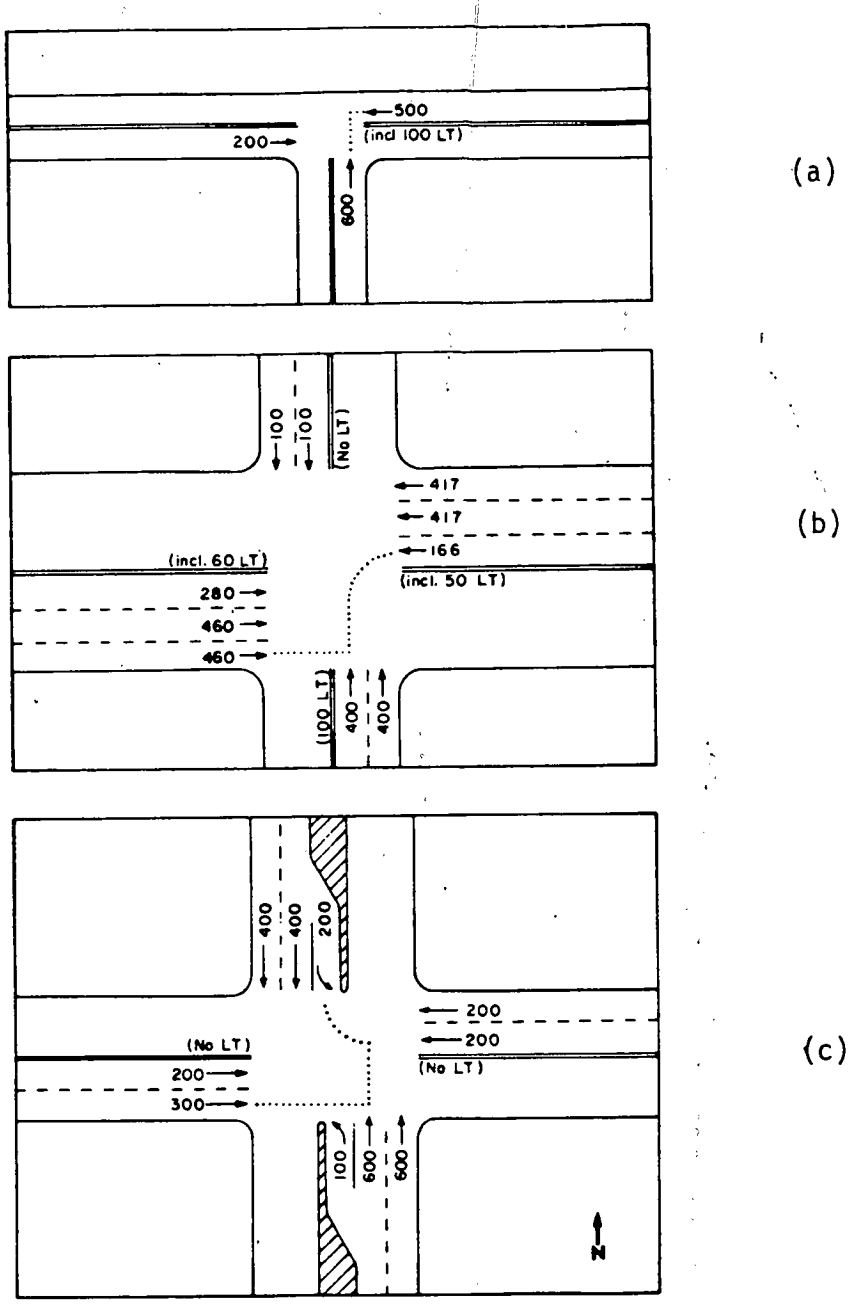


FIGURE 5 Cases for determination of critical-lane volumes (*19*).

Figure 5c shows a common design for a north-south arterial with separate left-turn bays and separate left-turn signals. The arterial has a controller phase for northbound flow, a separate one for southbound, and one for each of the left-turn bays. One phase controls the east-west side street. The cycle begins with dual left-turn arrows. The bay with the lower turning volume (100) gaps out first. The other (200) continues and the southbound movement begins to time concurrently as an overlap. When the 200 left-turning vehicles have passed, the 600 northbound vehicles discharge. The overlap movement may at first seem to complicate the determination of critical volume, but in fact it need not. The lesson of sequential movement taught in Figure 5(b) is applied to (c). That is, the 100 + 400 is compared with the 200 + 600 to give a critical volume of 800. The sum for the intersection is 800 + 300 = 1,100 vph.

The same principles are used with minimum green times, instead of volumes, to calculate a minimum cycle length for the intersection. This is the shortest acceptable cycle length, no matter how low the volumes. If pedestrians use the intersection, the through movements will have minimum greens calculated as described under the next major heading, Pedestrian Intervals. Left-turn phases seldom include concurrent pedestrian move-

ments and can be assigned short minimums. The green times (plus the clearance times) are added just as were the volumes in the explanation of the three cases in Figure 5.

The Transportation Research Board's *Circular No. 212: Interim Materials on Highway Capacity* (*19*, p. 6) uses critical lane volumes to explain a procedure called critical movement analysis. Because the method uses the sum of the critical lane volumes for the intersection, the overall intersection level of service and effects of design on level of service can be determined, and operational changes can be made.

Webster's Equation for Cycle Length

Webster used computer simulation and field observation to develop a cycle-optimization equation intended to minimize delay when arrivals are random (*20*). It is widely quoted and is the basis for the cycle-length computations used by most computer programs. It can be expressed in an easily readable way as follows:

$$C = \frac{1.5L + 5}{1.0 - Y} \tag{3}$$

where

C = optimum cycle length, seconds
Y = critical lane volume divided by the saturation flow (vph), summed over the phases
L = lost time per cycle, seconds

Obviously the equation is very sensitive to small changes in lost time and saturation flow. For moderate traffic volumes the equation tends to yield very short cycle lengths. For heavy volumes, where Y approaches 1.0, clearly the equation will produce very long cycles; the 1985 *Highway Capacity Manual* (*3*) includes an example calculation on pages 9-54 in which the first trial for a multi-phase actuated controller resulted in a calculated cycle length of 633 sec. Some system master controls (masters) use the Webster calculation only as a "pointer" for selection within a range of predetermined acceptable cycle lengths.

Questionnaire Responses on Cycle Length and Split

The respondents reported using all the methods to calculate cycle and split that were introduced above. Manual calculations may be based on Greenshields' headway measurements, Davidson's curves combining Greenshields' data with a Poisson solution, or the Webster equation. A detailed example of a manual calculation from the records of Montgomery County, Maryland, is in Appendix B. SOAP84 is a popular program for this work, and TRANSYT-7F has also become very useful since the distribution of Release 5. Splits are commonly based on critical lane volumes, subject to minimum greens imposed by pedestrian needs or driver expectancy. One consultant in the Southeast uses the following guidelines for driver expectancy:

Type of Phase	Minimum Green in Seconds (not including clearance)
Left turn	5
Minor side st. or driveway	8–10
Typical side street	10–15
Major side street	15–20
Arterial	20 or speed limit/2

Several respondents use the *Highway Capacity Manual* (*3*) or critical lane methods (*19*). These respondents provided no examples, but the cited references provide many for the interested reader.

Queue Buildup on Oversaturated Approaches

A queue may lengthen in a turn lane or on a through approach to the point where special action must be taken. For example, where a lagging left-turn phase is used, during the preceding through movement so many left-turning vehicles may accumulate that the bay overflows into the adjacent through lane, blocking traffic. Turning lanes need to be long enough, if possible, to avoid this. If a turning lane cannot be made long enough, the cycle length should be shortened or other phasing schemes considered.

A through approach may back up so far as to threaten traffic operation at an upstream intersection. "Gridlock" can result, particularly when motorists "block the box" delineating the interior of an intersection. For this reason, the TRANSYT-7F (*2*) program, for example, reports not only maximum queue but also the "maximum back of queue." There is an important difference. Consider an approach that has been discharging on a green for some time. Several vehicles at the front are now in motion and are no longer counted as in a queue. Behind are queued vehicles; new arrivals may be stopping and moving the "back of queue" upstream. The threat to the upstream intersection could be underestimated by considering "number in queue" rather than "back of queue."

A first step in evaluating an oversaturated approach is to observe whether the problem is caused by nothing more than poor split control, that is, by excessive green time on the other critical approach. A field study to discriminate between this condition and a truly congested situation was developed as part of the Urban Traffic Control System (UTCS) research project in Washington, D.C. (*21*). At the time of day under study, observers watch the two critical, conflicting movements for "cycle failure" or "queue failure." If both movements experience a failure of either type in any one cycle, that is a "simultaneous failure." Many such failures during the study period suggest that neither approach is regularly being given too much green. The intersection is truly "congested" and needs split optimization on a cycle-by-cycle basis. During a particular period of the day, one approach may be the critical one for queue control. The cycle length should be kept short enough, and the split tailored, so that this approach operates acceptably.

Excessive buildup of a queue may threaten an upstream intersection; it is easily detected on surface streets (and freeways) by installing an inductive loop 25 to 30 ft long at a strategic location typically several hundred feet upstream of the bottleneck intersection. Several seconds of delay are set on the detector unit. A

stationary vehicle will produce an actuation, which can call an appropriate timing plan or other measure to prevent further buildup.

Most of the engineers who responded to the survey questionnaire mentioned field fine-tuning on a regular basis (such as quarterly) as their method for timing oversaturated intersections, because computer solutions have not been found to be helpful. As a first principle, the fine-tuning attempts to balance or equalize delay to the competing streams. Beyond that, fine-tuning places the queues in the least damaging locations. Queues should not overflow turn bays or create unsafe backups on freeway off-ramps. Upstream intersections should be kept clear for cross traffic.

Oversaturation is widely viewed as being caused by insufficient lanes rather than a deficiency in signal timing. A southeastern consultant recommends to his operating agency client that a solution beyond signal timing is needed at an oversaturated intersection:

> All that can be done with signal timing is to spread the misery around somewhat equally. I use the proportional method for split calculation and the highest possible cycle length (highest being in the range of 130 to 150 seconds). Adjustments to the proportional split might be made to give more time to a phase whose back-up will (1) affect an upstream signal, or (2) create a safety hazard, as in the case of a freeway-ramp back-up.
>
> Solutions such as additional lanes, turn prohibitions, or grade separations might be considered.

Opinions were sharply divided as to cycle-length strategy: Some respondents stress minimizing it; others are of the opposite opinion. Long cycle lengths of 2 to 3 min, or even much more, are used by some respondents to minimize lost time caused by start-up and clearance periods. A southeastern consultant bases the timing of oversaturated intersections on a determination of the tolerable maximum cycle length. During times of day associated with oversaturation, one Southwest city switches the actuated controllers to a longer Max II (the second selectable extended green interval maximum limit) and then locks in detector inputs to prevent a straggler from causing a premature gap-out.

The state of Delaware uses very long cycle lengths to cope with extreme oversaturation. A 6-min cycle is split to give 5 min to the arterial street and 1 min to the side street. The timing for each location, however, is tailored to the circumstances. "In general, equal queues are a first consideration, but other factors such as emergency-vehicle routes and the relative classification of roadways must be considered," says the respondent.

Other respondents emphasize minimizing cycle lengths or at least holding them to no more than 120 to 130 sec. Excessively long cycles will overflow left-turn bays or back queues into upstream intersections or driveways. Drivers on a side street appear to expect to wait no longer than about 90 sec, it was reported, after which they may conclude that the signal is "stuck" and run the red. The Texas State Department of Transportation (TxDOT) attempts to time phases for no longer than 40 sec, because it has found that there is a "lowering of saturation flow at that point." The respondent from a Southeast urban county reports that gaps begin to widen around 60 sec. Caltrans has found that, in general, shortening maximum intervals tends to reduce headway and increase saturation flow, thereby relieving oversaturation; the Caltrans respondent has observed that shorter cycles cause motorists to perceive less delay, although that may not actually be the case.

One resource, titled *Traffic Signal Control for Saturated Conditions* (*22*), prepared in late 1988 by KLD Associates in conjunction with the Texas Transportation Institute (KLD/TTI), offers many practical suggestions that can help the practicing traffic engineer. With regard to the issue of cycle length at oversaturated intersections the report states (*22*):

> The conventional wisdom is still that a reduction in cycle length will result in a reduction in capacity. The research team concurs with an alternative view, which is that avoiding spillback and blockage is the real means of realizing capacity and throughput improvements.

Oversaturation in coordinated systems is addressed in Chapter Three, under the heading Timing for Systems with Oversaturated Intersections.

PEDESTRIAN INTERVALS

The purpose of this section is to discuss the traffic-flow principles influencing signal timing for pedestrians. A related reference is the Transportation Research Board's *Synthesis of Highway Practice 139: Pedestrians and Traffic-Control Measures* (*23*, pp. 39-41).

The MUTCD (*1*) states that "pedestrians should be assured of sufficient time to cross the roadway at a signalized intersection." "Time to cross" means time to react to the signal change, step off the curb, and walk to a location that will be safe when a green is given to conflicting vehicles.

The following discussion pertains to locations where pedestrians cross the street concurrently with traffic on the parallel street.

Case without Pedestrian Signals

When there are no WALK/DON'T WALK signals, pedestrians cross the street during the green, yellow, and any all-red given to the street parallel to their path. These intervals should combine to provide time equal to that obtained for the case where pedestrian signals are used. (That case is explained in the next section.) The first 4 to 7 sec of green are assumed to be needed for pedestrian start-off. The remainder of the green, plus the yellow and any all-red, should be sufficient for the pedestrian to walk from the near curb to the center of the farthest traveled lane at the assumed walking speed.

Case with Pedestrian Signals

If pedestrian signals are used to provide WALK and DON'T WALK messages, then "under normal conditions the WALK interval should be at least 4 to 7 seconds in length so that pedestrians will have adequate opportunity to leave the curb before their clearance interval is shown" (*1*). This section of the MUTCD goes on to describe several situations in which the lower values in this range may be appropriate. The *Traffic Control Devices Handbook* (TCDH) (*24*) states that "a WALK interval of 4 seconds is adequate when fewer than 10 pedestrians per

cycle are expected." The reason that as much as 4 or 5 sec is needed is that many pedestrians waiting at the curb watch the traffic, not the signals. When they see conflicting traffic coming to a stop, they will then look at the signal to check that it has changed in their favor. If they are waiting at a "right-hand curb," they will often take time to glance to their left rear to see if an entering vehicle is about to make a right turn across their path. A pedestrian reasonably close to the curb and alert to a normal degree can be observed to require up to 4 or 5 sec for this reaction, timed from when the signal changes to indicate that it is safe to cross to stepping off the curb.

The MUTCD calls for a flashing DON'T WALK interval, after the WALK, to provide pedestrian clearance (*1*):

> The duration should be sufficient to allow a pedestrian crossing in the crosswalk to leave the curb and travel to the center of the farthest traveled lane before opposing vehicles receive a green indication. (Normal walking speed is assumed to be 4 feet per second.)

At the center of the farthest traveled lane the pedestrian should be readily visible to the driver of the first vehicle to start up in that lane. Although the MUTCD allows the DON'T WALK to be flashed until the instant opposing traffic is released, some engineers prefer that the yellow (and any all-red) be accompanied by a steady DON'T WALK; the change from flashing to steady DON'T WALK seems to be well understood as a last warning to get out of the street.

The engineer needs to be alert to situations in which the pedestrian clearance period should be designed using a walking speed of less than 4 ft/sec. The TCDH (*24*) explains:

> . . . research verifies that one-third of all pedestrians cross streets at a rate slower than 4 fps and 15 percent walk at or below 3.5 fps. Those having slower walking speeds have the moral and legal right to complete their crossing once they have lawfully entered the crossing. Vehicular traffic is to yield the right of way to pedestrians lawfully within the intersection.
>
> This suggests that the timing of pedestrian signal indications near facilities that serve segments of the population with slower walking speeds should be calculated based on a slower walking speed. Such populations should be anticipated near shopping centers, convalescent or rest homes, therapy centers, etc.
>
> Some traffic engineers tend to resist using the slower walking rate because it may result in less-favorable signal splits and longer cycle lengths resulting in longer vehicular delays. Engineering studies and judgment should be exercised for each problem intersection to obtain the optimum balance between pedestrian and vehicular traffic.

Responses to the Questionnaire

When pedestrian signals are present, timing begins with a WALK interval specified by the MUTCD to be normally at least 4 to 7 sec. Several respondents use 7 sec as a standard but choose 4 where phase timing must be minimized and pedestrians do not need more time. On the other hand, a school crossing may have as much as 15 sec of WALK. The Florida DOT and the city of Durham, Ontario, provide sufficient WALK time for the pedestrian to reach the middle of the street, so that the pedestrian will not turn around when the flashing DON'T WALK begins. Florida times the WALK as (D/4)/2 where D is the street width, and Durham adds 1 sec to this, presumably as a pedestrian start-off period. The state of Delaware also has found that "pedestrians do not react well to the short WALK, long flashing DON'T WALK timing pattern. They equate the flashing with a vehicle yellow period. We are tending to move away from it." A Southeast consultant agrees: "For pretimed controllers, I would time WALK as the phase split minus vehicular clearances and flashing DON'T WALK; i.e., provide as much of the phase time as possible as green/WALK."

Most respondents calculate the flashing DON'T WALK interval using a walking speed of 4 ft/sec and a distance from the near curb to the center of the last traffic lane or edge of parking lane. However, many are sensitive to the need to reduce walking speed to 3.5 or even 3 ft/sec if schoolchildren or senior citizens are in the majority at a specific crosswalk. Because pedestrians can walk safely during the yellow and all-red, it is common to subtract these two intervals from the calculated walking time; the flashing DON'T WALK changes to steady DON'T WALK during the yellow and all-red, giving pedestrians several seconds of final warning to get out of the street before cross traffic is released.

There are some situations at an intersection with a pretimed signal where there are no pedestrian signals, but pedestrians occasionally desire to cross at a certain phase. It is possible, depending on the specific conditions present, to time a green phase equal to the WALK and flashing DON'T WALK calculated as just described. It is also possible to time an actuated controller that has received a pedestrian call with a green of that length.

Pedestrian Safety versus Arterial Capacity

Traffic engineers in some circumstances can find it difficult to time for pedestrians. On one hand is their duty to consider the safety of pedestrians, including those who walk at slower-than-normal speeds. The pedestrian who starts during the green or WALK should be able to complete the crossing. This means that the occasional slow walker should not be at risk but should, however, be encouraged to wait for the beginning of the next green or WALK signal. On the other is their responsibility to operate busy arterials to their full capacity, minimizing stops and delay. The two goals are potentially in conflict if every side street green must be timed long enough to accommodate pedestrians, whether or not they are in fact present. The obvious solution is to provide push buttons and WALK/DON'T WALK signals, expect pedestrians to use the buttons and obey the signals, and provide adequate time for crossing only if the button has been pushed. The difficulty here is that some pedestrians seem to believe that it is acceptable to cross on the green, along with moving traffic. They believe it is not necessary to press the button if waiting vehicles bring the green. Also, they seem to assume that if the pedestrian signal still shows DON'T WALK, then it needs to be fixed.

The traffic engineer should be in touch with the expectancies of road users and should act on this knowledge. If pedestrians (and perhaps also children on bicycles) are known to be crossing arterials on the DON'T WALK, the engineer should recognize this as hazardous. If the hazard cannot easily be removed, there is a need to warn of the hazard through public-information programs and the use of appropriate signs at the crossings such as the standard CROSS ON WALK SIGNAL ONLY regulatory sign. Stringent police enforcement should also be requested. An-

other approach is to construct a pedestrian refuge island with a push button in the middle of the arterial. This, however, may lead to issues related to traffic safety and winter maintenance.

If the traffic engineer is aware of such a hazard and is not certain how to deal with it effectively, the agency's legal counsel should be consulted because in such cases the use of engineering judgement may have implications for the legal liability of the agency and the engineer. Consideration of this aspect is important in management of the risk of tort liability.

Faced with the necessity to accommodate both the need for making efficient use of arterial street or highway capacity and the need to assure pedestrian safety, traffic engineers cannot be, and are not held to be guarantors of pedestrian safety under all circumstances, including those where pedestrians knowingly fail to comply with traffic control devices, pavement markings or other instructions furnished for their safety. Pedestrians who flout the law contribute to the risk of their own injury and may, under some circumstances, assume that entire risk.

At the same time, courts reserve the right to decide case-by-case whether the actions of the traffic engineer and transportation agency are reasonable under the circumstances. The duty of care required by the law may not always be met by following governmental or industry standards (such as the MUTCD) where circumstances indicate that the transportation agency and its engineers should have recognized the presence of a safety risk and did not act to remove or reduce it, even though they could have done so with reasonable effort.

Therefore, when consulting legal counsel regarding the trade-offs or compromises that entered into agency decisions, the traffic engineer should document these decisions fully with the factual record that was considered, the interpretation given to these facts, and the reasons for deciding to take the action in question. With such a record to explain the engineering determination, management of the risk of tort claims and tort liability will be put on its strongest basis.

PHASE-CHANGE INTERVALS

During the preparation of the second edition (1983) of the Institute of Transportation Engineers' (ITE) *Transportation and Traffic Engineering Handbook* (*13*), no subject was as controversial as the timing of the yellow and all-red intervals. The profession had reached a consensus that the calculation of the yellow should use a comfortable and attainable deceleration rate of 10 ft/sec/sec rather than the "emergency" rate of 15 that had been standard for many years. This change added about 1 sec to the calculated yellow. Some engineers believed that the entire second would be time lost to traffic movement and would seriously reduce intersection capacity and efficiency.

Yellow Interval

At least half the states in the United States follow the "permissive yellow rule" found in the MUTCD (*1*) and the current version of the *Uniform Vehicle Code* (UVC) (*25*). It allows vehicles to enter the intersection on a yellow signal and to be in the intersection when the signal turns red. Approximately 50 years ago the UVC required more restrictive yellow rules that were virtually impossible for the driver to obey or for the police to enforce. Although some state laws still follow one of the more restrictive rules, the scope of this synthesis is limited to the permissive rule. An FHWA report titled "A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals" (*26*) names the states that were following the various rules as of 1980 and discusses the rules in more detail.

The MUTCD (*1*) states, "Yellow vehicle change intervals should have a range of approximately 3 to 6 seconds. Generally the longer intervals are appropriate to higher approach speeds."

But in *Fraley* v. *City of Flint*, 221 N.W.2d 394 (Mich. 1974), a Michigan court held that it is not enough that a yellow time merely be between 3 and 6 sec (*27*). It must be designed for intersection-specific conditions, such as truck usage. The TCDH (*24*) advises:

> Since excessively long yellow intervals may encourage driver disrespect, a maximum of about 5 seconds is usually used for the yellow interval if a long yellow interval is required. If a longer phase-change interval is needed then the additional time is provided by an all-red interval.

The timing of the yellow interval is determined manually, following a well-accepted formula (*24, 27*). It is:

$$Y = t + \frac{v}{2a \pm 64.4g} \tag{4}$$

where

- Y = yellow interval, sec
- t = driver perception-reaction time for stopping, taken as 1 sec
- v = approach speed, fps, taken as the 85th percentile speed or the speed limit
- g = percent of grade divided by 100 (positive for upgrade, negative for downgrade)
- a = deceleration rate for stopping taken as 10 ft/sec/sec so as to be attainable comfortably on a wet road by both cars and trucks

This equation gives a yellow interval long enough so that a clearing driver will not be forced to enter the intersection on the red, an unlawful act. The fact that the vehicle is clearing can be confusing, because the equation includes two terms associated with stopping. The derivation starts with the equation for stopping distance s in feet, as follows:

$$s = v_o t + v_o^2/2a \tag{5}$$

where the $v_o t$ gives the distance traveled at initial speed v_o during braking perception-reaction time t, and the $v_o^2/2a$ is the braking distance to a final speed v of zero, proceeding from the fundamental equation of linear kinematics:

$$v^2 = v_o^2 + 2as \tag{6}$$

where a is negative (a deceleration).

Now, if the yellow begins when a vehicle is farther away from the intersection than the stopping distance, we can expect the driver to stop. But if the vehicle has barely the calculated stopping distance needed (or has less), it is reasonable for the driver to decide to clear rather than stop. The minimum required yellow

time will carry the clearing vehicle just into the intersection; it has legally entered, just before the red begins. The time required for this is calculated by dividing equation (5) by v_o, yielding equation (4) as yet uncorrected for approach grade.

When the yellow begins, the driver need only decide whether it is possible to stop comfortably, without violating the stop bar or a pedestrian crosswalk. There is no other decision involved, such as whether the entire intersection can be cleared. Therefore, the driver need not predict the length of the yellow. It follows that the yellow need not be standardized to meet the traffic engineer's estimate of what a driver might expect. It simply needs to be calculated using equation (4).

Equation (4) may yield long yellows, more than 5 sec, on high-speed downgrades. The remedy is to use 5 sec and add the excess to the all-red interval. The engineer may then calculate the speed that gives 5 sec and post it as an advisory speed plate below a suitable warning sign, such as the hill sign or signal-ahead sign.

A yellow time calculated using equation (4) carries the clearing vehicle just into the intersection by the time it ends, as shown by vehicle A in Figure 6a. (In this figure, the "intersection" is taken to begin after the stop bar, where conflict with pedestrians could start.) If there is no all-way red interval, then vehicle B and pedestrian C are now released on a green signal. Vehicle A could strike vehicle B or pedestrian C as shown in Figure 6b.

The driver of vehicle B has a duty to yield the right-of-way to vehicle A lawfully within the intersection. However, unpublished research (*28*) showed that 60 percent of 239 drivers interviewed did not know that this is the law. Also, 60 percent did not say anything about looking when asked the question "Suppose you are stopped for a red light and you are first in line. What do you do when you see the signal go green?" Another question asked was "What would you think if traffic engineers decided to time yellow lights so that there may be a vehicle going through the intersection when you get your green?" Of the 239 drivers, 69 percent said that they disapproved because it sounded dangerous.

The interviews were supplemented by field observations of the looking behavior of 795 drivers who were first in line when the signal turned green. Studies were done at two intersections so wide that a driver could check for conflicting traffic only by turning his or her head. A different observer studied each location. The results were similar to those from the interviews; it was found that 64 percent did not look before entering the intersection. The results were similar for each observer and each intersection.

These interviews and field studies suggest that it would be naive for a traffic engineer to expect the driver of vehicle B to meet the legal duty to yield the right-of-way to vehicle A in Figure 6a. Pedestrian C does not even have such a duty, and is free to step out into the street without yielding. Most traffic engineers could be expected to realize that the scenario of Figure 6a and b is potentially hazardous. One solution is to provide an all-way red clearance interval, discussed next.

All-Red Interval

Interviews and field observations show that, in order to time phase-change intervals for safety, traffic engineers sometimes need to go beyond the minimums implied by the rules of the road. An all-red clearance interval should be considered in some cases in addition to the yellow.

The MUTCD (*1*) states, "The yellow vehicle change interval may be followed by a red clearance interval, of sufficient duration to permit traffic to clear the intersection before conflicting traffic movements are released." There is, however, a lack of consensus at this time on whether this means that the clearance interval should be sufficiently long to completely clear the intersection and the degree to which the concept should be applied systemwide. The TCDH (*24*) specifically recognizes the various points of view: "The policy of some jurisdictions is to time the phase change interval to allow the outset of the green interval for conflicting movements without the intersection having been cleared." In such cases, the TCDH states that equation (4) may be used.

However, the TCDH also states the following:

> Some authorities believe that the timing of a phase-change interval should enable a vehicle to clear the intersection before the onset of the green for conflicting movements. The following equation may be used to determine the phase change interval. It includes a reaction time, deceleration element, and an intersection clearing time.

$$CP = t + \frac{v}{2a \pm 64.4g} + (W + L)/v \qquad (7)$$

where CP is the nondilemma change period (yellow plus all-red), in seconds; v is the approach speed; and t, a, and g are as defined above for equation (4).

The TCDH goes on to suggest that

> . . . the yellow change interval be equal to the first two terms of the equation rounded up to the next 1/2 second, but no less than 3 seconds and no greater than 5 seconds. The remainder of the change period should consist of an all-red interval.

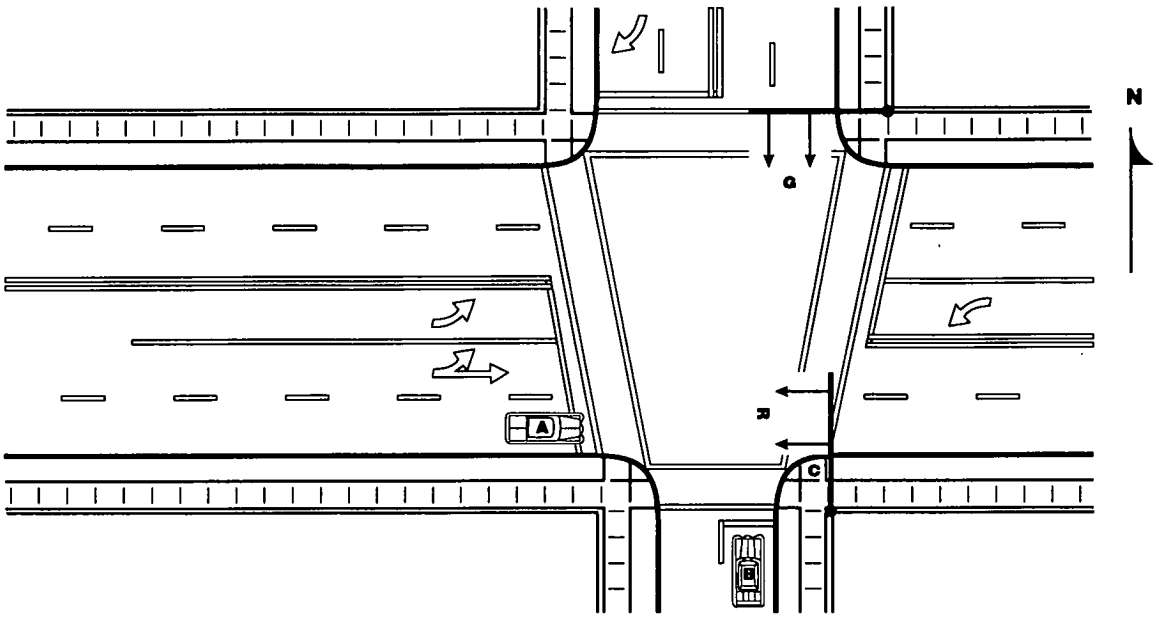
The last term of equation (7) is the suggested all-red intersection clearing time, where:

> W is the width of the intersection in feet, measured from the upstream stop bar to the downstream extended edge of pavement
> L is the length of the clearing vehicle, normally 20 ft, and
> v is the approach speed in fps and should take into account slow-moving trucks if they are significant at the location.

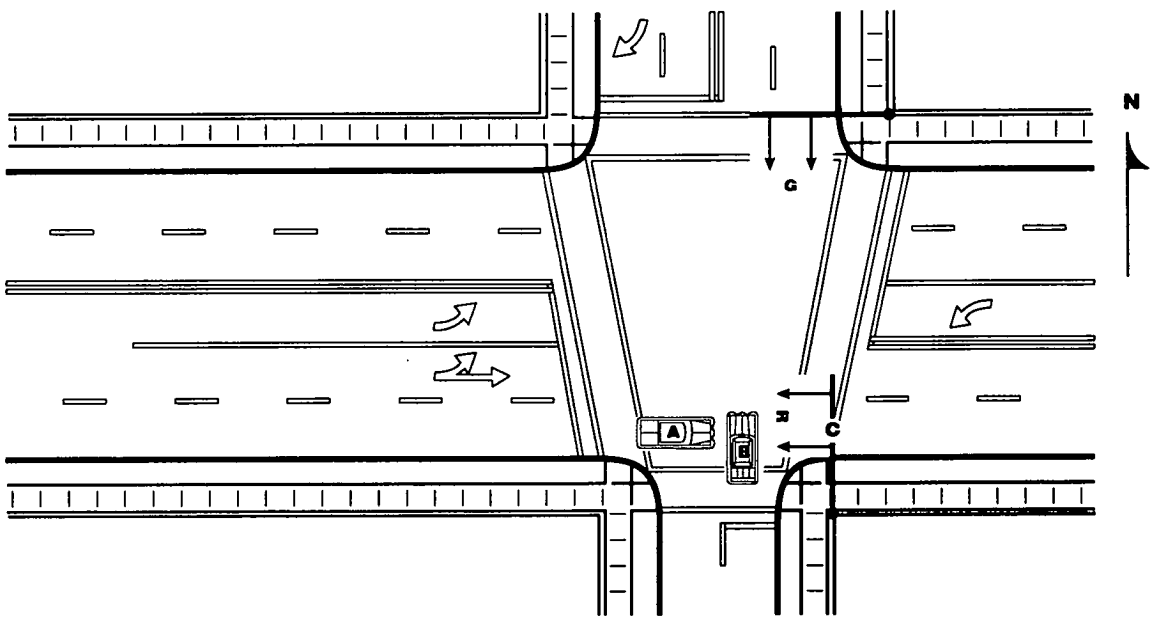
When the first two terms of equation (7) comprise the yellow, and the third term is the all-red clearance interval, the scenario becomes as shown in Figure 7, which shows the position of clearing vehicle A at the time vehicle B and pedestrian C are released on a green. This scenario is probably safer than the one of Figure 6.

There is a general feeling among traffic engineers that driver respect for the onset of the red signal is deteriorating. Other than the obvious remedy of stricter enforcement by law-enforcement agencies, traffic engineers do not appear to have a solution for this and are unsure how to deal with the problem.

One approach to the problem has been stated as (*27*):

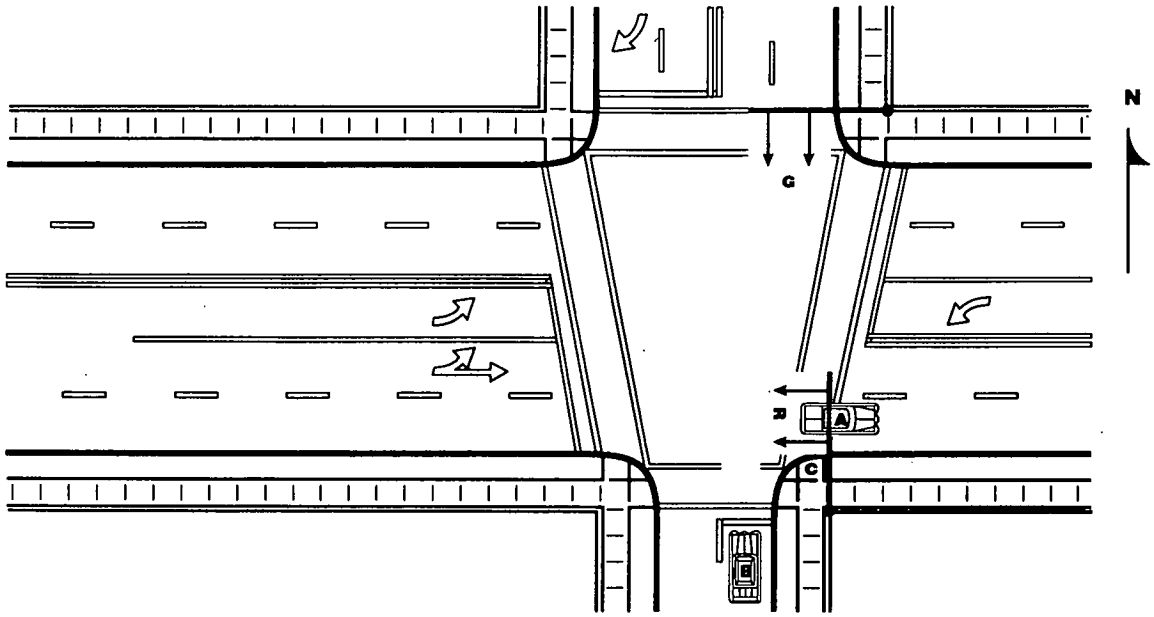


a Eastbound car is clearing after having barely entered the intersection by the time the red begins. There is no all-red, so northbound car receives the green immediately.



b Northbound car fails to yield right-of-way to car legally in the intersection, enters soon after receiving the green, and is struck.

FIGURE 6 Possible scenario with no all-red clearance.



Eastbound car clears intersection by the time northbound car receives green.

FIGURE 7 Scenario when an all-red clearance is used.

There is an excellent analogy here with the design of signal timing for pedestrians. Once the pedestrian enters the crosswalk legally on the green or "Walk" indications, he or she has the right of way, even after cross-traffic is released. Our design standards, however, do not rely on such restraint by cross-street motorists. We prudently include a pedestrian-clearance interval in our signal timing. We should give clearing vehicles the same level of protection and safety that we afford pedestrians.

Responses to the Questionnaire

Regarding the yellow change interval and the all-red clearance interval, the New York State DOT has a published standard (*18*) based on the previously mentioned FHWA report (*26*). The NYSDOT standard calls for the yellow change interval to be timed using equation (4). Perception-reaction time t is taken as 1 sec and the deceleration rate a is taken to be 10 ft/sec/sec. On a level grade (g = 0) the equation yields the following:

Approach Speed		Yellow Time
mph	(fps)	sec:
30	(44)	3.2
35	(51)	3.6
40	(59)	3.9
45	(66)	4.3
50	(74)	4.7
55	(81)	5.0
60	(88)	5.4

The New York State DOT (*18*) adds this note:

Except where steep downgrades dictate the use of long yellow intervals, yellow intervals longer than 5 seconds should not be used. At approach speeds greater than 55 mph, the excess yellow time above 5.0 seconds should be included in the All Red setting. Interval timing on steep downgrades can be reduced by posting appropriate warning signs.

New York State's policy on the all-red clearance interval is also taken from the FHWA report (*26*) and is stated as follows (*18*):

a. Intersections with a right-angle accident rate greater than 0.8 right-angle accidents per million entering vehicles should be considered for addition of an all-red interval.

b. The limiting length of the all-red interval should be that implied by the *Transportation and Traffic Engineering Handbook* (*13*):

$$\frac{W + L}{V} \tag{8}$$

where
W = Width of the cross street or last collision point
L = Vehicle length (L = 20 ft)
V = Appraoch speed (in ft/sec)

Item a. of the NYSDOT all-red policy, above, is based on the rate of right-angle accidents. In 1985 the ITE published a proposed recommended practice for "Determining Vehicle Change Intervals" (*29*). Like the NYSDOT policy, the ITE proposal also mentions guidelines based on rates of right-angle accidents.

The only respondent that clearly appears to be using an accident guideline is the city of Tulsa, which adds 1 to 2 sec of all-red to those two-phase intersections that have "accident problems." At multi-phase intersections, Tulsa always adds 1 sec of all-red to through movements, and 1 to 2 sec to left-turn movements at wide (150-ft to 180-ft) intersections.

Two southeastern consultants who have experience with signal timing use equation (4) to time the yellow and time the all-red using equation (8), except that the "yellows are forced into the range of 3 to 5 seconds," said one of them. The other added, "Left-turn phases receive 4 seconds of yellow and no all-red unless unusual geometrics or inadequate sight distance require special clearance intervals."

The Iowa DOT uses a yellow interval calculated as the sum of equations (4) and (8). The Iowa respondent made no mention of all-red intervals. A consultant from the Southeast also uses the sum of equations (4) and (8). Rather than use equation (4) for yellow and (8) for all-red, he prefers to standardize on just three yellow times based on speed: Approach speeds up to 25 mph receive the minimum 3 sec, those from 25 to 40 mph receive 4, and speeds greater than 40 mph are timed with 5 sec of yellow. He then subtracts the yellow from the sum of equations (4) and (8) to obtain the all-red interval.

The North Carolina DOT also sums equations (4) and (8) and specifies that the yellow and all-red together not be less than that sum. The yellow must not be less than equation (4), nor less than 3 sec, nor greater than 5 sec, unless that is necessary to satisfy the equation. The North Carolina DOT has prepared convenient tables of the sum of equations (4) and (8) for downgrades varying from 0 to 8 percent. The use of the all-red is optional, as noted in comments on the questionnaire, for example: "An all-red interval should be considered whenever the total clearance interval required exceeds 5 seconds."

Montgomery County, Maryland, uses a "rule of thumb" based on speed, as follows: 3 sec for speeds up to 35 mph, 4 sec for 35 to 45 mph, and 5 sec for greater than 45 mph. Currently a 1-sec all-red is used routinely after a left-turn phase "due to the increased incidence of motorists running the start of red." Montgomery County generally (but not always) uses 1 sec of all-red for all other phases "except special cases—longer for unusually wide intersections, etc."

The state of Delaware and a New Jersey consulting firm time the yellow as the posted speed limit divided by 10 (with a minimum of 3 sec), a procedure seen to satisfy equation (4) provided the approach is level. On occasion, Delaware makes an exception on high truck-volume routes. Delaware uses all-red "universally except in CBD and low-speed urban areas. The truck problem is generally resolved using this interval. As was expected, the yellow is typically "green' and the all-red functions as the old yellow."

Toronto has standardized the yellow and all-red times based on intersection geometry. Four-legged intersections are timed with 4 sec of yellow and 2 sec of all-red. At a T-intersection, 1 sec is taken from the yellow and added to the all-red. "In special cases different values are used, mainly for safety reasons, e.g. greater approach speeds."

The city of Lakewood, Colorado, uses yellow times, for both through and left-turn movements, taken from the *Transportation and Traffic Engineering Handbook* (*13*), rounded up to the nearest 0.5 sec. The result is as follows:

Approach Speed		Yellow Time
mph	(fps)	sec
20	(29)	3.0
25	(37)	3.0
30	(44)	3.5
35	(51)	4.0
40	(59)	4.0
45	(66)	4.5
50	(74)	5.0
55	(81)	5.5

Lakewood uses the 85th percentile speed to enter this table. If a speed study is not available, the speed limit plus 5 mph is used. Where controller operation allows, Lakewood uses a 1-sec all-red clearance interval for left-turn movements and an all-red for through movements taken from the following table:

Approach Design Speed	Width[a] 40 Feet	80 Feet	120 Feet
40 or less	1.5	2.0	2.0
45	1.0	2.0	2.0
50 or 55	1.0	1.5	2.0

[a] The width is the distance from the stop line to the edge of the far travel lane. Vehicle length is 20 ft.

SETTINGS OF ACTUATED CONTROLLERS

The general practice of the TxDOT is to "set Minimum Intervals and Passage Times according to detector placement. Set the Maximum Intervals for best pretimed operation during the peak. Put major phases on recall."

The relationship between controller settings and detector placement depends on so many factors that a separate Appendix B is included on this topic. Also, there is more information to be found under the later heading "Choice of Actuated Controller/ Detector Configuration." When a small-area (6 ft x 6 ft) loop is used upstream from the stop line, respondents are calculating the minimum interval using the Greenshields equation and the number of vehicles that could be stored between the stop line and the detector. When a large-area (e.g., 6 ft x 20 ft) loop is used, as in Florida, a minimum-interval setting of 5 sec was suggested by one respondent for left-turn phases, and 10 sec for through movements, subject to adjustment for pedestrians and driver expectancy. A passage time or extension of 3 sec has been found suitable for the 6 ft x 20 ft loops, which are usually of Quadrupole design and are located at or slightly downstream of the stopbar.

Maximum Intervals

One respondent sets the maximum interval according to the following recommendation by Kell and Fullerton (*30*, p. 153):

> . . . Compute optimum cycle length and green times in the same way these intervals are determined for pretimed controllers. The computed green intervals are then multiplied by a factor ranging between 1.25 and 1.50 to obtain the maximum green.

Maximum intervals of so great a length accommodate most cycle-to-cycle peaks. Some agencies resist long maximum intervals for various reasons: Failed loops or detector units may lock in calls, producing excessive timing and wasted green; also, vehicle headways tend to lengthen as the green extends.

New York State DOT does not directly use the factors 1.25 to 1.50, but arrives at a similar result by designing green intervals and cycle lengths for a 95 percent "success rate," or "probability of performance or clearance" (*18*). (This concept was developed earlier in this chapter.) Figure 4, adapted by the NYSDOT from Davidson's work (*15*), can be entered to estimate the green time needed to pass all of the vehicles that arrive during the cycle, 95 percent of the time, assuming peaks and surges that follow the Poisson statistical distribution. Because the cycle length must first be estimated, the procedure is iterative; that is, in the last trial the green times obtained from the lower figure must add (when combined with their yellows) to the cycle length entered in the upper figure in that trial. (These details are made clear in the examples in Appendix C.) The NYSDOT (*18*) provides a method for an initial estimate of cycle lengths to give 95 percent success:

Sum of Critical Lane Volumes	Minimum Cycle Length in Seconds		
	2 Phase	3 Phase	4 Phase
600	40	50	60
700	50	60	80
800	60	80	100
900	70	100	120
1000	90	120	—
1100	120	—	—

The NYSDOT suggests that a phase set to maximum vehicle recall be timed using a maximum interval obtained by dividing the green time for 95 percent success by 1.5. "This value will give a good average green . . . ." Dividing by 1.5 results in a green long enough to pass average arrivals, corresponding to 50 percent success.

Density Timing

The timing of "density" ("volume density") features, whether on a National Electrical Manufacturers Association (NEMA) standard or Type 170 controller, has been a source of confusion for decades and is one of the areas in which users have indicated a need for more assistance than they have found in the literature.

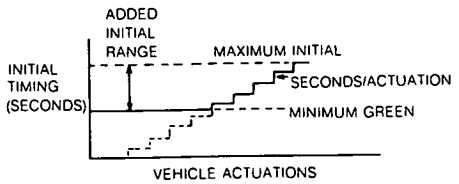
Density control is used on major approaches, especially those with high speeds. Several respondents mentioned the use of detector layouts and set-backs designed to solve the problem of the "zone of difficult decision," or "option zone" (*31*). [The literature, and the respondents, often refer to this situation as a "dilemma-zone" problem. However, the term dilemma-zone is best reserved to describe a situation associated only with a short yellow interval; an approaching driver located within the dilemma zone when the signal changes to yellow can neither stop nor clear without (illegally) entering the intersection on the red signal (*32*).] The option-zone problem exists even when the yellow is long enough to avoid the classic "dilemma-zone" problem. Approach speeds are high enough (40 mph or greater) that a driver can be indecisive if the yellow begins when the vehicle is within the range of distance from the intersection that may best be termed an option zone. An abrupt stop may produce a rear-end collision, whereas an initial decision to stop, followed by a decision to clear instead, may produce entry on the red and a right-angle collision. The respondents use long set-backs to detect the vehicle before it reaches the option zone and to attempt to hold the signal green until it has cleared the zone. When the yellow begins, the driver now has no difficulty in deciding to clear. Appendix B includes further information on the option zone.

The initial interval of a density phase is variable from cycle to cycle. It can range from a minimum green time, usually called simply the initial, to a maximum variable initial. The purpose of this variable interval is to provide enough green time to clear the queue of vehicles that has formed between the stop line and the detector during the previous red. (The detector may be several hundred feet upstream of the stopline. Vehicles queued upstream of the detector are not of interest here, because after the green begins they will have an opportunity to cross the detector and extend the green beyond the initial interval.)

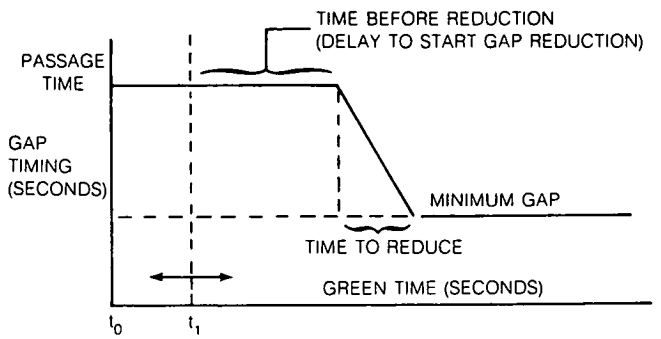
Initial (minimum green) timing is calculated by applying the Greenshields equation to a minimal queue associated with off-peak conditions and taking into account pedestrian minimums and driver expectancy. The NYSDOT recommends 8 to 15 sec for this.

The variable initial time period increases from zero in increments that depend on the seconds per actuation that has been set. As a first principle, this will be 2.0 to 2.5 sec/vehicle. If the controller phase is associated with just one approach, say eastbound, and the approach has only one lane, then this principle holds. However, if the approach has three lanes, then nine actuations produce a queue of only three vehicles, so the 2.0 to 2.5 would be too high. If the controller phase is associated with both eastbound and westbound approaches, then actuations from both directions are "feeding" the count maintained by the variable initial feature; the user now must reduce the 2.0 to 2.5 by factors that consider the directional distribution of traffic at various times of day. If this seems complicated, at least it is better than the "good old days," when it was necessary to compensate also for the fact that some detectors gave one actuation per vehicle, whereas others gave two (to mimic the operation of a pressure-sensitive treadle). Figure 8a shows the variable initial incrementing as vehicles cross the detector on the red; it does not govern until it exceeds the initial (minimum green) set on the controller phase. The figure shows a maximum variable initial setting. The reader might ask why one is necessary, because actuations will automatically end when the queue stretches back to the detector. The reason is that a loop-detector unit can fail in a pulsing mode; it could "pump" several minutes worth of variable initial time into the controller phase. Some controllers are set internally with a 30-sec maximum, whereas others allow the user to set it as desired up to 60 sec.

The other "density" feature is the reduction of allowable gap based on time waiting on the red. An explanation begins with the need to locate the detector approximately 5 sec of travel time (e.g., 400 ft) from the intersection to detect the vehicle before it enters the option zone, explained earlier. Figure 8b shows that the allowable gap timing begins at the value set for the passage time. That should be the travel time from the detector to the



(a) Variable initial interval



(a) t_0 = Start of phase GREEN.
(b) t_1 = Registration of serviceable conflicting call.
(c) TIME BEFORE REDUCTION shall not start timing before t_0.
(d) MAXIMUM timer start shall be conditional upon being in the GREEN interval and registration of a serviceable conflicting call--vehicle or pedestrian.
(e) PASSAGE TIME portion of GREEN interval *must* time concurrently with INITIAL subject to vehicle actuations.

(b) Time-waiting gap reduction

FIGURE 8 Variable initial interval and gap reduction (*33*).

stop line, 5 sec. When a waiting call is registered, the controller times a user-set time before reduction. This interval has the purpose of preventing a premature gap-out caused by too-early reduction of gap. The NYSDOT states that "this interval should be set in a range of 10 to 20 seconds to dissolve the queue at the stop line and have the vehicles arriving with a uniform headway. This value should never be less than the Initial setting."

The allowable gap now decreases over a time to reduce until it reaches the minimum gap, both user-set. The objective is to force moving traffic to meet increasingly stringent requirements for maintaining efficient headways as green time elapses; if a "gap-out" does not occur and the green "maxes out," then there is no protection against the "option zone." A minimum gap set lower than 2.5 sec will also tend to defeat option-zone protection, as explained in Appendix B. The NYSDOT seems to suggest 15 sec for time to reduce; with the maximum interval set high at 60 to 99 sec, the time to reduce must be set in the field to be short enough to prevent "max-outs." If traffic is so heavy, over so many lanes, that this cannot be accomplished, then compromise settings of minimum gap lower than 2.5 sec will be needed.

CHOICE OF CONTROLLER TYPE

In answer to the second question of the survey, respondents typically stated that full-actuated control is always used for single-intersection control, unless intersection coordination is planned for the near future. In that case, semi-actuated control is selected. An isolated intersection may not require any interconnection to another signal but may be connected to a master to allow the traffic-control center to monitor the intersection's performance and signal equipment diagnostics and to remotely implement fallback functions until signal technicians can respond.

Semi-actuated control is rarely used for single intersections not planned for interconnection soon. An exception might be a location that meets the signal warrant for interruption of continuous flow or the peak-hour-delay warrant. Cross street demand in this case (typically less than 100 vph/lane) is barely enough to warrant a signal.

Full-actuated control may be implemented with the major road set either to minimum recall or "soft recall." Soft recall is a "beyond-NEMA" feature that brings the right-of-way back to that phase only for the purpose of allowing it to rest there; that phase is not called into the sequence if other conflicting calls are waiting and there are no calls on the soft-recall phase.

The respondents reported few situations in which they would select fixed-time control at a single intersection. Temporary signals and those associated with roadway construction are examples of instances where detector loops may be more trouble than they are worth. (Overhead or "side fire-mounted" detectors, however, often can be easily mounted to an existing pole.)

CHOICE OF ACTUATED CONTROLLER/DETECTOR CONFIGURATION

The third question of the survey focused on actuated control and asked if the respondent has different selections of controller (e.g., density) and loop layout for various conditions. Appendix B provides more information on this for the interested reader.

In general, for minor approaches the respondents use long (6 ft x 30 ft to 6 ft x 60 ft) presence loops located near the stop bar, with the controller phase operated in the nonlocking detection-memory mode. The detector is often operated in the delayed-call mode to screen out false calls for the green, as with right turn on red. Several small loops wired in series are preferred by some respondents to a long loop.

For high-speed, major approaches, many respondents use long detector setbacks to attempt to solve the problem of the option zone, explained earlier. Some respondents locate this advance detector loop by calculating safe stopping distance based on a 1-sec perception-reaction time and a deceleration rate of 10 ft/sec/sec, whereas others have in-house graphs yielding approximately the same results. For example, a Minnesota graph indicates a setback of 350 ft to 450 ft for an approach speed of 55 mph.

With these long set-backs, Minnesota and others, including Montgomery County, Maryland, use density control with certain loop configurations on their major-street approaches. Other respondents, such as Overland Park, Kansas, use a basic controller and additional loops to control allowable gap.

Although density features are not the extra-cost item they once were, many agencies report that they prefer not to use them. Texas rarely uses density features; for high-speed approaches it uses a basic controller and a multiple-loop design based on a perception-reaction time of 1 sec and AASHTO stopping distances for wet pavement. The California Department of Transportation's District 8 also avoids added initial interval and gap reduction, which are the two main features of density control. Instead, Caltrans uses a detector layout that it has found provides density-control results with the use of the timing intervals of a basic controller. "The advantage is that the signal is very responsive to all traffic volumes up to maximum limits. Typical values for minimum intervals are 5 seconds . . . and for extension and gap intervals are 2 seconds." Figure 9 shows its typical detection layout.

A number of the respondents are sophisticated in their consideration of trucks and grades in the use of delayed- and extended-call detection and detector-omit circuits to solve specific needs. Examples from Montgomery County, Maryland, and the state of Delaware are detailed in Appendix C.

USE OF SOAP OR OTHER PROGRAMS

Of the 43 respondents, 38 stated in response to question 4 that they used SOAP or some other computer program to time single intersections. Of the 38, eight had used something other than SOAP, such as PASSER, TRANSYT, proprietary models, or the HCM software (*3*). Of the 30 SOAP users, 10 said that they like the program or are generally pleased with it. Several of these respondents are located in Florida; one mentioned the benefit of his telephone contacts with the McTrans Center at the University of Florida for questions and assistance. Another uses SOAP extensively and praised the SOAPDIM data input manager: It greatly simplifies the coding effort and "makes the program useful to non-computer people." A Florida consultant has been pleased with SOAP to optimize phasing and lane assignments: "However, I have seen SOAP results that were contrary to my independent engineering judgment. Also, SOAP cannot evaluate the effects of permissive left turns and cannot assess relative safety of various phasing options." The latter comment refers to the fact that SOAP and other programs will recommend a phasing that is dangerous because of the "left-turn trap," a term explained in detail in Chapter Three under the heading Leading versus Lagging Arrows to Optimize Bandwidth.

TxDOT likes SOAP mainly for setting up dials. Some of the respondents reported SOAP to be practical only for pretimed controllers. They said they believed that SOAP does not adequately represent actuated control, which is the type they prefer at single intersections. A Texas consultant mistrusts SOAP results in high-volume situations.

The respondent from the New York State DOT included the results of an evaluation of mainframe SOAP conducted in the early 1980s to help establish timing procedures for the 170 microcomputer controller. The NYSDOT found that SOAP cycle lengths "were either too short or long for practical application in the field." It also concluded that SOAP "does not lend itself to determination of control parameters for traffic actuated signals."

The NYSDOT study was the only example of data furnished in response to Question 5, which asked if any studies have been made to compare the results of SOAP (or other computer techniques) with manual results or with field measurements of queue length or delay or other measures. A Texas consultant commented that clients are not usually willing to pay for such comparisons.

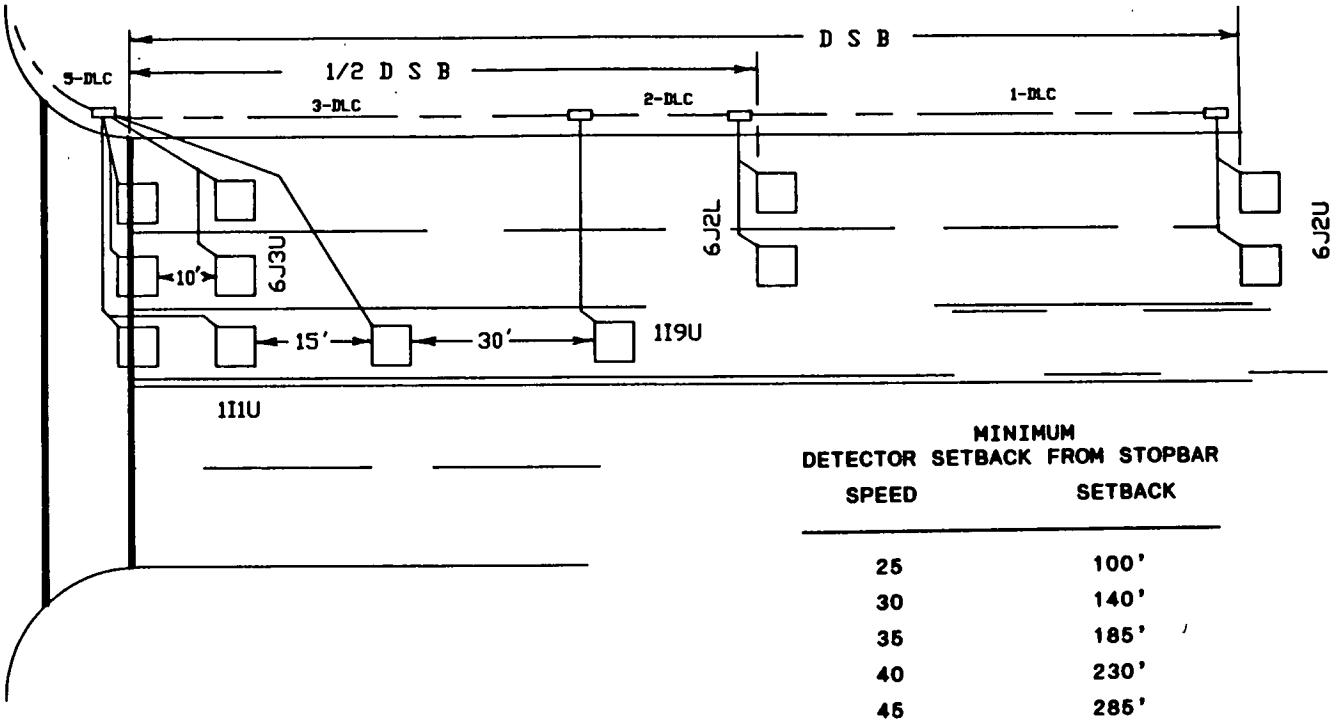
ARTFUL FINE-TUNING OF SOLUTIONS

Many respondents stated that they use field observation, exclusively, to fine-tune their manual or computer-based solutions to reduce queues and delay. Excessive delay is often judged to be present whenever there is a "cycle failure," meaning that one or more vehicles failed to clear during the next green indication. Some base their fine-tuning on complaints received, whereas others follow a schedule for initial and periodic adjustments.

Most respondents fine-tune to reduce queue buildup in left-turn lanes. One Midwest city reported that PASSER II gives more realistic left-turn times than other programs. The engineer for a California city reported the following:

> Our main scheme has been to observe the intersection at peak periods and reduce the maximum green times for all phases to attain the most acceptable cycle length. Our problems on left-turn lanes deal with short pockets, and the shortest acceptable cycle lengths seem to work best. In these locations we minimize the maximum initial, and "tighten" up the gap parameters, as much as possible. We will not incorporate any "Maximum Extension" features.

The opposite was reported by a Mississippi consultant, who field fine-tunes by increasing maximum timing, and possibly also extension timing on presence loops, to help clear queues.



MINIMUM DETECTOR SETBACK FROM STOPBAR	
SPEED	SETBACK
25	100'
30	140'
35	185'
40	230'
45	285'
50	345'
55	405'

TYPICAL DETECTION LAYOUT

NO SCALE

FIGURE 9 California Department of Transportation's typical detection layout.

Fine-tuning for other reasons than left-turn lanes is common. New York State is especially attentive to closely spaced intersections. Texas is aware that computer programs must make many assumptions, not all of which may be true. "Many features, especially the change interval and the passage time, may need fine-tuning, particularly for full-actuated operation."

In one Colorado city, visits are made to an intersection during known congested times of day to check on queue clearance and "perceived" delay. "Adjustments normally are changes in the extension time or passage time. Also, off-peak adjustments can be made to decrease delay. Some of the largest perceived gains can be achieved in off-peak, weekend, and holiday periods."

Montgomery County, Maryland, uses only manual solutions and fine-tunes them through field observations performed at initial turn-on and over the next several days during both peak and off-peak periods. The county uses an excellent scheme for pedestrian phasing at those locations where the side street is split-phased (meaning that the two side-street approaches discharge sequentially rather than in the normal concurrent manner). Ordinarily, it would seem necessary for each of the two side street phases to provide complete timing for pedestrians because the other phase might be skipped under light traffic conditions. The county's scheme assigns the associated pedestrian call to phase 8. Phase 8 times concurrently with both side street approaches. One direction of the side street is 3 + 8 and is followed by the other direction, 4 + 8, in one example. A pedestrian actuation places a minimum call on phase 3, with any remainder of phase 8 WALK/DON'T WALK time caused to time concurrent with phase 4. This avoids having to time either phase 3 or phase 4 longer than needed by vehicles so that the one phase will completely meet pedestrian-timing needs. Safe pedestrian timing is assured without providing unneeded side street time that would tend to cripple arterial capacity. See Appendix C for a detailed example.

Montgomery County also seeks to minimize delay by "creative attention" to the assignment of detectors to multi-phase operation involving overlaps. For example, a freeway off-ramp may require this attention where heavy right turns immediately encounter a signalized frontage road. See Appendix C.

Of those respondents who use computer solutions, most reported that they perform iterations to attempt to make this work more than just a coding exercise. Some consultants stated that iterations are needed more for coordinated systems than for single intersections. Several respondents noted that computer programs are not perfect in their ability to model, and that they rely heavily on the accuracy of the field data collected and capacity values selected. The Iowa DOT performs iterations to test alternative phasing schemes and cycle lengths. One East Coast consultant compares predicted high volume/capacity ratios and queue lengths with those observed in the field. Another

varies capacity, phase sequences, and number of left-turn sneakers.

MEASURES OF EFFECTIVENESS

The effectiveness of signalization at an isolated intersection can be measured visually in a qualitative way and also by studies that provide quantitative measures, such as the percentage reduction in stops, delay, accidents, or irate phone calls.

Control at an isolated intersection is desirably full-actuated; effective green timing for vehicular operation is characterized by absence of premature gap-outs (because of deficiency in the setting of minimum green time or allowable gap or both); absence of unused green time (caused by a too-long allowable-gap setting that prevents appropriate gap-outs); absence of frequent max-outs (because of long allowable-gap settings or short maximum green times or both); and absence of excessive queues on any approach (caused by too short a maximum green interval setting or by excessive demand at the intersection). Field observation by an experienced traffic engineer or timing technician, at key times of day, can lead to "snappy," efficient controller settings. A primary principle is to allow the controller to detect gaps in moving traffic (to change the green to a waiting phase) by setting short allowable gaps and long maximum green times. Failure to do this will produce max-outs so frequent that operation is reduced to that of a fixed-time controller.

Criteria for levels of service A through F are defined by the current *Highway Capacity Manual* (*3*) based on the number of seconds of stopped-time delay per approaching vehicle. For example, the level dips below C when delay exceeds 25 sec. The conventional method (*34*) to measure stopped-time delay requires two observers even under favorable conditions, and therefore can be expensive. Today, a single observer with a laptop computer and available proprietary software can do the job of two observers following the older procedure.

The effectiveness of a signal-improvement project at an isolated intersection can be measured economically. For the time of day under study, two critical approaches can be selected and observed for stopped-time delay in an alternating fashion. That is, one approach is observed for, say, 6 min and then the other is observed for 6 min, etc., for the duration of the study period. A second study one week later should give results within roughly 15 percent of the first, or else a third visit may be warranted. Studies done before and after an improvement project can show the effectiveness of it in terms of reduction in delay in vehicle-hours, stops and percent stopping, cost of stops and delay, fuel consumption, and pollutant emission (*6,35*). Estimates of these measures can be obtained before the project by computer programs such as TRANSYT-7F (*2,8*).

PROBLEMS REMAINING UNSOLVED

Some problems remain largely unsolved in this area, according to the responses. Those mentioned most frequently are listed first:

- Oversaturated intersections that lack sufficient capacity. One respondent stated that oversaturation could possibly be alleviated by improving progression or by adding lanes. Another stated that control equipment does not adequately adjust for fluctuations in traffic flow at oversaturated locations. He added that artful techniques to deal with oversaturation do exist among experienced engineers but have never been set down in writing; thus, there is a lack of useful literature. It is clear from the responses that further work is certainly needed to guide selection of cycle length. As many traffic engineers are bent on minimizing it as are trying to push it to the tolerable limit. According to a respondent from TxDOT:

> Until data collection becomes easier, the area of timing isolated signals will not advance very far. The engineer has many things to do, and retiming an isolated intersection generally does not rank highly. Accurate data is needed for good timing but is very expensive. Recent research by the Center for Transportation Research in developing an automated method for turning movement counts could help.

- Lack of effective data collection, including traffic counts, delay, and saturation flow. Lack of data hinders the traffic engineer in making effective choices and decisions.
- Insufficient personnel to conduct the field surveillance that is essential to keep signal timings optimized (from Montgomery County, Maryland).
- Determination of settings for density controllers. (This was mentioned by two respondents, both consultants in the Southeast.)
- Need for computer control of queues.
- Need for the development of better computer programs for signal simulation and optimization.
- Lack of field observation. According to an East Coast consultant:

> I feel that too many computer-based timing solutions are utilized without any field observation. I obtained my signal-timing background while in the field 90 percent of the time and have compared both timing methods and feel that field timing gives the best end result, though computer solutions are a great start. I also realize that extensive field adjustments are not practical if even possible.

- Detection of an approaching platoon. This aids the ability to predetermine when an actuated signal will change to yellow, so that PREPARE TO STOP WHEN FLASHING signs can begin to flash at the proper time.
- Improved pedestrian timing.
- Determination of the precise effect of cycle length and lost time on through-put.
- Determination of optimal values for maximum cycle length.
- Determination of optimal detector location.
- Need for variable passage time, yellow and all-red, because of adverse weather or certain volume conditions. Higher clearance intervals could be used in light-volume conditions to increase safety (from a consultant in the Southeast).
- Need for protection against the left-turn trap. (The left-turn trap is explained in detail in the next chapter, under the heading Leading versus Lagging Arrows to Optimize Bandwidth.) According to a southeastern consultant who chaired a major study on left-turn phasing:

> Too many operating agencies are not protecting against the left-turn trap that occurs at a four-way permissive left-turn intersection when the controller "backs up" from phases 2 and 6 to phases 1 and 6, to cite one example.

- Need for a signal-head configuration that would allow protected-only left-turn phasing during part of a day and permissive (or protected-permissive) phasing during other parts.
- Tendency of computer program-generated cycle lengths to be too short.
- Need for improvement of controller logic. If an actuated controller under low loading skips a phase, there is ample time to serve it out of sequence if someone arrives there before the artery is again served.

CHAPTER THREE

COORDINATED SYSTEMS

Two or more signalized intersections can be operated in coordination, meaning that their signals have a fixed time relationship to one another. The relationship can be imposed through a communications system of some sort or through accurate time-of-day clocks at each intersection. If the beginning of green is offset from one intersection to another, then a "green wave" or cascade effect can be created that promotes nonstop flow. "Progression" is achieved if the offsets between adjacent signals permit continuous operation of groups of vehicles at a planned rate of speed.

Many of the potential benefits from signalizing intersections are realized only when a number of them along an arterial (or in a grid) are coordinated and progressive movement of platoons of vehicles is achieved. Continuous flow promotes the realization of the traffic engineer's "bottom line" goals—the reduction of stops, delay, vehicle operating cost, motorist time cost, accidents, fuel consumption, and pollutant emissions. Good signal timing can produce progressive movement on an uncongested one-way street, but progression in both directions on a two-way street can be difficult or impossible. Two-way progression, even in the absence of congestion, depends on a harmonious relationship or "dovetailing" among signal timing, traffic speed, and signal spacing. The traffic engineer has little power to change the latter two factors; the most powerful computers running the latest in sophisticated software cannot force awkward block lengths to coordinate well with desired traffic speeds and reasonable cycle lengths.

CYCLE LENGTHS IN COORDINATED SYSTEMS

It is common for all the intersections in a coordinated system (or control section) to have the same cycle length, which generally is long enough to provide sufficient capacity at the busiest intersection. However, in some cases, shorter cycles may produce fewer network stops and less delay. The system cycle length is found through a series of steps. The first is to determine the minimum (optimum) cycle length needed at each intersection, as though it were isolated. The longest of these cycles is termed the "maxmin" cycle length by the documentation for the PASSER program (*12*). If the signal system is an arterial with equal block lengths, the traffic engineer will consider the "single alternate" or "double alternate" timing pattern. [See primary references such as Pignataro (*17*, p. 375) for an explanation of the alternate systems.] For each pattern there is a mathematical formula relating cycle length, desired speed of progression, and block length. The latter two are entered into the formula and solved for the cycle length. The pattern that has a cycle length closer to, but not less than, the "maxmin" cycle length is chosen.

The PASSER documentation suggests that the cycle length be selected from a range that has a lower limit calculated as 90 percent of the "maxmin" cycle length. This seems reasonable, because in Chapter One it was shown by Figure 1 that an optimum cycle length can be reduced to three-quarters of its value before delay increases appreciably. (In fact, some engineers set the lower limit at 75 percent of the maxmin.) The PASSER program is run for cycle lengths over a range from this minimum to a value normally only 10 sec greater. The optimum cycle length is usually taken to be the one within this range that produces the best arterial progression as shown by the bandwidths of a time-space diagram.

The PASSER procedure decidedly seeks to minimize system cycle length, to the extent allowed by considerations of capacity and pedestrian needs. This is a most desirable goal no matter what particular timing method is used. Overly long cycle lengths increase delay and can produce excessive queue lengths that may threaten upstream-intersection operation.

Too-long cycle lengths also reduce left-turn capacity on those approaches where left-turn phasing is of the "permitted" type. Traffic engineers are careful to take into account the average of two vehicles that legally "sneak" the left turn at the end of each circular green. A 60-sec cycle allows an average of 120 such lefts per hour for each phase; doubling the cycle length would cut this range in half and increase the pressure for a protected left-turn phase.

The TRANSYT-7F program (*2*) determines the optimum cycle length in a much different way from PASSER. TRANSYT includes a traffic-simulation model that considers platoons rather than individual vehicles. TRANSYT further considers the spreading-out, or dispersion, of these platoons as they discharge on a green signal and proceed down the block. The program calculates accurately the stops, delay, and other system measures for these platoons. Through an optimization algorithm it seeks to minimize a "performance index" (PI) based on stops and delay (*2*):

> The user inputs a minimum and maximum cycle length and a cycle length increment and TRANSYT-7F optimizes offsets and phase lengths... for every cycle length....The "best" cycle length is the one that results in the lowest PI after offsets and phase lengths have been optimized. This process accounts for not only the effects of volume and capacity at individual intersections, but also the effects of traffic flow patterns in the network and spacing between signals. Methods such as Webster's formula only account for volume relationships at individual intersections.

OFFSETS IN A COORDINATED SYSTEM

The offset is the number of seconds or percent of cycle length that the green indication appears at a given signal after a certain instant used as a time-reference base. Figure 10 (*36*) is a time-space diagram showing offsets measured in seconds from the beginning of green at the first intersection.

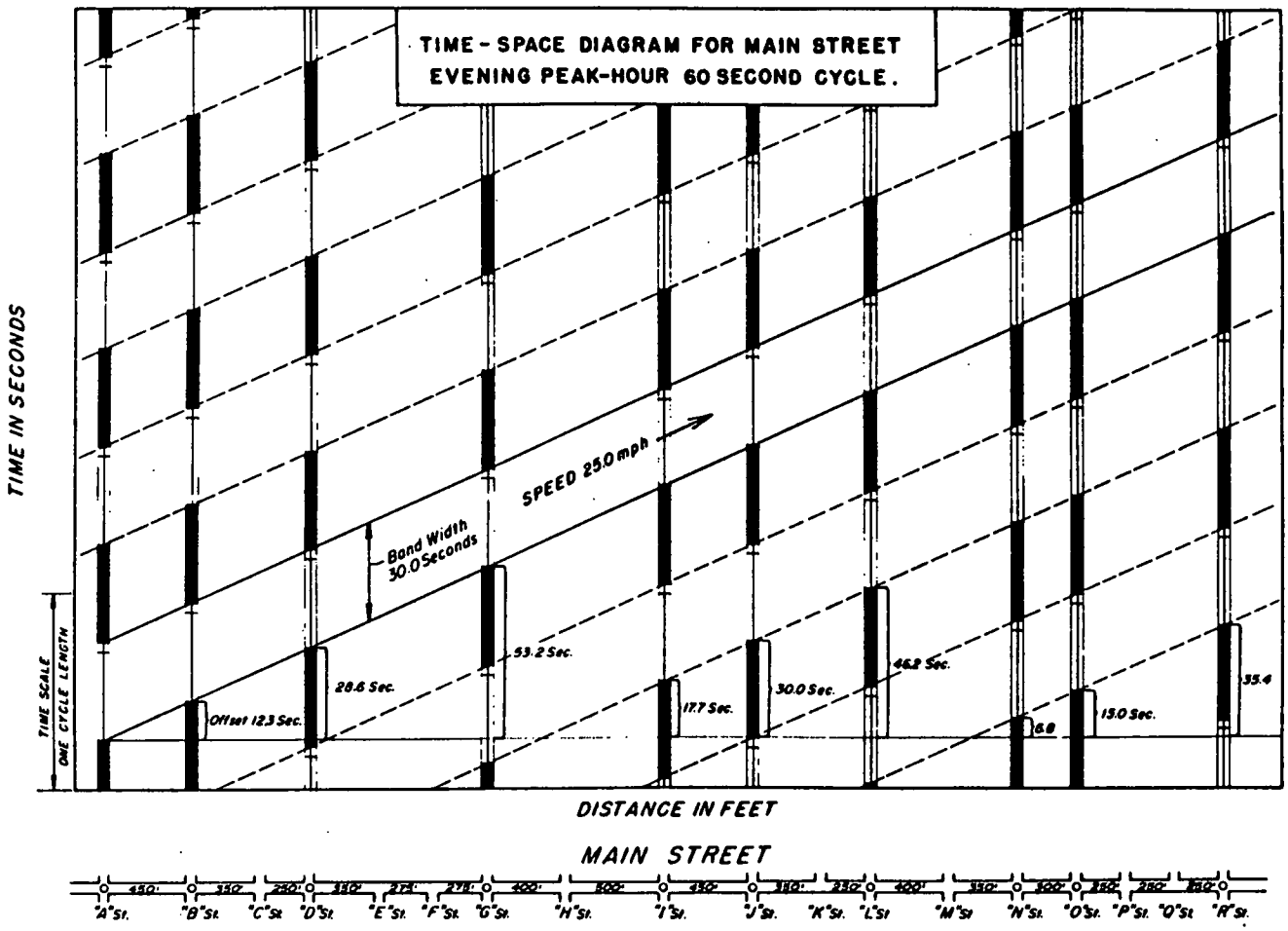


FIGURE 10 Time-space diagram favoring one direction of flow (*36*).

Speed of Traffic versus Speed of Progression

The offsets labeled in Figure 10 are shown to have resulted in a speed of progression of 25.0 mph. The speed of progression is often selected to be the free-flow speed of traffic, meaning the speed that might be observed when volumes are light and the signals are continuously green on that route.

As traffic becomes heavy, during the peak periods of flow, traffic speeds tend to drop because of congestion. Traffic starting up on a green signal may be stopped by a queue not yet into motion at the next signal downstream.

Accounting for Queue Buildup

A queue at a downstream intersection can build up from congestion or from traffic turning into the artery from a cross-street upstream. The need is to release that queue early enough so that it will not interfere with traffic arriving from behind. The first principle to avoid this interference is to increase the speed of progression, thereby reducing the offset and getting the downstream queue under way sooner. The term "speed of progression" is used here to mean the speed of the "green wave," not vehicle speed. The Virginia Department of Transportation (VDOT) (*37*) has developed a procedure to calculate a progression speed that will assure that the mainline platoon will not have to decelerate or stop for vehicles queued at the downstream intersection of a link. Such calculations often determine needed progression much higher than the speed limit. For many years, traffic engineers have used for their "heavy-traffic offset" an infinitely high speed of progression, in which all signals in that direction turn green at the same time. This plan is termed "simultaneous offsets." Thus, during heavy traffic, there is no need to limit the speed of the green wave to the posted speed limit for fear of encouraging speeding. (During light traffic, however, simultaneous offsets would encourage speeding if used on a road so straight that drivers could see several signals changing to green at once.) The band of actual traffic speed needs to be shown on the time-space diagram.

If increasing progression speed does not eliminate this problem, the next step is to introduce a "queue-clearance factor" at one or more key intersections. The PASSER program (*12*), for example, permits the user to force the arterial green at selected intersections to begin up to 10 seconds earlier than the "arrival" of the through band. The time-space diagram in Figure 11 (*36*) was not derived from the PASSER program but can be used to explain queue-clearance factors. Suppose that traffic moving from right to left is being delayed by a queue on Main Street at

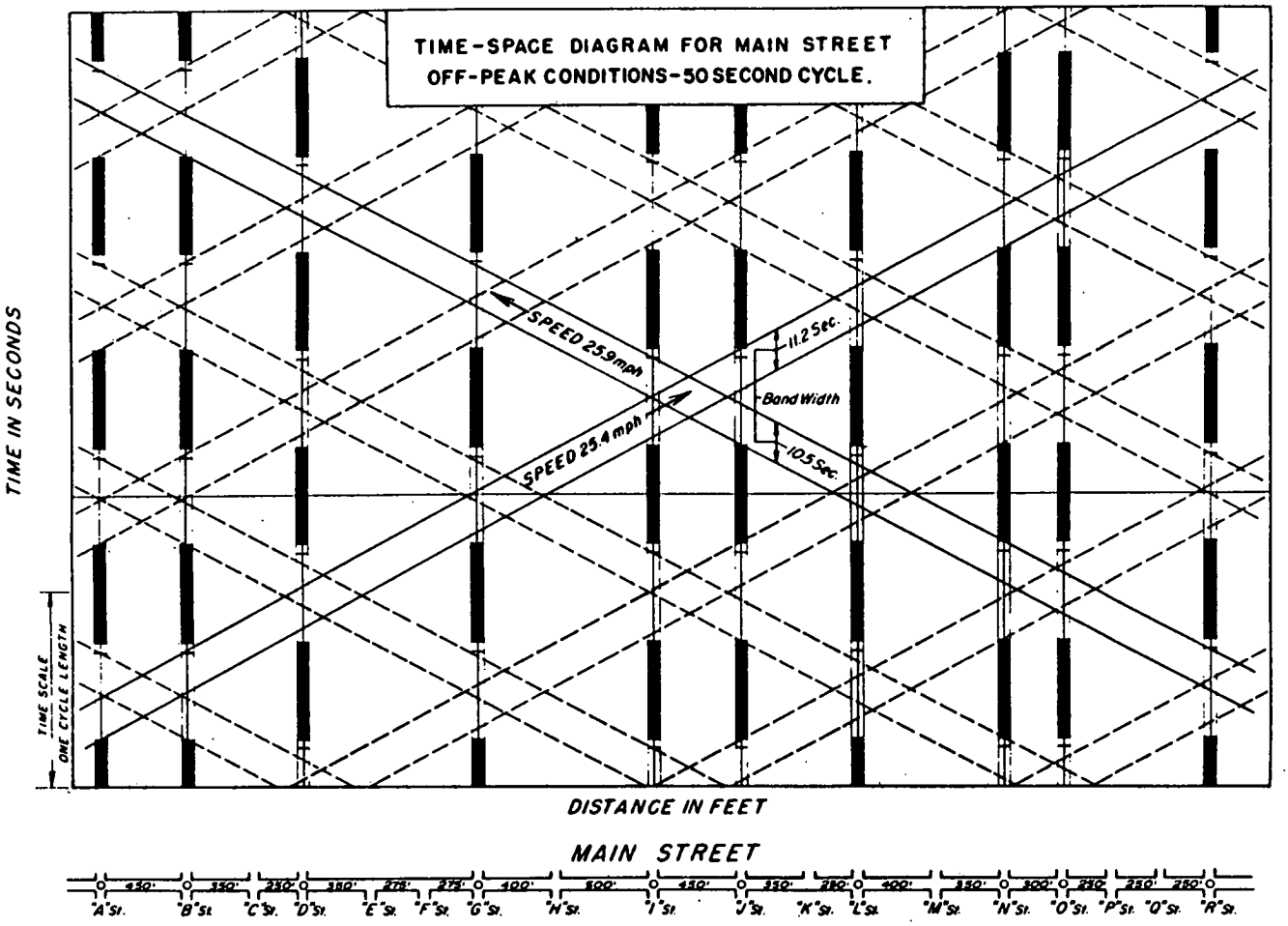


FIGURE 11 Time-space diagram favoring both directions equally (*36*).

the intersection with B Street. The timing strip at B Street can be moved down by an amount seen to be a little less than the yellow time (e.g., 3 sec) before blocking the through band for the opposite direction. Therefore, traffic can be released up to 3 sec earlier than the arrival of the through band. A program such as PASSER could adjust the entire time-space diagram to give even greater queue-clearance time, if desired.

The allowable movement up or down of a timing strip, without the red entering a through band, will hereinafter be called "allowable adjustment."

Traffic Turning into System

Time-space diagrams such as Figures 10 and 11 can be easily seen to be directed primarily to moving main-street platoons over the entire length of the artery with minimum stops and delays. In reality there may be significant flows turning into the artery from one or more cross-streets or driveways; these flows may not be well served unless adjustments are made to the diagram. For example, in Figure 11, suppose that a significant volume of traffic turns right from I Street onto Main Street. Because of heavy traffic on Main Street, most of the right turns cannot be made on red but must wait until I Street green (corresponding, of course, to Main Street red on the diagram). The arrows drawn from I Street to the right show that the turning traffic will experience "stair-stepping," requiring successive stops at J, L, and N streets. Little can be done through manual adjustment of the timing strips, because there is no allowable adjustment in the strips at J and N streets. Fortunately, turn-in drivers expect stops. The best solution to this stair-stepping problem would be obtained from a program such as TRANSYT-7F (*2*) that can minimize stops and delay while explicitly modeling the turn-in traffic.

Adjustments at End Intersections

Time-space diagrams usually are constructed so that, at the first intersection, the start of the through band coincides with the start of green. This is so at A Street in Figure 11 for traffic entering the system by moving from left to right. At the end intersection, however, there may well be no such correspondence. Such is the case at R Street for traffic entering the system; it does not start up where the through band would suggest but instead gets into motion at the start of the green. Motorists entering the system will stop at O Street and again at J Street, as shown by the arrows. At R Street there is enough allowable

adjustment in the timing strip for it to be raised almost enough for traffic to clear O Street without stopping. If it were raised more, the red would enter the through band for traffic moving from left to right. Because R street is the last intersection for that traffic, the engineer might strike a compromise between the need for left-bound traffic to "get off on the right foot" and the need for right-bound traffic to clear its last intersection. There is a proprietary computer program that allows the user to specify a "leading edge" option; it reports only those solutions in which the start of the green coincides with the beginning of the through band at the end intersections.

Early release of through traffic at the beginning and end intersections is more of a problem if the controllers are semi-actuated. That is because the start of green on the artery is not a fixed point in the cycle. (The end of main-street green is fixed instead.) At a semi-actuated intersection the sensors on the side-street approaches can cause the side-street green to end early by gap-out or later by max-out. The problem can be solved by locking side-street calls into those controllers whenever the system is in a coordinated mode of operation (i.e., whenever the intersection controllers are not running free because of light traffic). Thus, the controllers are operated on fixed time to avoid early release. If the cross-streets at the end intersections have sufficient volume, fixed-time operation for this purpose may be acceptable.

A 1988 FHWA report, *Progression Through a Series of Traffic Actuated Controllers,* by Skabardonis (*38*) describes procedures for translating pretimed timings to actuated controllers' settings for both arterials and grid networks. Criteria for choosing the type of control at selected intersections in coordinated systems are also presented.

Adjustments for Left-Turn Phases

Regardless of the method used to construct a time-space diagram, the timing strip at each intersection can be interpreted for opportunities for left-turn phases of various kinds. Figure 12 shows six examples, labeled as Intersections 1 through 6, of interpretations based on the relative times of arrival of the two through bands. At Intersection 1 the first car in the inbound through band does not arrive at the intersection until well after the first outbound car. An amount of time shown by the asterisk, therefore, is theoretically available for an advanced left turn to be shown to outbound traffic. (The theory relies on there being no inbound traffic already queued at the start of the green, as from traffic turning in from a cross-street upstream.)

Intersections 2 through 6 show other examples. Intersection 5 shows an opportunity for a delayed left for only one of the directions. (It is the responsibility of the engineer to be aware that such phasing can cause a dangerous "left-turn trap" and to use one of several methods available to remove the danger. The left-turn trap is described in detail later in this chapter, under the heading Leading versus Lagging Arrows to Optimize Bandwidth.)

The last subsection introduced the need to control the start of main-street green at the end intersections, to avoid early release of traffic entering the system. The problem is compounded if main-street green is preceded by an actuated left-turn phase; the arrow phase is of uncertain duration because of the minute-to-minute fluctuations in actuations from left-turning traffic. The phase could end early by gap-out or later by max-out. A delayed (lagging) green would be preferable, to keep the start of main-street green "pure" to traffic entering the system. (The design should present no "left-turn trap" to drivers, as discussed in detail later.)

Offsets for Maximum Bandwidth

One strategy for the timing of arterial signals is to use offsets resulting in the maximum width of the through band. This philosophy emphasizes the interests of arterial through traffic rather than those of conflicting flows. The leading program for this is PASSER II (*12*), which has as its objective function the maximization of the sum of the two arterial bandwidths. (In fact, it maximizes the sum of the two bandwidth efficiencies, defined as bandwidth divided by cycle length.) The user may specify that the sum be apportioned to the two directions as desired, such as 60/40 or proportional to the traffic volumes in each direction, for example. Moreover, the program explicitly considers left-turn phasing, including overlaps, and can determine the phasing at each intersection that will maximize the bandwidths. In addition, the user may specify different speeds for each link and each direction. There are many other excellent features, such as user-specified queue-clearance times at selected intersections. Figure 13 shows an example time-space diagram produced by PASSER II-87 and illustrates the explicit consideration of left-turn phasing (*12*).

Since the 1984 version, PASSER II has used an additional procedure to minimize delay by fine-tuning the offsets (*39*). Once the through bands have been maximized, they are held fixed in position, and the program proceeds to make the best use of the allowable adjustment ("slack time") in each timing strip. Reductions in delay of 5 to 15 percent have been reported, with the greatest improvement found at closely spaced intersections (*39*).

Cohen and Liu have developed another fine-tuning procedure (*40*). First the user obtains a time-space diagram with acceptable bandwidths, using PASSER II or some other procedure. These bands are then held fixed in position, and the TRANSYT-7F program proceeds to make the best use of the allowable adjustment in each timing strip. This minimizes stops and delay within the constraints of the bands. This was implemented in Release 6 of TRANSYT-7F and included in the new Arterial Analysis Package (AAP).

The acceptability of a PASSER II solution (with or without fine-tuning) depends on obtaining bands of acceptable width. In the 1970s, arterial systems yielding narrow bandwidths from PASSER II had to be adjusted manually. The user could introduce a "planned stop," perhaps at a single intersection and in just one direction, and often obtain much better bandwidths in both directions over the entire route (except, of course, at the location of the planned stop). Intuitively it seemed that the planned stop reduced system-wide delay. Today, manual adjustment can be replaced with TRANSYT-7F used with an Arterial Priority Option, as discussed next.

Offsets for Minimum Stops and Delay

The TRANSYT-7F program was described and discussed briefly in Chapter One in connection with the National Signal

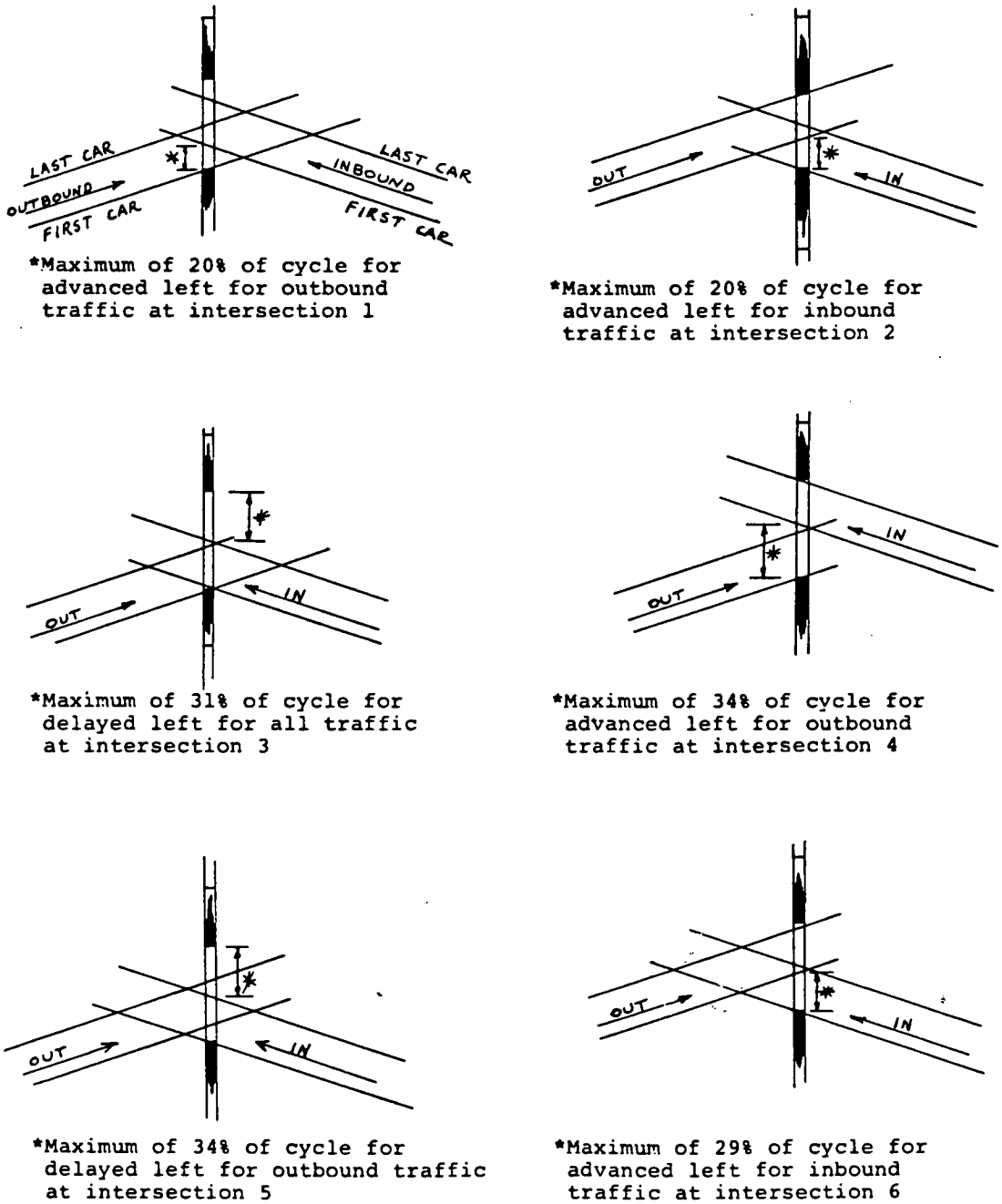


FIGURE 12 Left-turn phasing related to through-band arrivals.

Timing Optimization Project carried out in the early 1980s. Unlike PASSER II, TRANSYT-7F minimizes stops and delay to all traffic without regard to bandwidth; time-space diagrams were not included by the British developers of TRANSYT. Time-space diagrams were added to the output of the program by United States traffic engineers in the creation of the 7F version (*2*). Briefly, TRANSYT uses an excellent traffic model that, although not microscopic, simulates traffic dispersion as shown in simplified form in Figure 14. A second component of the program optimizes offsets (and other parameters) to produce minimum stops and delay. The results are graphed primarily through histograms called flow profiles, as explained in Figure 15. The flow profiles for two adjacent links can be offset from one another to account for travel time, as shown in Figure 16, permitting a good visual impression of the progression.

TRANSYT tends to produce excellent results for grid networks, because normally there is not much difference in priority among the intersecting streets. For arterials it has not been as successful, at least in some applications, because TRANSYT gives no priority to arterial through traffic. That is, stops and delay to conflicting movements are equally worthy to be minimized. TRANSYT's philosophy is in contrast to PASSER II's

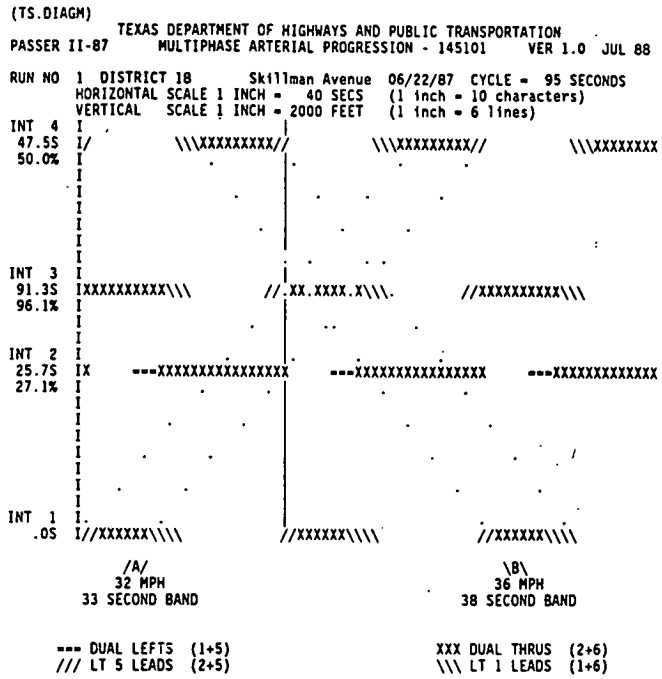


FIGURE 13 Example of time-space diagram from PASSER II (87) (*12*).

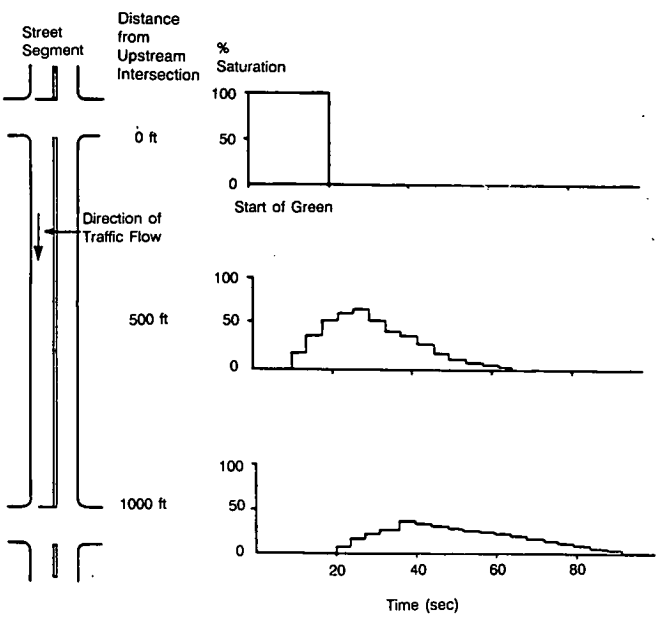


FIGURE 14 Simple case of platoon dispersion in TRANSYT-7F (*2*).

and may lead to timing plans that do not provide traditional progression bands along arterial routes.

Moskaluk and Parsonson (*41*) have presented a solution to this difficulty, the Arterial Priority Option (APO). Briefly, the user specifies which links are to receive priority and the desired degree of saturation for the minor movements (nonpriority links). TRANSYT then minimizes stops and delay only for the priority links. The degree of saturation specified for the minor movements holds the performance to acceptable levels. The results of a program run may be used to make changes to the list of priority links and to the required degree of saturation of one or more nonpriority links, in the judgment of the engineer. APO is thus user-interactive. The engineer retains firm control over the relative priority given to the various movements in an arterial system.

EXAMPLE OF TIMING CALCULATIONS

The respondents to the questionnaire furnished a number of examples of PASSER and TRANSYT runs and a few manually constructed time-space diagrams. One of the latter included comments that "brought it to life" and made it suitable for this report. Figure 17 shows a time-space diagram that pertains to a 2.5-mile section of Northwest Highway in Dallas, Texas. A shopping center near the center of the system was a candidate for a signal at an exit. Proposal "A" was to place the signal at the Plaza Main intersection, whereas "B" would place it at Admiral. The time-space diagram had two main purposes: to determine which signal would have less bandwidth impact and therefore would be the better location for the signalized exit, and to determine a good midday pattern of cycle, offset, and split for this section of the arterial.

The intersections were plotted on a horizontal axis calibrated in seconds of travel time rather than feet of distance. (This procedure compensates for differing link speeds, but the speed and travel time in each link must be the same in both directions.) The vertical axis was calibrated in percent of cycle length, so 200 means two cycles up from the origin. The split at each intersection was determined using existing conditions. It was desired to achieve equal two-way progression, meaning that each band would have the same width and the same slope (speed).

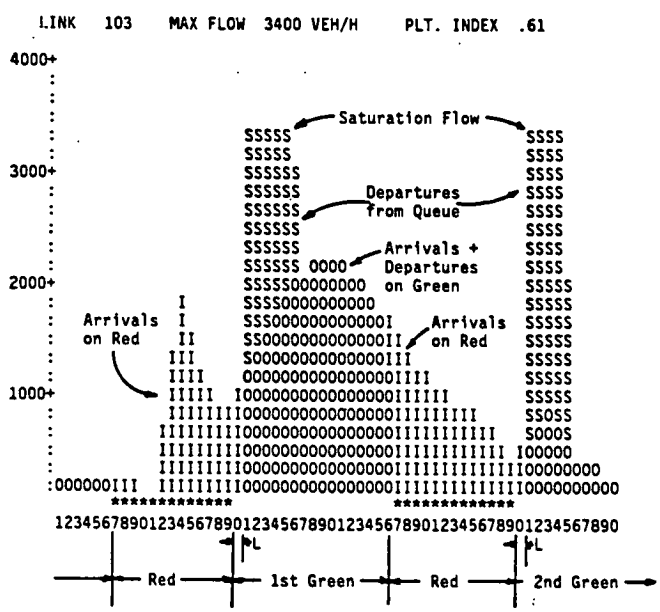


FIGURE 15 Interpretation of TRANSYT flow profile diagrams (*2*).

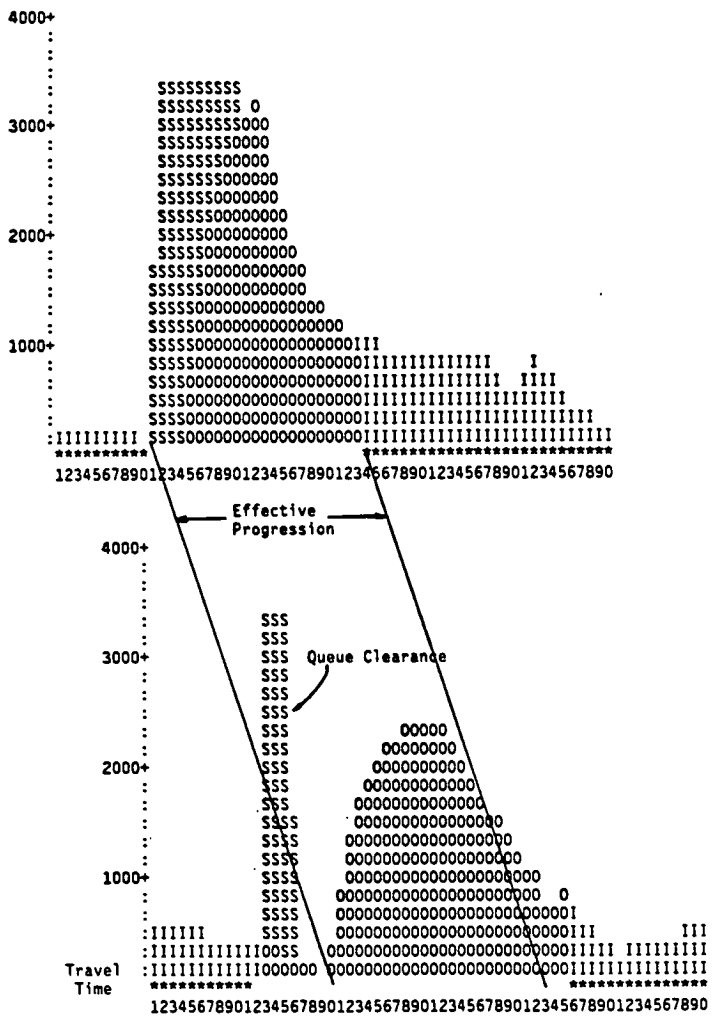


FIGURE 16 Use of TRANSYT flow profiles to evaluate progression (*2*).

Equal two-way progression can be obtained by a manual construction procedure explained in several texts (*13, 16*). A working line is drawn horizontally, such as through the 100 percent point. A paper timing strip is then cut and colored for each intersection. (The yellow is included in the green for this analysis.) At the first intersection, the strip is fixed so that the working line bisects a green interval, as shown in Figure 17. At each successive intersection either a red or a green is centered; the choice depends on the desired slope of the band. (In this diagram the horizontal axis is travel time. The time to travel each block is fixed, so speed is fixed. Therefore, the slope of the band determines the cycle length. If the horizontal axis is distance, then the slope of the band determines speed multiplied by cycle length, a certain constant; for a given slope the user can choose any desired combination of speed and cycle length that, when multiplied together, yield that constant.) Leading and lagging greens complicate the centering, as shown at the diamond interchange. Centering either a red or a green will always produce equal two-way progression. (See Figure 11 for another example of this.) After the centering is completed, the through bands are drawn in. If they are forced to have the same slope, then they will automatically have the same width.

Several different solutions will commonly result if the user varies the slope of the band in these trials. It is convenient to characterize a solution by the sequence of colors centered on the working line; considering the six existing intersections (omitting the shopping center), we would call the solution a GRRRRG.

The solution is now checked to see if it has an acceptable cycle length, calculated by dividing the travel time through the system (on the horizontal axis) by the elapsed time measured in cycles (on the vertical axis). Summing the block travel times of 32, 16, etc., sec from Lakefield to Inwood gives 215. The trip eastbound begins at 90 percent C on the vertical scale at the Lakefield intersection and ends at 285 percent C at Inwood, for an elapsed time of 195 percent C or 1.95C. If 1.95C = 215 sec, then C = 110 sec. If that cycle length provides just enough capacity at the critical intersection or is required by a crossing system, then it is adopted. If a shorter cycle length would be acceptable, then the timing strips would be shifted to force a greater slope to the through bands. The solution would no longer be GRRRRG, the

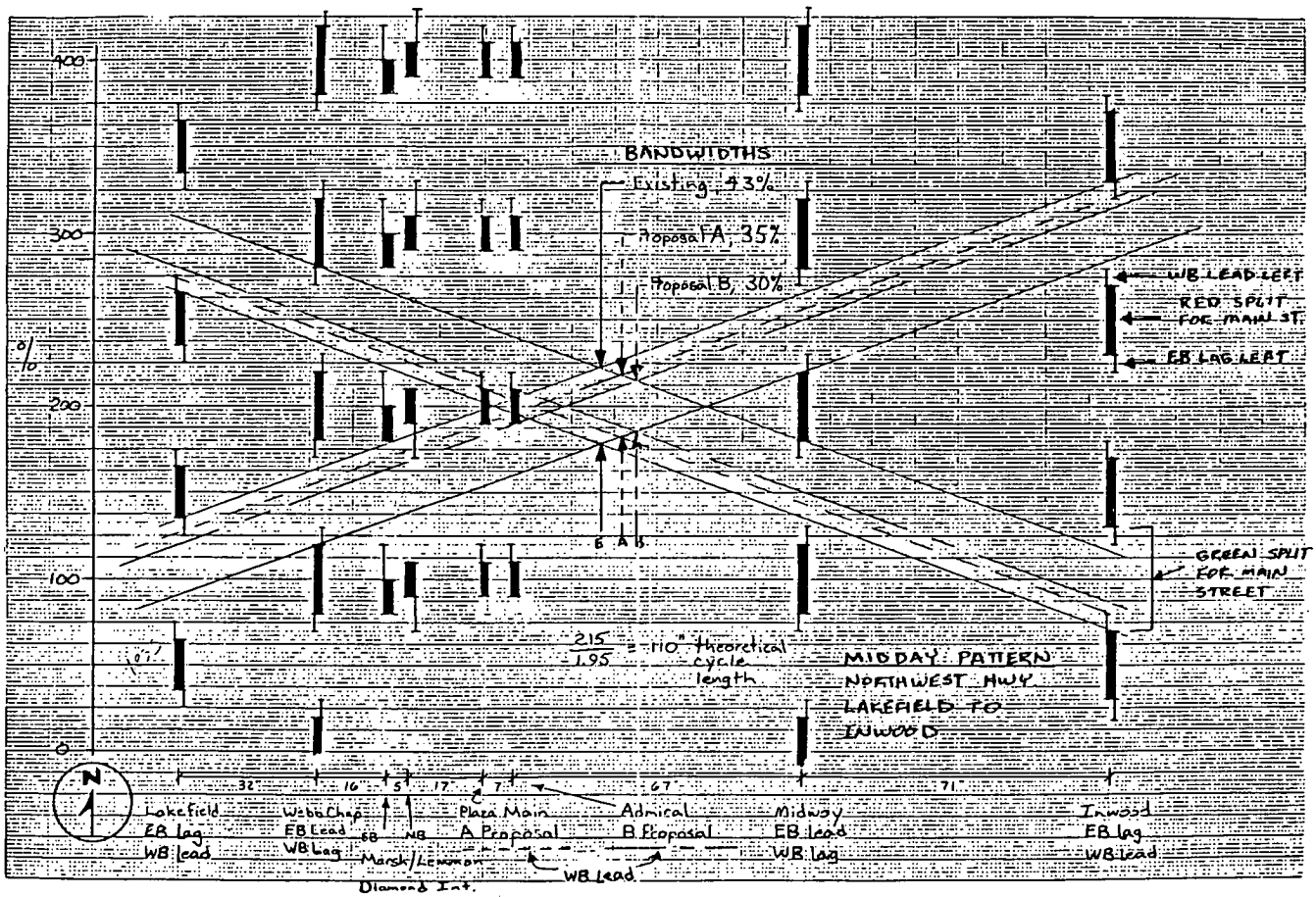


FIGURE 17 Manually prepared time-space diagram.

1.95C would increase to a larger multiple of C, and the cycle length would decrease as desired. The bandwidth would change.

Each of the four existing intersections not located at the diamond interchange has lead-lag phasing; a westbound leading left is followed by both through movements and then a lagging left turn for eastbound traffic. The efficiency of this scheme is especially evident at the Midway intersection. It is easy to see why computer programs often point to lead-lag phasing as the most efficient. However, lagging left turns can be dangerous if their design results in the "left-turn trap." It is the responsibility of the traffic engineer to implement any lagging left turns in a safe manner, such as that explained later under the heading Leading versus Lagging Arrows to Optimize Bandwidth.

Now each of the candidate locations for the new shopping center signal can be tested. Proposal A places it at Plaza Main and is shown to decrease the bandwidth to 35 percent. Proposal B, for the Admiral location, reduces it to 30 percent. The Plaza Main location is selected.

CHOICE OF CONTROLLER TYPE

Most respondents stated that semi-actuated control is the normal choice for any system outside the central business district (CBD). Within the CBD, semi-actuated control may still prevail, particularly at busy multi-phase locations, but there may be a number of fixed-time intersections mixed in. A closely spaced grid system with low vehicular volumes and high pedestrian volumes seems to lend itself best to fixed-time control. Montgomery County, Maryland, had this thoughtful response to this question on choice of type of control:

> Fixed time is used only in CBDs at some intersections where ped volumes are high all day and into the evening, and ped timing is the major determining factor in the signal timing. We also use fixed time in a few very unusual circumstances with extremely complex overlaps or other special phasing requirements dictated by unusual intersection geometry. This might be a five-legged intersection with *all* approaches major arterials. For example, we have overlapped a pedestrian movement with a portion of a left-turn phase, while holding the parallel through movement red during that portion, and then releasing it for the remainder of the left-turn phase; this cannot readily be done with a NEMA actuated controller.

This quotation shows that fixed-time control is important where phases must keep a precise relationship to one another through stringent control. An East Coast consultant responded that he has found that, at certain times of day, an arterial may move better under fixed-time operation than it will with the normal semi-actuated control. His example focused on the familiar problem of early gap-out on the side street causing an early release of main-street flow: "The cars waiting at this light would

start out on the green, then hit the back of the queue at the next light, and the whole arterial would collapse for a couple of cycles." With the flexibility of modern computer control, it is easy enough to lock in a call for side-street traffic at selected locations and critical times of day. Another East Coast consultant stated that, for arterials, the end intersections, at least, should be fixed time, because the problem of early release is worst there.

Yet another East Coast consultant commented that, where two arterials cross one another, the main-street phases for both (normally phases 4 and 8) would be operated on "maximum recall." This would create basically a fixed-time operation, with actuated left-turn phases. Toronto uses fixed-time control at the intersection of two major arterials and semi-actuated control at the intersection of an arterial and a collector; Toronto does not use full-actuated control at all.

Most respondents stated that they mix fixed-time and semi-actuated controllers in systems; those who do not mix tend to be those who simply do not ever use fixed-time control.

A number of respondents use full-actuated control in systems, but there is no evidence that these controllers are operated full-actuated during coordinated levels of operation (when the intersections are "on-line"), except, perhaps, where two arterials cross. Instead, during coordinated operation most full-actuated controllers are operated semi-actuated or fixed-time, as just discussed; full-actuated operation primarily is reserved for light-traffic periods when the master takes the intersections off-line to run "free." When master-computer control is lost because of malfunction or other reasons, some agencies prefer to have the controllers fall back to full-actuated control as the standby mode; others prefer a fallback to time-based coordination of semi-actuated controllers.

USE OF PASSER, TRANSYT, OR OTHER PROGRAMS

This section summarizes the responses to Questions 13 and 14, relating to respondents' use of various computer programs to time coordinated signal systems. These programs were described briefly in the introduction; more detailed information may be found in the Bibliography. Of the 34 respondents, 28 had used PASSER II; 11, PASSER III; 27, TRANSYT; and 8, NETSIM. Most of the NETSIM users mentioned that they had little experience with the program.

Of those who had used PASSER II, about half currently use it to evaluate alternative phasings. One East Coast consultant uses multiple TRANSYT-7F runs for this purpose. Several consultants noted that normally the client specifies the phasing.

SIDE-STREET MINIMUM GREEN FOR COMPUTER SOLUTIONS

Question 15 asked, "When coding PASSER or TRANSYT, do you use ped minimum time for your side-street minimum? Or do you use vehicle minimum time for that, because a ped call will preempt the intersection out of the normal system?"

Many respondents who do use PASSER or TRANSYT said simply that they always use ped minimum time, that selection is widely seen as a conservative, worst-case scenario vital for safety and liability. If a pedestrian call does not occur, the main-street green time will be better than modeled by the program. However, field fine-tuning of actuated equipment will often produce an offset based on vehicle-minimum time.

A number of respondents vary their selection according to circumstances. One West Coast engineer has found that, at certain offsets, a pedestrian actuation can "ruin" the timing plan for several cycles thereafter. Therefore, his policy is that, if pedestrians are negligible and a serviced pedestrian does not "damage" the system, he uses vehicle minimums. On the other hand, if pedestrian calls are entered in, say, 25 percent or more of the cycles, or if cross-traffic is significant, then he uses pedestrian minimums. Other respondents reported similar policies based on estimates of the probability of pedestrian actuation. The threshold for the proportion of pedestrian-actuated cycles, below which vehicle minimums would be used, was reported to be about 20 to 25 percent, or 10 to 15 cycles per hour. (Note that a pedestrian call is always fully serviced with safe timings; this discussion deals only with the decision to select offsets in one way or another.)

The respondent from the city of Lakewood, Colorado, said that when a pedestrian call is entered and pedestrian-split time exceeds vehicle-split time, its system will "steal time from another phase. . . and allow the phase with the ped movement to be serviced longer without disrupting coordination. . . ."

LEADING VERSUS LAGGING ARROWS TO OPTIMIZE BANDWIDTH

Question 16 asked for the respondents' position on leading versus lagging left-turn arrows to optimize bandwidth. In many cases the typical operation is for the left-turn phase to lead the through movement, an early gap-out of an actuated left-turn phase will cause early release of the oncoming through movement, potentially damaging the progression. A lagging left-turn phase keeps the start of the through movement "pure." Although most equipment is designed to default to leading left-turn arrows, programs such as PASSER often will indicate lag, or lead-lag, as more efficient, because it improves bandwidth.

A lagging left-turn phase should be used only if the bay provides sufficient storage; any overflow of the bay during the preceding through movement will spill into the adjacent through lane, blocking it. A lag should also be reserved for those situations in which opposing left-turn movements (or U turns) are safe from the left-turn trap (or are prohibited).

The left-turn trap is a safety hazard that has been explained clearly in at least one reference (*24*, p. 4–17), as follows:

> Discretion should be used with lag-left turn phasing as they may introduce operational problems which should be avoided. . . .By far the most critical of these problems is where one approach's right of way is terminated while the opposing [oncoming] approach continues with a green arrow and an adjacent through movement. This may result in a "trap" for left-turning drivers facing a yellow indication. Ordinarily, the left-turning driver facing a yellow display will expect the opposing through traffic also to have a yellow signal and since the through traffic will be stopping, he believes that he can complete the turn on the yellow indication or immediately after. Since through traffic is not stopping, a potentially hazardous condition exists.

The left-turn trap hazard will exist when the following conditions are present and may be relieved as described:

- The intersection must be a four-way intersection or a T-intersection with a driveway opposite the stem of the T, making it operate as a four-way intersection. The trap cannot occur at a T-intersection or at one that has been made to operate as a T by prohibiting the "trapped" left turn by means of a regulatory sign or by converting the two-way cross-street to one-way operation. In fact, these are two candidate countermeasures to remove the trap.
- The trapped left-turn vehicle must be facing a (permissive) green ball while opposing (oncoming) traffic is moving, and the signal must turn yellow when the adjacent through traffic receives its yellow. (Protected-permitted phasing is not a requirement for the trap to exist.) The trap can be defeated if the hazardous left-turn movement can be given protected-only phasing, with a green arrow followed by a red indication.

One respondent pointed out that, although NEMA controllers tend to produce leading left-turn phases, in the absence of a cross-street call the artery through movement can easily be followed by a left-turn phase. Such a lagging left turn can produce an unsafe trap; the controller can be prevented from "backing up" in its sequence by asserting a "left-turn-phase omit" command during the through green.

The respondents generally use leading arrows but will use a lagging green one to favor bandwidth at locations where the left-turn trap is not a factor. These include T-intersections, those where the left turn (or U turn) opposing the lagging green arrow is prohibited or is allowed only on a green arrow (protected-only phasing).

Several respondents stated that driver expectancy weighs heavily in favor of leading left turns. One West Coast engineer has found that driver confusion over lagging left turns results in start-up losses. He resorts to lagging left turns "only when benefits are very visible." Lagging left turns were not popular with many respondents and were used only when necessary and safe.

One East Coast consultant stated that he would consider the use of lagging only if it avoided the left-turn trap by a protected-only design. He believes that a driver-expectancy problem may exist when phase-sequencing is changed by time of day to obtain a better bandwidth.

Another East Coast consultant felt that it may be unsafe to change phase sequencing by time of day (or from one timing plan to another). This consultant is well aware of the left-turn-trap problem accompanying some lag solutions; he dislikes the inefficiency of using dual lagging left turns to avoid the problem, because both left-turn phases will be given the same time regardless of relative volumes. He noted that often the lag phase must operate on maximum recall if the through-band width depends on a fully timed left-turn phase.

A third East Coast consultant favors a leading left-turn arrow when both left turns and opposing through movements are heavy. When lefts and opposing throughs are both light, lagging is preferred. A combination of lead-lag is selected when there are heavy left turns with heavy opposing through movements and coordination is crucial. This strategy "eliminates the option for permissive protection for leading lefts and therefore reduces flexibility for reversing lead/lag."

TxDOT is a heavy user of PASSER and uses both phasings extensively. The respondent did not discuss the left-turn-trap problem, but reported that no difficulties have been encountered.

EXPERIENCE WITH COMPUTER PROGRAMS

Question 17 asked, "What has been your experience with computer programs?" Answers varied widely; for every respondent who liked computer programs because they "saved time," there was another who complained that so many data were required that he "didn't have time" to use them. The respondent from Washington, D.C., stated flatly: "We believe that even finely calibrated computer programs cannot match the work of a traffic engineer." At the other extreme, a number of respondents had in-depth experience with most of the widely distributed programs and simply reported overall satisfaction with them all. The respondent from Lakewood, Colorado, made this response:

> Nearly all our timing plans are generated using microcomputer-based programs (TRANSYT-7F, PASSER II-84, Intercalc, etc.). Rarely do we use manual techniques. Our arterial streets with multiple phasings, pedestrian and vehicle timings, actuated control, varying intersection geometrics, intersections at freeway interchanges, irregular spacings, and generally oversaturated conditions in the peak hours are much too complicated to be done with manual techniques.
>
> We have used the TRANSYT-7F microcomputer version for nearly four years and have been very pleased with the results on our multiphase controlled arterial streets. TRANSYT-7F does a good job of incorporating high volumes of turning traffic into the arterial flow-profile displays. These flow profiles also predict queue build-up which can point to offset and split adjustments to alleviate some of this problem. Multiple phases can be incorporated into the TRANSYT-7F time-space diagrams. These will assist in the decision of lead/lag operations to improve bandwidth. Since nearly all our signals are actuated, the offset (green return) time does vary. However, our oversaturated conditions operate with regularity and cause a return to offset during rush-hour conditions.

Some respondents stated that computer programs are a "good place to start," particularly for the inexperienced person. However, such a user may allow major errors to slip by for lack of an adequate check of the reality of the output. "Too many times we see someone code TRANSYT, and when it runs without an error, feel that he is done." Therefore, the computer user needs to have a perspective that apparently can come only from "hands on" field experience in timing. Computer usage needs to be handled with care; when it is, it can be extremely useful. It is imperative to inspect the results for reasonableness and to interpret them using good engineering judgment.

It was widely reported that several computer runs are usually required before the user can be confident enough with the results to put them on the street. A significant amount of field work is always needed to optimize the computer-generated settings.

PASSER II (84) is widely understood and liked by many users. It is considered the best for arterials, considering especially its reasonable requirements for input data. PASSER solutions are seen as more acceptable from the driver's point of view than are TRANSYT's.

TRANSYT-7F does not yet enjoy the acceptance given to PASSER II (84), partially because of its complexity and appetite for data. On the other hand, some users see a great deal of potential in this program. At least one other user has been "burned" by TRANSYT's lack of priority to arterial through movements; a report from one traffic engineer follows:

> We had an arterial system of over 20 signalized intersections that had been timed using traditional manual methods (paper strips

> for time-space diagrams, etc., plus field adjustments of splits and offsets). The system was set up to optimize arterial bandwidths, and had been "fine-tuned" to produce what we thought was the best possible traffic flow on the arterial. As an experiment we used TRANSYT to produce cycles, splits, and offsets that would *theoretically* optimize stops and delays. We installed the TRANSYT-produced settings in the field and, even after several attempts to refine the settings based on field observations, the result was a disaster. What had been a non-stop "green wave" through the 20 intersections was transformed by TRANSYT into a series of starts and stops for traffic on the arterial that made no sense. The citizens who drove this route daily became infuriated! After several weeks of trying unsuccessfully to refine what TRANSYT had produced, we went back to our previous settings and the result was a restoration of smooth, well coordinated traffic flow on the arterial. In my opinion TRANSYT or any other program that does not optimize bandwidth is totally inappropriate for arterial roads.

Users who reported good experience with both PASSER and TRANSYT often owe their success to a judicious application of each. Currently it is axiomatic that "PASSER is used for arterials and TRANSYT for grids." Says one respondent:

> It is often appropriate to use both programs to analyze one network. For example, a downtown grid may have a single arterial which crosses the network. It may be advisable to analyze the arterial with PASSER, fix the signal timings, and then optimize the remainder of the network with TRANSYT.

Question 18 was related to the respondents' experience with computer programs and asked if they had done any comparison studies. A few answered that they compared outputs from PASSER and TRANSYT. Those who have been active in California's FETSIM program, described earlier, have performed before-and-after comparisons using TRANSYT's simulation model.

Toronto provided reports describing in detail the procedures followed and the results produced by its Traffic Signal Timings Upgrade Team (*42*). Field measurements of speeds, stops, and delay showed large benefits, compared with costs, on a downtown one-way pair, a suburban arterial commuter route, and two commercial arterials.

TxDOT has performed comparison studies. Its respondent stated:

> Most models have their own niche where they are most effective. In the areas where they overlap, most of the differences between them are not significant compared to the day-to-day variations in traffic flow. Generally, results from different models should not be used in making comparisons between different designs.

Texas provided a report documenting large benefits from the use of the PASSER III program to time six full-actuated diamond-interchange controllers installed on an arterial with a median so wide that each intersection had to be treated as a diamond (*43*).

ARTFUL FINE-TUNING OF SOLUTIONS

Questions 19 through 24 asked if the respondents do any "artful" fine-tuning of manual or computer-based solutions to account for certain specified situations. Virtually everyone fine-tunes in the field; the question was seeking specifics of field or office procedures.

The specified situations were:

- *Excessive queue buildup threatening upstream intersections.* One consultant allocates weighting factors on stops on TRANSYT links. He says he hopes that future TRANSYT versions will use an optimization process that looks at link-queuing capacity directly on critical links. (Release 6 of TRANSYT-7F does this.)
- *Traffic turning into the system from a cross-street.* TRANSYT generally models this, but field fine-tuning is sometimes necessary. Some respondents manually adjust the start of the green to clear the queued vehicles before the upstream platoon arrives.
- *Queue buildup downstream, requiring early release.* TRANSYT normally allows for downstream queue dispersal if link travel speeds are input accurately. Some respondents manually adjust the start of the green to clear the queued vehicles before the upstream platoon arrives. PASSER's "Queue Clearance Factor" allows the downstream queue to get under way up to 10 sec before the arrival of the through band.
- *Multiphase intersections in two-phase time-space diagram.* Toronto codes multiphase intersections as two-phase in the time-space diagram.
- *Actuated controllers used with a "fixed" time-space diagram.* Progression should be based on the average duration of actuated phases.

Toronto mixes the two types of controllers routinely: "All semi-actuated intersections have background cycles; the time-space diagram is adjusted so that if any stops on the arterial are inevitable they are planned to occur at these intersections. Hence, if no call is made, there will be no stop."

- *Other.* It is common to adjust offsets where actual speeds turn out to be different from those anticipated. Texas also adjusts for pedestrians, bay length, and "many other factors." One respondent mentioned adjusting very closely spaced intersections to operate in a simultaneous pattern. A consultant well versed in the TRANSYT program attempts to give priority to through traffic from the upstream intersection (as opposed to turning traffic) by the use of shared stoplines and weighting factors. Another consultant adjusts phase times to accommodate platoons and not stragglers.

Some respondents use data from their system sensors to supplement their floating-car field checks.

One city traffic engineer said, "I do virtually no "paper' fine-tuning, believing instead in using field conditions rather than staff memory and office estimates." He explained the details of his field work as follows:

> I use techniques applicable to the UTCS-based systems where radio communication can be maintained between the system operator and field personnel. Computer-model output, checked and polished, is implemented. A driver and an observer ride through the arterial/grid and call in changes in offset and split to the system operator. Changes are implemented immediately, and after two or three cycles, the system is resynched and ready to be driven again. Two-way arterials can be completely checked within a three-hour period using "real-life" traffic volumes, not projections or smoothed estimates.

For computer solutions, the respondents were asked if they do iterations in an attempt to make it more than just a coding exercise. Most replies were strongly positive. Toronto's practice is to use different cycle lengths within PASSER II(84), put

the resulting offsets into a separate program PROG that plots time-space diagrams on a line printer, and then adjust by hand. Others mentioned supplementing their iterations with floating-car checks. One consultant said that PASSER tends to give excessively long cycle lengths (greater than 150 sec) to maximize progression; he added, "You have to find that fine line between great progression and too much side-street delay."

One traffic engineer does no iterations: "However, output graphs and tables are thoroughly checked, by a technician and then by me, for anything unusual or conflicting with our intuitions." If the output does not pass this check, then another run is ordered: "We ran the 69-signal downtown grid five or six times before we were satisfied with the solution."

As explained earlier, Lakewood, Colorado, makes maximum use of TRANSYT-7F. A series of runs is made to determine an optimum timing plan, considering delay, stops, fuel consumption, and performance index criteria:

> The TRANSYT-7F optimization runs are further analyzed using the flow profiles, time-space diagrams, and platoon-progression diagrams. These outputs are closely evaluated for offset and split adjustments and to identify the potential for oversaturated conditions. Other outputs that are considered in the performance of each optimization plan include the "degree of saturation" and "maximum back of queue" calculations shown in the performance table.
>
> Finally, the evaluation results are used with engineering judgment to determine a preferred timing plan. Minor adjustments are made to splits and offsets on paper to conform to known street conditions (e.g., hills, early release, oversaturated conditions). Most of these adjustments are made based on the accepted goal of moving the major-street traffic as efficiently as possible. This goal does result in some delay to minor movements including side street and signalized left turns. However, we have reached such oversaturated conditions on two of our arterials that we must use long cycle lengths (up to 136 seconds) to move the large arterial volumes. The major-street emphasis is then a reasonable and achievable goal.

Lakewood has a system that includes a "split monitor" to assist timing evaluation and fine-tuning. The split monitor measures and reports the amount of green time being used by each phase for each cycle based on detector inputs. This indicates how well the selected splits are being used. It shows if any intersection is actually timing out on all approaches, if pedestrian time is being used, or if any approach appears to be oversaturated. The respondent added:

> Next, these splits are used with the existing cycle length and offsets as inputs to TRANSYT-7F for a simulation analysis to determine a base for optimization-run comparisons. While this process is occurring, travel time and delay studies are performed in the corridor using the floating-car technique. This data is compiled on a spreadsheet to be used in a "before-after" type analysis.

The selected timing plan is then put on the street and there is a field review that includes driving the route and observing individual intersections:

> From this review, adjustments to offsets are sometimes made to account for early release at intersections with relatively undersaturated conditions.
>
> In order to close out the process, the split monitor routine will again be run for selected locations to determine effectiveness of the selected splits. Then, the "after" phase of the travel time and delay study is performed to measure the results of the changes.

One consulting firm that uses both PASSER and TRANSYT routinely has organized the iterations as follows:

> With PASSER, we typically split a long arterial into two or more shorter sections at the same cycle length and then manually tie the two solutions together. We will make several runs with different boundaries between sections. For arterials, we use PASSER and TRANSYT for comparison, and sometimes code the results of PASSER into TRANSYT for evaluation and optimization. With TRANSYT, we make a number of runs using various ranges of cycle length, sometimes with changes in boundaries. Whether arterials or grids, we always conduct multiple runs to determine the best cycle length and performance data. The first successful run does not finish the work.

TIMING FOR SYSTEMS WITH OVERSATURATED INTERSECTIONS

Many respondents drew no distinction between single oversaturated intersections and those in systems; their opinions on oversaturation are described in Chapter Two.

One consultant drew a distinction, as follows:

> Coordination through an oversaturated intersection is nice on paper but has no practical use in the real world. Therefore, I try to provide good throughbands for platoons being released from oversaturated intersections, but do not try to time greenbands into them or through them.
>
> Also, if the long cycle length that would be reasonable to use at a single oversaturated intersection is too high for the adjacent intersections, I would run the oversaturated intersection in an isolated manner and set up shorter cycle-length patterns for the rest of the signals in the adjacent control sections.

Several respondents stressed the need to abandon any attempt to provide progression through an oversaturated intersection. As the respondent from Toronto put it, "If the intersection is over-capacity, then it is left out of the coordination exercise." A consultant went into more detail on this point:

> We attempt to isolate an oversaturated intersection from the remainder of the network to the extent possible, such as by using a different (higher) cycle length, as progression through an oversaturated intersection is meaningless anyway. For groups of consecutive oversaturated intersections we manually set up timings to ensure that the downstream block has at least some holding capacity before we release the upstream through traffic.

These concepts of metering traffic into the bottleneck and progressing traffic away from it are shared by the respondent from Montgomery County, Maryland:

> The goal is to *equalize delays* on each of the major approaches to the oversaturated intersections. . . .Then, the offsets of the downstream intersections are set to progress the major flows out of the oversaturated intersection. If possible, intersections upstream from the oversaturated location are provided with offsets that will tend to meter flow. Sometimes that flow is purposely metered by artificial phasing constraints (keeping a through phase red while its associated left-turn phase is green, even though there's no opposing left turn).

The oversaturated conditions in Lakewood, Colorado, tend to result in longer cycle lengths. Its most congested arterial requires cycles from 85 to 135 sec throughout the day:

> We have found that the public accepts the fact that they may have to wait at *one* location in the rush hour, but once they begin to move, can do so unimpeded for several signals before they must stop again. This operation appears to be more acceptable than stopping and starting along an arterial several times with relatively shorter cycle lengths. We have also found that the addition of double left turns on our arterials and connecting side streets improves capacity and can result in shorter cycle lengths even if additional through lanes are not added to the arterial section (*22*).

An oversaturated intersection can be selected for "critical intersection control" (CIC). Detectors are installed on all approaches to help tailor the split on a cycle-by-cycle basis.

Computer programs such as TRANSYT are not designed for oversaturated conditions, but TRANSYT timings are considered by some to be the best place to start field fine-tuning. One consultant has found that TRANSYT appears to be more sensitive to oversaturated conditions than is PASSER. The North Carolina DOT has found that TRANSYT-7F does not give good results at oversaturated intersections; if it is necessary to break a system into subsystems, then the oversaturated intersections are used as breakpoints.

The National Cooperative Highway Research Program is conducting a study on signal control at oversaturated intersections. Among the many topics addressed are strategies for metering traffic into the bottleneck and the use and value of upstream and downstream detectors (*22*).

PROCEDURES TO TIME SPECIFIC TYPES OF INTERSECTIONS

Questions 27 through 32 asked if the respondents have different procedures to time suburban arterials, commercial arterials, collectors/mixed land use, downtown grids, freeway interchanges (e.g., diamonds), and two arterials crossing in a suburban-type grid.

Regarding the first three types, the respondent from Montgomery County, Maryland, stated:

> All arterials and major collectors are systemized with the same basic philosophy—provide optimum green band for the arterial; favor the arterial over the side streets; if delays are necessary, make them occur on the side-street phases; but once motorists get onto the arterial give them good, smooth, uninterrupted flow (if possible).

Texas uses PASSER II on all arterials, adjusting for the appropriate speed. If pedestrians are significant, then adjustments for the right turns are made as shown in the 1985 *Highway Capacity Manual* (*4*).

For arterials or collectors of any of the three types, it is common to use PASSER II to evaluate cycle length and phase sequence and then to use TRANSYT for the final runs. One consultant would not time these three types differently based on classification, but rather on traffic conditions:

> For an arterial in which there are no substantial turning movements or high through volumes, I would use PASSER for best progression. Otherwise I would use TRANSYT, weighting the preferential links with the stop-penalty factor.

This consulting firm commonly uses TRANSYT's stop and delay penalty cards (37 and 38) to emphasize the critical links and nodes in an attempt to provide preferential treatment when a dominant traffic flow exists. It reports good experience in obtaining arterial progression from TRANSYT in this way. [On the other hand, Moskaluk (*44*) found that these penalties do not reliably produce the desired preferential treatment.]

A California city traffic engineer handles the three different arterial/collector situations by using TRANSYT with different platoon dispersion factors.

For downtown grids, many respondents cited the need for fixed-time control. The respondent from Texas and many others use TRANSYT-7F to time a grid configuration.

The respondent from Montgomery County, Maryland, does not use TRANSYT for grids but instead follows this procedure:

> In downtown grids, the streets with the heaviest volumes get the priority for progressive movement. Starting with the oversaturated intersections (if any), we then look at the major street with the highest volume and optimize progression on that street. Next we look at the most major of the streets that cross the highest-volume street and, using that intersection as the determining factor, develop a time-space diagram for that crossing street. This process continues until all major traffic flow patterns in the grid have been analyzed and the best possible progression patterns for those movements, within the constraints, have been set up. The grid pattern naturally results in less-than-optimum progression for some movements.

At diamond interchanges many cited the need to provide sufficient ramp timing to avoid backup onto the freeway. Texas uses PASSER III to time its diamonds: "If we are tying the diamond into an arterial, then we start at the diamond and work outward." A California city engineer uses a Texas "four-phase leading" scheme with two overlaps during high-volume periods and free operation during times of low flow. This four-phase scheme was popular with other respondents; basically, it recognizes that there are four approaches entering the interchange, serves them sequentially, and allows each to clear the interchange without a stop inside it. Another Texas scheme using the "three-phase lag-lag" pattern was mentioned as an alternative. Several respondents use a diamond-interchange program that has been developed for their Type 170 controllers.

The respondent from Montgomery County, Maryland, stressed the importance of maintaining coordination through the diamond interchange:

> We attempt to incorporate the signals at freeway ramp terminals into the arterial signal systems. Turning movements at ramp terminals are often greater than at typical "side street" intersections, but usually they are not high enough to require the ramp intersections to be divorced from the adjacent arterial system. A special full-actuated "diamond interchange" controller, running in isolated mode, is automatically a *disruption* to otherwise smooth green-band progression along the arterial. Except in unusual cases, we prefer to use three-phase semi-actuated *coordinated* control at both of the ramp terminals of a diamond interchange and maintain progressive flow along the arterial through the interchange.

One consultant uses PASSER III for determining phase sequencing and timing at a diamond interchange. Then he models the interchange as a single intersection in PASSER II or else "freezes" the relationship between the two interchange signals in TRANSYT-7F.

For two arterials crossing in a suburban-type grid, Texas uses PASSER II on each arterial: "We fit the minor streets in as best

we can." Other responses included the need to run both arterials on a common cycle length and to fix the offset of one with respect to the other by "meshing" the two timing plans. That is, PASSER is first run for the predominant arterial to obtain a cycle length and split for the common intersection. These results are then locked in to the minimums for the PASSER run for the minor arterial. If the cycle length for the predominant arterial is too long to be used for the minor arterial, then this intersection will become a "break point" or "planned stop" for it. Sometimes a compromise cycle length can be obtained by examining a range of cycle lengths acceptable to both arterials. This intersection in common is the starting point for selecting offsets at adjacent intersections in all four directions.

The solution will change if the minor arterial becomes the predominant one at certain times of day or days of week.

If the intersection of the two arterials is operated under a central UTC system and oversaturation is present, then CIC would be considered. Most CIC schemes simply allow the intersection to run "free" under isolated full-actuated control; there is no progression on either arterial (oversaturation prevents that), but split can change on a cycle-by-cycle basis.

FIELD FINE-TUNING

Responses varied widely to the question "What percent of a timing project is devoted to field fine-tuning?" Estimates varied from 1 to 75 percent, and did not seem to depend on whether the respondent was a consultant, a local-government engineer, or one from a state agency. Some of those estimating 5 to 10 percent indicated that they would do more if they had the personnel. Those estimating 50 percent or more invariably leaned toward manual solutions in Question 36. The responses as a whole suggested that field fine-tuning is considered to warrant at least 25 percent of the engineering effort (i.e., data collection excluded) and could increase because of the complexity of field conditions such as the impact of side-street and mid-block volumes, volume/capacity ratios, and accuracy and extent of required input data.

PREFERENCE FOR MANUAL OR COMPUTER METHODS

The local-government engineer who devotes the "vast majority" of a signal-timing project to field fine-tuning had this to say in response to Question 36 about his preference for manual or computer methods:

> I have not seen a computer program yet that can time a signal or a system as well as a trained and experienced engineer or technician who actually observes traffic flow at the intersection. I can see the value of a computer program that could automate some of the more repetitive tasks of doing a manual time-space diagram, and I think such programs are now becoming available. Perhaps "expert systems" of the future could be of some assistance.

On the other hand, an engineer in charge of the signal timing for a large Southeast city answered, "There is no question that manual methods can no longer be considered the primary traffic signal coordinating tools." Another city engineer from the Rocky Mountain area was even more enthusiastic: "Computers. By all means."

Most respondents qualified their answers. Some lean toward manual methods for undersaturated arterials but are ready to move to TRANSYT-7F as flows approach capacity or for grids (if the required data can be obtained).

A state government engineer sees advantages in both methods: "Manual methods are more flexible and allow more "judgment' factors to be used. Computer methods are faster and allow more iterations." Time savings and the ability to allow more possibilities to be considered were mentioned by others as tipping the balance in favor of computer methods. They give good starting values but must be adjusted manually and then field fine-tuned by experienced personnel. In the conclusion of one respondent: "The computer is an essential middle step, coming after data collecting and organizing, and before adjusting, coding, installing and fine-tuning."

MEASURES OF EFFECTIVENESS

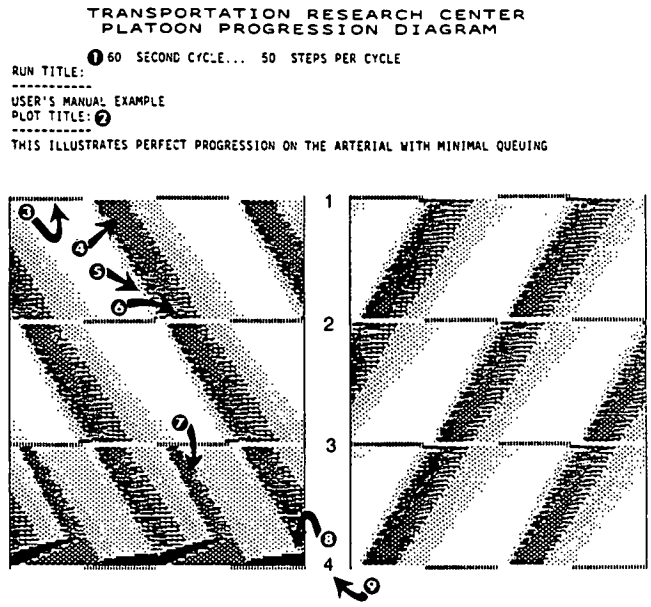
Measures of effectiveness in coordinated signal systems include those just mentioned plus others relating to the quality of progressive movement. In the United States, timings derived by hand involve the construction of time-space diagrams, as explained in many sources (e.g., *13*). The objective is to maximize the widths of through bands that show graphically the trajectories in time and space of vehicles driving the length of the route at a certain speed without stopping. The through band expressed as a decimal fraction of the cycle length is the "efficiency" of that band.

Computer programs for timing coordinated systems provide many more measures of effectiveness. TRANSYT-7F seeks to minimize a weighted combination of stops and delay and, optionally, queue. Together these make up a performance index that is reported as a part of the output. Other measures estimated by the program include degree of saturation in percent, queuing, fuel consumption, total distance traveled in vehicle-miles/hr, total travel time in vehicle-hours/hr, average overall travel speed, and total operating cost.

Computers excel at generating numerical output, challenging the traffic engineer to interpret and evaluate so much data. Recent contributions from Courage and Wallace of the University of Florida are doing much to help in this area. One is the platoon progression diagram (PPD) (*2*), intended to combine the best features of the two plots that are output by TRANSYT-7F itself (i.e., the flow profile diagrams and the time-space diagram). The density of printed dots represents traffic density (see Figure 18). The PPD is excellent in showing platoon dispersion and the potentially adverse effects of queues. Although the PPD can be obtained only for a linear route (i.e, no turns), it will show platoons resulting from traffic turning into the artery from a cross-street. The PPD shows at a glance the effectiveness of an arterial timing plan.

Courage and Wallace have also developed an animated graphics program to show the effectiveness of network timings derived from TRANSYT-7F or any other source. Named Signal Network Animated Graphics, the program displays a map of the network with moving green bands of traffic platoons.

As signal timing becomes more of a science, it is becoming more difficult to visualize the significance of extensive computer



1. The cycle length and step size are TRANSYT inputs.
2. The titles are also entered into TRANSYT.
3. Signal status (red or green) is shown just like a time-space diagram.
4. The platoon is saturated as it leaves the intersection.
5. The platoon spreads out as it progresses down the street.
6. A small queue builds up here because a few vehicles have been stopped on red.
7. This platoon results from the turning traffic entering the link on the cross-street green.
8. This large queue builds up because the link volumes are near capacity.
9. The node numbers are printed if the number of steps is less than 60.

FIGURE 18 TRANSYT's platoon progression diagram.

output; in the future there will surely be increasing reliance on animated graphics to show the effectiveness of timing plans for both arterials and networks.

PROBLEMS REMAINING UNSOLVED

Respondents listed many types of problems that they feel are still unsolved in the timing and operation of coordinated signal systems. Some echoed their responses to Question 9, unsolved problems at single intersections. Some problems mentioned are not likely to be solved soon, whereas others may already have solutions that are not widely known.

Strategies to handle oversaturation, such as computer control of queue lengths, ranked high in the responses.

Several engineers complained of budget limitations that prevent engineers from keeping their timing plans optimized. The respondent from the state of Delaware put it this way: "A system which allows recalculation of signal timing based upon measured data without significant operator input would be helpful. An A.I. [artificial intelligence] system which learns and optimizes each day of the year and selects it next year would be great." The respondent from Texas agreed: "Data collection is still a big concern. We are developing a 1.5 generation system for our Flexible Advanced Computer Traffic Signal (FACTS) System and this will be a big help in pattern development."

The difficulty of controlling the start of main-phase green in an actuated system was mentioned. Early release of a main-street platoon, caused by light side-street demand, produces unnecessary stops and delay. When high-speed arterials are operated as the coordinated phases of a system, there is no option-zone protection for high-speed vehicles traveling behind the through-band.

Reliable detection is needed. The one respondent who mentioned this did not describe his materials and procedures. Loop-detector installation has advanced to the point at which embedded wire seldom fails unless broken out by road equipment (*45*, p. 31).

Many improvements to TRANSYT were cited and several were incorporated into Release 7, including:

- A new TRANSYT-7F executive menu and file processing for a microcomputer version, called McT7F
- Explicit optimization of progression opportunities (PROS)
- New split algorithm, based on degrees of saturation
- Revised stops algorithm, for better accuracy near or at saturation
- Revised handling of permitted turning for sign-controlled simulations

- Revised optimization algorthim to improve performance
- Replacement of the random delay estimate with the one from the HCM

Other suggestions include:

- TRANSYT needs better integration of semi-actuated controllers. In particular, it needs the capability to determine the operation of a dual-ring controller automatically, instead of assuming a fixed six-phase operation.
- TRANSYT needs to model the effects of queuing on upstream intersections.

Time-space diagrams from both PASSER and TRANSYT are printed on dot-matrix printers as text files, not graphics plots. This removes the burden on the programmer to support a host of graphics printers but leaves the user with a time-space diagram lacking in adequate resolution. Also, a means of manually adjusting time-space plots is needed so that hand adjustments and results of fine-tuning can be documented on an attractive, easy-to-read plot instead of one that is crossed out and changed. The platoon progression diagram partially responds to this need. There is also a proprietary program available that provides automated plots.

"Bandwidth" programs such as PASSER need to be able to adjust noncritical intersections to provide large "internal" bands; they also need the ability to sacrifice some bandwidth for better handling of very closely spaced intersections.

CHAPTER FOUR

FLASHING OPERATION AND TIMING FOR ADVERSE WEATHER

This chapter discusses two topics that, although not central to the topic of signal timing, are too important to be overlooked in this synthesis. Modern traffic signal equipment is sensitive to electrical disturbances of various kinds and can be triggered to the flashing mode by the conflict monitor rather easily. Newer models of conflict monitors can identify more types of malfunctions than before, so it can be expected that signals will be going into the conflict-flash mode more often in the future. They may remain in flash longer, because controller units are not as interchangeable as they used to be. This means that traffic engineers need to rethink their approach to the flashing mode, with more attention paid to ensuring reasonably safe operation while the motorists are proceeding on a see-and-be-seen basis.

Inclement weather can greatly affect highway conditions and interfere with normal vehicle operation. The extent to which signal timing can and should adapt to degraded conditions is also explored in this chapter.

FLASHING OPERATION

The MUTCD (*1*) requires that "when a signal is put on flashing operation, normally a yellow indication should be used for the major street and a red indication for the other approaches."

Flashing Yellow/Red Operation

A discussion of flashing operation in the TCDH (*24*) includes this statement:

> Flashing yellow/red operation may be appropriate at simple, four-legged or three-legged, intersections where the minor street drivers have an unrestricted view of approaching main street traffic, and the traffic volumes are low.

This statement may have been directed primarily to the decision whether to go to flash intentionally at certain times (such as at night). However, flashing operation is also used as the fallback level of operation when the signal malfunctions.

AASHTO Case III Sight Distance and Flashing Red/Yellow Operation

AASHTO standards include guidelines for four types of control that apply to at-grade intersections (*46*, p. 760). The third type—Case III—pertains to minor-road traffic waiting at a STOP sign either to cross or to turn in to a major highway. The waiting driver "must have sufficient sight distance for a safe departure from the stopped position even though the approaching vehicle comes in view as the stopped vehicle begins its departure movements." AASHTO provides various graphs and equations that permit a check of sight distance over a range of conditions. These can be used to check an existing intersection for sight distance under flashing yellow/red operation.

Flashing yellow/red operation can be hazardous at locations where major-street flow is so heavy that few gaps for crossing traffic exist, and/or where sight-distance problems make it hazardous to operate the intersection on a see-and-be-seen basis. The current AASHTO "Green Book" (*46*) warns of the problem:

> The hazard associated with unanticipated vehicle conflicts at signalized intersections, such as violation of the signal, right turns on red, malfunction of the signal, or use of flashing red/yellow mode, further substantiate the need for incorporation of Case III sight distance even at signal-controlled intersections.

The AASHTO "Green Book" pertains to the design of new and major reconstruction projects. The quotation does not mean that sight distance must be provided at existing intersections. It does point out that a signal-controlled intersection lacking Case III sight distance may be hazardous during flashing red/yellow operation.

Need for Intersection Evaluations

The traffic engineer should consider evaluating the use of flashing yellow/red operation at all signalized intersections. In some cases all-way flashing red may be an alternative that could be selected at intersections that do not meet AASHTO requirements for Case III sight distance. When all-way flashing red at such locations may be undesirable for the major street operation, perhaps because of heavy volumes, a reasonably safe solution should be sought. For example, the signal status could be monitored continuously by a master located at curbside or at the traffic operations center. When flashing operation is detected, a traffic-control police officer (and the signal-repair crew) could be dispatched to the intersection. Another possible solution could be to use a flashing yellow indication on the major street; other approaches not having safe sight distance conceivably could have their flashing red indication supplemented by a blank-out sign message such as RIGHT TURN ONLY. There could be a public information program encouraging motorists facing a flashing red to edge out carefully and then turn right if traffic is heavy or if they cannot see very far.

PRINCIPLES OF TIMING FOR ADVERSE WEATHER

Roadways made slippery by precipitation are the principal focus of this section. Under these conditions drivers tend to start

up on a green signal more slowly than on a dry road, to avoid slipping. When queued vehicles start up in such adverse conditions, the rate of discharge is sluggish, as under congested conditions. When roads are covered by snow or ice, it is desirable that signals be timed to minimize stops at intersections with significant approach grades. The need is to reduce the number of vehicles sliding through an intersection or rear-ending others on downgrades, and to keep vehicles in motion on slippery upgrades. Coordinated signals under central control can appropriately be set for a heavy-traffic offset, typically the "simultaneous" pattern in which all intersections along the route begin green at the same time. As an example of how this condition can operate effectively, the author was told many years ago, before computer control, of a Midwest city that had several fire runs, each set up to go to simultaneous offsets upon command from city hall or a fire station. On snowy days the traffic personnel would repeatedly push the fire-run buttons, creating simultaneous offsets for long periods of time.

Actuated controllers conforming to the NEMA standard have an available "per ring" external command function to "omit all red." It is intended that the command be asserted under normal weather conditions, when the traffic engineer might feel that no all-red clearance is needed at this particular location. In bad weather the command is dropped, thereby adding an all-red clearance interval intended to allow for longer stopping distances and times. The arrangement is "fail-safe" in that the all-red interval will automatically be invoked in the event of failure of the master computer or the communications. Any decision to omit an all-red clearance should consider the points made in Chapter Two under the heading Phase-Change Intervals.

Timing for snowstorms, hurricanes, etc., clearly is not a high-priority consideration for those who responded to the questionnaire. Very few have implemented anything for this contingency, but several had ideas to offer.

The city of Tulsa, Oklahoma, manually switches its downtown grid to "snow pattern" during winter storms. This pattern uses off-peak splits and offsets but has a longer cycle length. Lakewood, Colorado, uses its master computer to assert recall on all phases during snowy conditions. The lane markings are not visible, so the vehicles may not cross the loops: "In heavier snow storms street crews try to clear one through lane, which may be between loop locations." Other respondents suggested that severe-weather timing feature lower progression speeds, longer cycles, longer minimum greens and passage times for actuated phases, longer clearance time, or flashing operation.

Broward County (Ft. Lauderdale), Florida, flashes selected signals when evacuating the beach during hurricane warnings.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

During the 1980s there were many large-scale signal retiming projects in the United States, funded primarily by petroleum overcharge rebates to the states. An example project, California's FETSIM program, has been shown to be extremely cost-effective. Avoided fuel expenditures in the first year after retiming have amounted to four times program costs. If benefits are broadened to include savings in motorist travel time and vehicle wear and tear, a 16:1 first-year benefit-to-cost ratio results.

One of the principal reasons for preparing a synthesis of current practice is to determine what problems remain largely unsolved. The respondents to the questionnaire listed a number of these, both for single intersections and for coordinated systems. Those responses are described in detail in the body of the synthesis and are summarized here, with recommendations. Other problems previously noted are also identified.

- *Need for adequate funding for retiming.* Retiming an intersection by conventional methods requires approximately one person-week of effort and is needed every year in high-growth areas. (Perhaps every two to three years is adequate in more stable localities.) It remains to be seen whether state and local governments will continue to retime regularly after the petroleum-rebate funds have been depleted. Traffic engineers need to communicate aggressively to the public and elected leaders the overwhelming cost-effectiveness of regular retiming (and of prompt maintenance of equipment and replacement of old hardware).
- *Need for uniform timing practices and procedures.* There is a need for research to transform the area into less of an art and more of a science. Although computer programs such as PASSER, SOAP, and TRANSYT-7F have been available for years, there remain many competent engineers who shy away from them in preference to manual methods and/or field observation. In the early to mid 1980s many governments found it necessary to do their computer work through consultants. This, however, changed in the late 1980s, because by then even the TRANSYT program could be run easily on inexpensive desk-top computers. Recent graduates of programs in transportation engineering are very comfortable with a variety of computer programs; these young engineers will no doubt want to use the computer as a "what if" tool to examine a range of options. No matter whether office methods are manual or computer-assisted, they will be no more than starting points for the field fine-tuning that is always needed.
- *Need for enhancements to the TRANSYT-7F program.* TRANSYT is seen by many as having enormous potential, not only for grids but also for arterials. Funds need to be made available to further improve the program in three major areas: better integration of actuated controllers, improved modeling of oversaturated approaches, and simulation testing to compare existing procedures to improve arterial progression. Some of these concerns are actively being addressed, and major improvements were incorporated into Release 7.
- *Need for cost-effective field data-gathering procedures.* Rather than merely hope for more funds, the traffic engineer needs to push for the development of cheaper ways to obtain the data on volumes and other factors needed for the retiming effort. Despite the development of microprocessor-based, hand-held count boards and lap-top computers, field data gathering remains labor intensive and is still oriented primarily to pencil and paper. Updated "1.5 generation" systems that will automate data gathering and timing recalculation are sorely needed. As long as loop detectors continue to be required for automated data gathering, 1.5 generation will require adequately installed and maintained loop detectors.
- *Need for strategies to handle oversaturation.* Although the respondents are experienced in "spreading the misery around the intersection" in the least damaging way, there seems to be no agreement on selection of cycle length. It appears that for every traffic engineer bent on minimizing the cycle length there is another pushing it to the limit that motorists will tolerate before running the red. If a literature review does not uncover a good answer to this, then research using a microscopic simulation program such as TRAF-NETSIM will be needed. The Transportation Research Board's current NCHRP Project 3-38(4), "Traffic Signal Control for Saturated Conditions" (*22*), may give insight into this difficult problem.
- *Need for proper management of the pedestrian situation at signalized intersections.* Traffic engineers need more guidance in the selection of the length of the WALK interval to meet conflicting goals. On the one hand, arterial congestion influences the traffic engineer to minimize pedestrian WALK timing at 4 sec. On the other hand, some respondents reported the need for much longer WALK time, so that the pedestrian reaches the middle of the street by the time the flashing DON'T WALK begins.
- *Need for research regarding safe timing of the yellow and all-red intervals.* Opinion within the profession varies regarding the timing of yellow and all-red phasing. There is a need for research on driver expectancy and safety implications of these types of timing to resolve the issue.
- *Need for guidance in setting the timing of "density" phases, especially time before reduction, time to reduce, and the maximum interval.* Included is a need for a strategy to attempt to assure "option-zone" protection when heavy traffic tends to extend the green to the maximum interval. Guidelines for the use of actuated "prepare to stop" could be a part of this strategy. Again, simulation studies with TRAF-NETSIM could answer these questions if support for such research were available.
- *Need to disseminate information to traffic engineers on the "left-turn trap" and its engineering solution.* Computer programs may recommend lagging left-turn phases in the interest of efficiency, but their documentation usually does not warn the inex-

perienced engineer of the need to avoid the trap. The documentation of TRANSYT-7F (*2*, pp. 2–18) does not address the issue of the left-turn trap nor does it mention its hazardous nature. Users should learn to use computer programs to hold or fix the phasing to disallow the trap. The point is that all of the programs need to be used with intelligence and good engineering judgment.

- *Need to disseminate information to traffic engineers on the hazards of flashing red/yellow operation at intersections lacking in Case III sight distance.* Traffic engineers should evaluate their intersections for this and consider the alternatives explained herein.
- *Need for a uniform public education program related to signalization.* A driver first in line when a green begins should know to look both ways before entering the intersection and to yield the green to any clearing vehicles. Proper respect for the yellow and the start of red should be taught in the schools and enforced vigorously. There should be stronger public information programs stressing the need for pedestrians to push the button and to obey the CROSS ON WALK SIGNAL ONLY signs. Flashing operation, in particular, is not well understood by drivers, who should be taught to consider turning right on a flashing red if traffic is heavy or sight distance is restricted. The "three E's" of Engineering, Education, and Enforcement all too often start and end with the first E; too great a burden is placed on the traffic engineer.

Many of these recommendations call for more engineering work, which requires both funding and public support for the operating agencies.

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APPENDIX A
SURVEY QUESTIONNAIRE AND RESPONDEES

This synthesis is based, in part, on the responses to a questionnaire sent to a number of traffic engineers considered knowledgeable about signal timing and optimization. This appendix includes a copy of the questionnaire. The selection and contacting of recipients are explained. There is a summary of the geographical distribution and the types of employment of the recipients and of those who responded. The details of the responses have been incorporated into the body of the synthesis.

The questionnaire was sent to 60 traffic engineers, each of whom met one or more of the following criteria:

- NCHRP Synthesis Topic Panel member
- Suggested by Topic Panel Member
- Member of TRB Committee on Traffic Control Devices
- Member of ITE Committee on Optimizing Traffic Signals
- Member of ITE Committee on Signal Timing
- Member of ITE Committee on Congestion Management
- Known to be knowledgeable in the field

Thirty-four of the 60 recipients made a substantial response. They were located in 19 states, including the large states of California, Texas, and New York. The breakdown by type of employment was as follows:

Employed By	Number of Recipients	Number of Respondees
Local government	18	11
State government	13	9
Federal government	3	2
Consultant	26	12

GEORGIA TECH / TRANSPORTATION RESEARCH BOARD

QUESTIONNAIRE ON SIGNAL TIMING AND OPTIMIZATION

Whom may we contact for more information on this filled-in questionnaire?
Name_____Phone_____

Can you send us any information on the following? **Please answer Yes, meaning you can provide something, or No, you can't, or write in a comment, and return this with any material you are able to furnish now. If you answer Yes for any item, and we don't receive anything enclosed with the questionnaire, or later (by June 1, say) we will phone a reminder.**

For Single Intersections

1. Examples of calculations of ped timing (with and without ped signals), clearance timing, cycle length, split, settings of actuated controllers_____

2. How do you choose among fixed-time, semi-actuated and full-actuated control at single intersections?_____

3. For actuated control, do you have different selections of controller (e.g. density) and loop layout for various conditions?_____

4. Have you used SOAP or any other computer program?_____Like it?_____

5. Have you made any studies to compare the results of SOAP (or other) with manual results or with field measurements of queue length or delay or other measures?_____

6. Do you do any "artful" fine tuning of manual or computer-based solutions, for example to reduce queue build-up in left-turn lanes?_____For other reasons?_____

7. For computer solutions, do you do iterations to attempt to make it more than just a coding exercise? _____

8. How do you time oversaturated intersections?_____

9. What problems remain largely unsolved in this area? _____

For Coordinated Systems

10. Examples of determinations of cycle, split and offset, including any time-space diagrams you develop_____

11. How do you choose among fixed-time and semi-actuated controllers in your systems?_____Do you ever mix them? _____

12. Do you ever use full-actuated controllers in systems?_____

13. Have you used PASSER II?_____ III?_____ TRANSYT?_____ NETSIM?_____

14. Do you use PASSER or other to evaluate alternative phasings?_____

15. When coding PASSER or TRANSYT, do you use ped minimum time for your side street minimum? _____Or do you use vehicle minimum time for that, because a ped call will preempt the intersection out of the normal system?_____

(OVER, PLEASE)

16. What is your position on leading vs. lagging left-turn arrows to optimize bandwidth?_____

17. What has been your experience with computer programs?_____

18. Does that experience include any comparison studies?_____

Do you do any "artful" fine tuning of manual or computer-based solutions, for example to account for the following:

19. Excesssive queue build-up threatening upstream intersections_____
20. Traffic turning into the system from a cross-street_____
21. Queue build-up downstream, requiring early release_____
22. Multiphase intersections in two-phase time-space diagram_____
23. Actuated controllers used with a "fixed" t-s diagram?_____
24. Other factors?_____

25. For computer solutions, do you do iterations in an attempt to make it more than just a coding exercise?_____

26. How do you time systems with oversaturated intersections?_____

Do you have different procedures to time the following?:

27,28 Suburban arterials_____ Commercial arterials_____
29,30 Collectors/mixed land use_____ Downtown grids_____
31. Freeway interchanges (such as diamonds)_____

32. How do you handle the coordination of two arterials crossing in a suburban-type grid?_____

33. What problems remain largely unsolved in this area?_____

<u>For Both Single Intersections and Systems</u>

34. What percent of a timing project is devoted to field fine tuning?_____

35. Would you use computers more if your office had more PCs?_____

36. Have you made a decision that manual methods are just as good or better?_____
That computer methods are better?_____

37. Do you have a procedure for timing signals for adverse weather?_____

<u>Signal Optimization</u>

38. Do you have any information indicating the impacts of good and bad signal timing?_____

39. How do you decide when a signal or system needs retiming?_____

40. Any data on the resources in personnel and dollars needed for retiming or detection of malfunctions or equipment upgrading?_____

Thank you for taking the time to read all this and consider helping!

Please return this in the enclosed envelope, along with whatever material you are able to furnish now. Please send any additional material by June 1 to:
Dr. Peter Parsonson, Civil Engrg., Georgia Tech, Atlanta GA 30332

APPENDIX B
DETECTOR/CONTROLLER CONFIGURATIONS FOR LOW AND HIGH APPROACH SPEEDS

(Text was developed by Peter Parsonson for NCHRP Project 3-27)

This section sumarizes the state of the art of detection design and location on approaches to signalized intersections and their appropriate use with different controller types. The material is organized under the following headings:

LOW-SPEED APPROACHES
- Controllers With Locking Detection Memory
  - Basic, Full-Actuated Controllers
  - Semi-Actuated Controllers
- Controllers With Nonlocking Detection Memory
- Application to Left-Turn Lanes
- Detection of Small Vehicles
- Loop-Length Design

HIGH-SPEED APPROACHES
- Controllers With Locking Detection Memory
  - Basic, Full-Actuated Controllers
  - Semi-Actuated Controllers
  - Volume-Density Controllers
- Controllers with Nonlocking Detection Memory
  - Basic, Full-Actuated Controllers
    - Extended-Call Design
    - EC-DC Design
  - Volume-Density Controllers

LOW-SPEED APPROACHES

Approaches experiencing speeds less than 35 mph (56 kph) are considered low-speed approaches. The design of the detection depends on whether the controller phase for that approach has been set by the traffic engineer to "locking" or "nonlocking" detection memory (sometimes termed "memory ON" or "memory OFF," respectively).

Controllers with Locking Detection Memory

The locking feature means that a vehicle call for the green is remembered or held by the controller, after the vehicle leaves the detection area, until it has been satisfied by the display of a green interval to that phase. Locking detection memory is associated with the use of small-area detection ("point" detection) such as a 6-ft-x-6-ft (2-m-x-2-m) loop. The advantage of this scheme, often termed "conventional control," is that detection cost is minimized. However, this type of control is incapable of screening out false calls for the green (such as occur with right turn on red). A report by the Southern Section, Institute of Transportation Engineers (*1*) and a related training film by the Georgia Institute of Technology (*2*) explain this type of control. The remainder of this section essentially is summarized from these two sources.

The two types of controllers appropriate for low-speed approaches and locking detection memory are the basic full-actuated and the semi-actuated types.

Basic Full-Actuated Controllers

A basic actuated controller is one that cannot count waiting vehicles beyond the first; that is, it does not have "variable initial interval." The initial interval is set by the traffic engineer at some value that stays constant from cycle to cycle.

Full-actuated control uses detection on all approaches. The green may be allowed to rest or dwell on the street that last called for it, or it may return automatically to a selected (actuated) phase.

Basic full-actuated controllers have a single timing adjustment (for each phase) labeled unit extension (or passage time or vehicle interval) that fixes both allowable gap (to hold the green) and passage time (from detection to stop line) at one common value. Inasmuch as the allowable gap is usually desired to be 3 or 4 sec, it follows that the detector ought to be located 3 or 4 sec of travel time back from the intersection.

However, the inability of the controller to count waiting cars creates a long minimum assured green if the "3- or 4-sec" principle is applied at approaches with speeds higher than 25 to 30 mph (40 to 48 kph). Therefore, the principle is amended to "3 to 4 sec of travel time, but not more than 120 ft (36m)," and this type of controller is not considered appropriate for approaches with speeds higher than those just mentioned. This amended principle can be summarized in a convenient table of detector locations and related timing adapted from Reference *1*:

TABLE 1
SUMMARY OF DETECTION LOCATIONS AND RELATED TIMING FOR BASIC ACTUATED CONTROLLERS OPERATED IN THE LOCKING DETECTION-MEMORY MODE

Approach Speed mph	kph	Detector ft	Set-Back m	Initial Int. sec		Unit Ext. sec		Minimum Assured Green sec
15	24	77	23	8.5	+	3.5	=	12
20	32	103	31	10.5	+	3.5	=	14
25	40	120	36	10.5	+	3.5	=	14
30	48	120	36	10.5	+	3.5	=	14
35 or more	56	Basic actuated controller not appropriate. Variable initial interval required.						

Semi-Actuated Controllers

With semi-actuated control, the objective in detectorization has been to minimize delay to the major street by using the shortest possible minimum assured green on the minor street. (Of course, the minimum must meet the needs of pedestrians if they are a factor at that location.) This objective points to the desirability of locating the detector as close to the stop line as possible. References *1* and *2* suggest set-backs as short as 45 ft and minimum greens as short as 8 sec. However, there are two factors that tend to compel set-backs no less than those suggested in Table 1 for full-actuated control: If a vehicle crosses the detector on the yellow and then clears the intersection, the controller will retain that false call and return the green to that approach unnecessarily. Zegeer's research in Kentucky (*3*) showed that at 30 mph (48 kph) the detector must be set back 4 sec of passage time to assure stopping. That is a distance of 176 ft (53 m). Therefore, it is out of the question to reduce detector set-backs to less than those given in Table 1. Most pedestrians seem to believe that they should be able to cross safely on a green, even if they have not pushed the button and even if their signal indicates DON'T WALK. Moreover, children on bicycles are unlikely to push the button. Therefore, there is reason for the traffic engineer to assure that *every* green meets the needs of pedestrians and bicyclists, provided they are a factor at that intersection. Thus it is not usually prudent for the traffic engineer to reduce the minimum assured green below the values shown in Table 1.

The emphasis on semi-actuated control ought to be to minimize delay to the major street by screening out false calls for the green to change unnecessarily to the minor street. Right turn on red is a prominent example of such a false call. Such screening requires nonlocking detection memory and large-area detectors. Therefore, the previous discussion of small-area-detector placement is moot. Semi-actuated intersections should use the nonlocking mode and long loops, as discussed later in this Appendix.

Controllers with Nonlocking Detection Memory

A controller phase may be switch-set by the traffic engineer to a detection memory circuit that is nonlocking rather than locking. The non-locking choice causes a waiting call to be dropped or forgotten by that controller phase as soon as the vehicle leaves the detection area. Nonlocking detection memory is associated with the use of large-area detection at the stopline, such as 6-ft-x-50-ft (2-m-x-16-m) loop or multiple magnetometer detectors. The advantage of this scheme, often called "loop-occupancy control," is that it avoids the problem with conventional control of giving no information on the traffic that is between the detector and the stop line. Therefore, it can reduce delay by screening out many of the false calls for the green. It reduces the frequency of unnecessary display of green to an approach that no longer has any vehicles waiting. One disadvantage is that large-area detection is inherently more expensive in first cost than small-area detection. At least as important in many localities is that long loops are more of a maintenance problem, particularly in localities where pavement condition is poor and ice and snow are major factors. A report by the Southern Section ITE (*4*) and a film by Georgia Tech (*5*) explain this type of control. The remainder of this section is summarized from these two sources.

No distinction is made between basic full-actuated and semi-actuated control in the design of detection. Both types of control follow the same principles, described next.

Application to Left-Turn Lanes

Left-turn lanes with separate signal control prompted the first use of nonlocking detection memory. A call placed during the yellow cannot bring the green back to an empty approach. Another potential advantage exists if the left-turn sequence of indications is "permissive" (i.e., the left turn is permitted to "filter" across oncoming traffic on the circular green shown to the through movement). Figure 1, taken from Reference *4*, shows such a left-turn lane at a T-intersection. If the left-turn lane is long enough to hold its queue, then the turn will usually be designed to lag the through movement because of the advantages described next. (Figure 1 is intentionally shown as a T-intersection because the use of a lagging green at a four-legged intersection may create a serious safety problem for left-turning drivers who mistakenly assume that both directions are being stopped at the same time. See References *6* and *7*.)

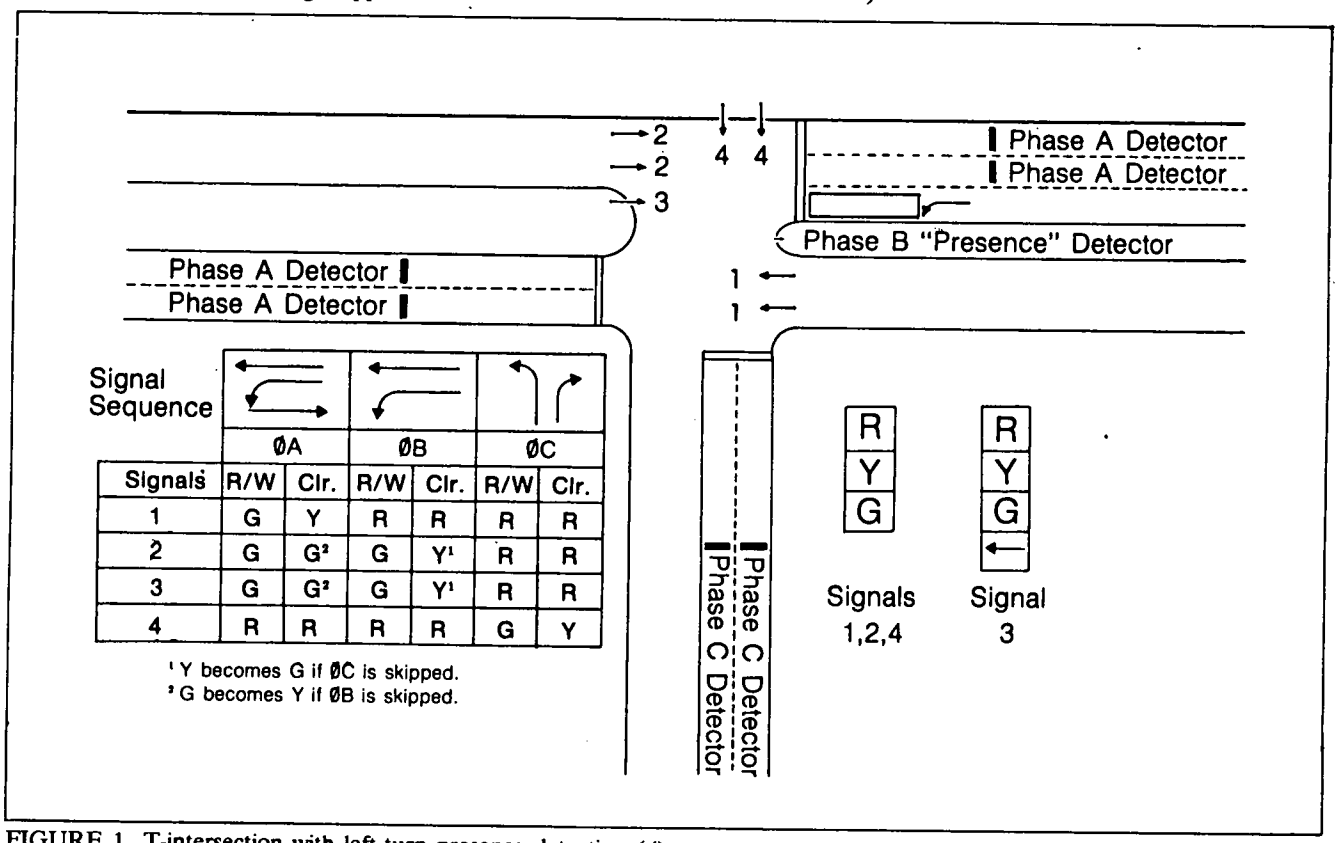

FIGURE 1 T-intersection with left-turn presence detection (*4*).

The left-turn bay uses a delayed-call detector, which is designed to output to the controller only if a vehicle is continuously detected beyond a time period (such as 5 sec) that has been preset by the traffic engineer. The use of a delayed-call detector in a left-turn bay allows the detector (and controller) to ignore vehicles that are "in transit" over the loop. They would be in transit if oncoming through traffic were light enough to permit them to "filter" through without the need for a protected left-turn phase. Thus, delay would be reduced by omitting the unneeded left-turn arrow. If, on the other hand, oncoming through traffic were so heavy that left-turning vehicles queued up over the loop, then the lagging green arrow would be called.

Detection of Small Vehicles

A presence detector should be able to detect a small motorcycle and hold its call until the display of a green to that phase. A bold time of 3 min commonly is specified. A detection loop longer than 20 ft (6 m) will not detect a small motorcycle. Reference *4* explains the use of modified long loops that include powerheads and angled powerheads. Multiple small loops are also discussed. In the mid 1970s the Canoga Controls Corporation developed a "quadrupole" configuration, shown in Figure 2, that adds a longitudinal saw-cut along the center of the lane. The loop wires are installed in such a way that the center wires have their currents flowing in the same direction. Their fields reinforce each other and improve the capability to detect small vehicles. To detect bicycles and small motorbikes, the configuration in Figure 2 is wound twice to give a double-layer design, termed a 2-4-2 installation. The advantages of the quadrupole proceed only from the configuration of the wire; any "amplifier" can be used.

Loop-Length Design

The required length of the detection area depends on vehicle speed and the controller settings. If the allowable gap is selected to be 3 sec, and the average vehicle length is not much different from 18 ft (6 m), then Figure 3 (from Reference *4*) gives the required loop length for a range of approach speeds and settings of the controller's vehicle interval (unit extension). The choice of vehicle interval depends on the traffic engineer's trade-off between cost of loop installation and maintenance on the one hand, and the cost of delay to motorists on the other. For example, an engineer in a Snow Belt state who is also on a very limited budget might select a VI of 1 1/2 or even 2 sec in order to minimize the length of the loop. (A minimum length of approximately 25 to 30 ft (8 m to 9 m) is required to assure that a vehicle waiting at the stop line will, in fact, occupy a portion of the loop. An even greater length is required if the loop extends downstream of the stopline.) The controller would not be able to gap out until the last car is 1 1/2 to 2 sec downstream of the detector, resulting in a sluggish transfer of the green. By contrast, an engineer in a mild climate, where pavement is sound and funds ample, might decide to minimize delay by selecting a "snappy" VI of 0 or 1/2 sec and a loop length of 70 ft (21 m).

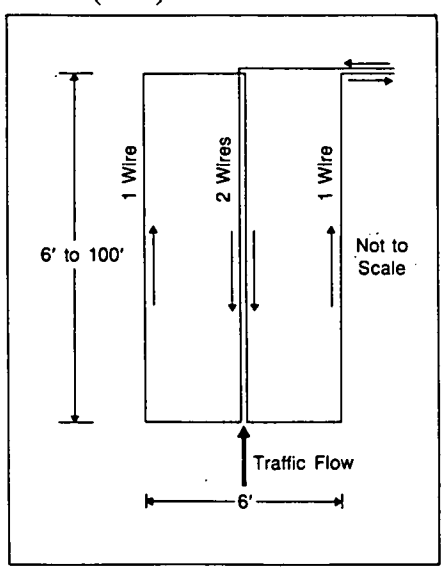

FIGURE 2 Quadrupole loop configuration, single layer (*4*).

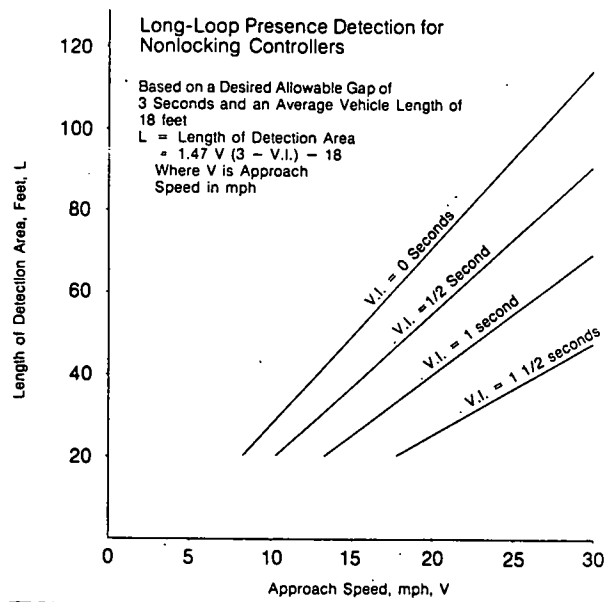

FIGURE 3 Design of loop length and vehicle interval (*4*).

HIGH-SPEED APPROACHES

Approaches experiencing speeds of 35 mph (56 kph) or higher are considered high-speed approaches. If the yellow comes on while the vehicle is in an "option zone" (zone of indecision), it may be difficult for the driver to decide whether to stop or clear the intersection. An abrupt stop may produce a rear-end collision. The decision to go through on the red may produce a right-angle accident. Table 2, from Reference *3*, shows the boundaries of the option zone. The traffic engineer can install a vehicle-actuated signal controller and appropriate detection, in an attempt to minimize the untimely display of yellow. A variety of schemes has been devised for controllers with locking and nonlocking detection memory, basic and volume-density controller circuitry, and various approaches to detection.

TABLE 2
OPTION-ZONE BOUNDARIES

Approach Speed		Distance from Intersection in ft (and m) for two Probabilities of Stopping	
mph	kph	10%	90%
35	56	102 (31)	254 (77)
40	64	122 (37)	284 (86)
45	72	152 (46)	327 (99)
50	80	172 (52)	353 (107)
55	89	234 (71)	386 (117)

Controllers with Locking Detection Memory

Detector/controller configurations using locking detection memory have been devised for basic full-actuated controllers, semi-actuated controllers, and volume-density controllers.

Basic Full-Actuated Controllers

Reference *8* suggests a high-speed design using a basic, locking controller and multiple small loops. However, the design assumes an emergency stop on a dry road; therefore, the first detector is not placed far enough upstream to give adequate option-zone protection. Attempts to improve the design in this respect result in an allowable gap that is so long that the controller would frequently "max out." This is unacceptable, because a vehicle may well be caught in the option zone if the green is extended to the maximum interval.

Semi-Actuated Controllers

A semi-actuated controller would use no detectors on the high-speed main roadway. The provision of option-zone protection would need to be based on detectors connected to an auxiliary logic unit that would hold the controller in phase A until the approaching vehicle had cleared the option zone. Such an auxiliary logic unit is offered commercially as a "green extension system" and is described in detail in Reference *1*. It consists of two or more extended--call detectors, one or more auxiliary timers that can disconnect or "force off" the extended-call detectors, and auxiliary electronics that can monitor the signal display, arm or enable the extended-call detectors, and control the yielding of the green to the side street (by activation of hold-in-phase circuits). Extended-call detectors, often termed "stretch detectors," have a "carryover output" (i.e., they hold or stretch the call of a vehicle for a period of seconds that has been set by the traffic engineer on an adjustable timer incorporated into the detector).

The concept of using extended-call detectors at high-speed approaches has merit. However, the choice of a semi-actuated controller, rather than a full-actuated model, requires that the extended-call detectors be accompanied by the considerable auxiliary logic explained previously. It would be much more straightforward, and equally effective, to choose a full-actuated controller at the outset. It would use extended-call detectors without any auxiliary circuitry. Such a scheme usually makes use of nonlocking detection memory; it is described subsequently under that heading.

Volume-Density Controllers

The most straightforward, conventional design for a high-speed approach uses a "density" controller with a single small-area detector at the upstream boundary of the option zone. This scheme is the "defender" in discussions of new configurations; it is the standard by which a challenging design is judged.

A "density" controller is an advanced actuated model that can count waiting vehicles beyond the first because it has a feature known as "variable initial interval." It will also have timing adjustments for the selection of allowable gap independent of passage time. For many years it was common to use "volume-density" controllers, which, in their two-phase models, had three gap-reduction factors. The NEMA functional standards for volume-density controllers, adopted in 1976, specify that the allowable gap will be reduced only on the basis of "time waiting" on the red. Such a machine is often termed a "modified density" or simply a "density" controller.

As shown in Figure 4, each approach has a small-area loop at the upstream end of the option zone, and a small-loop calling detector near the stopline. [A calling detector operates only when that phase is red (or yellow). It is disabled when the signal turns green so that it cannot extend the green.] The upstream detector is located 384 ft (117 m) from the intersection, which corresponds in Table 2 to a design speed of 55 mph (89 kph). It is easy to calculate from Table 2 that, for a typical approach speed of 50 mph (80 kph), the shortest setting of the allowable gap that would pass a vehicle through its option zone is about 2.5 sec. This constitutes a minimum desirable allowable gap; a shorter value would give snappier operation but could leave a vehicle in the option zone. Reference *9* points out a shortcoming of this design with regard to allowable gap:

> Upon termination of the green by gap-out...slow vehicles will not be protected by the "density" design if the Minimum Gap is set low at say, 2.5 seconds. There is a trade-off here between snappy operation and protection to the slower vehicles in the stream. One can be obtained only at the expense of the other. If a "density" design for 55 mph (89 kph) is to protect also the vehicle approaching at only 40 mph (64 kph), the Minimum Gap must be increased to 4.5 seconds.

An allowable gap of 4.5 sec is, of course, undesirable because the green may well be extended by moderate traffic to the maximum interval, thereby removing the option-zone protection.

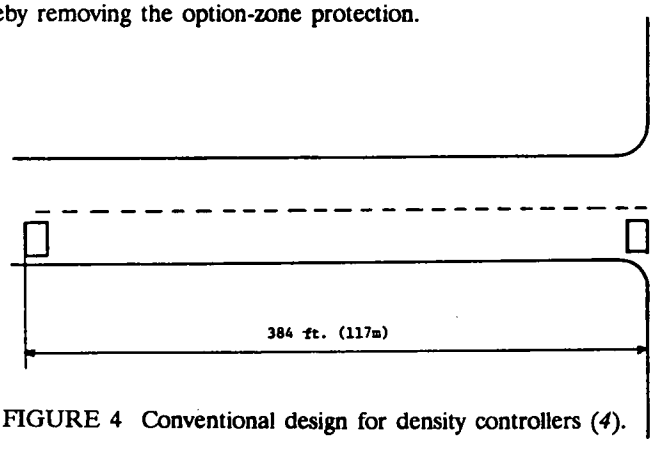

FIGURE 4 Conventional design for density controllers (*4*).

References *9* and *10* point out several other weaknesses of this scheme. They are all related to the lack of controller information on traffic at the stop line and for a distance of several hundred feet (approximately 100 m) upstream.

Density controllers usually offer an optional "last car passage" feature. If used, then upon gap-out the signal indication does not change until the last car has reached the stop line. Reference *10* points out that the *next* vehicle, called the "trailing car," may well be caught in the option zone. This is one of the reasons that the California DOT does not use this optional feature.

Reference *11* explains the features of density controllers and their application at high-speed approaches.

Controllers With Nonlocking Detection Memory

High-speed designs using nonlocking detection memory always include a long loop at the stop line (as well as one or more small ones upstream). The long loop improves the controller's knowledge of traffic at the stopline but tends to increase the allowable gap. Designs for both basic full-actuated and density controllers have been devised.

Basic Full-Actuated Controllers

Basic, full-actuated, nonlocking controllers have been used for a number of years with an extended-call detector just upstream of the option zone. Difficulties with "max-outs" under heavy-traffic conditions prompted the recent development of a novel EC-DC configuration that offers some advantages at modest extra cost.

Extended-Call Design The state of California uses a 70-ft (21-m) loop at the stop line supplemented by a single 6-x-6-ft (2-x-2-m) extended--call detector 250 ft (76 m) to 350 ft (106 m) from the stop line, depending on the approach speed (*5*, V.C. Dorsch, personal communication, 1973). Figure 5 shows this design for a speed of 55 mph (89 kph), based on Table 2 and the current quadrupole concept. The controller's unit extension is set at 0 or 1/2 sec. The setting of the "stretch" time on the extended-call detector should "carry" the vehicle approaching on the green through its option zone. Just as with the minimum gap setting on the density controller, the stretch setting here requires a compromise. If only 2.5 sec is used, the result is snappy operation but poor protection for the slower vehicles in the stream. If they are protected by increasing the stretch, then the green may be extended to the maximum interval. The difficulty with allowable gap is appreciably increased by the fact that vehicles create extensions of the green not only when they cross the stretch detector but also when they pass over the long loop at the stop line. This type of control appears to be limited to routes carrying no more than 8,000 to 10,000 ADT (*4*).

EC-DC Design Figure 6 (from Reference *9*) shows a recent attempt to make better use of basic, nonlocking controllers at high-speed approaches. The upstream detector is located in accordance with Table 2, for 55 mph (89 kph). The middle loop is placed 254 ft (77 m) from the intersection, which is the upstream boundary of the option zone for vehicles approaching at 35 mph (56 kph). Neither of these loops is an extended-call model; both use normal "amplifiers."

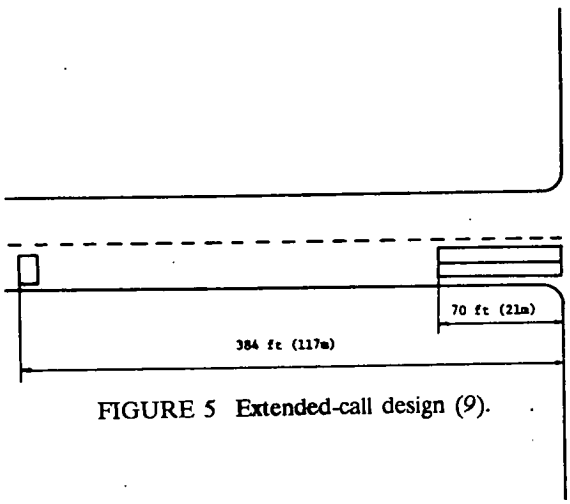

FIGURE 5 Extended-call design (*9*).

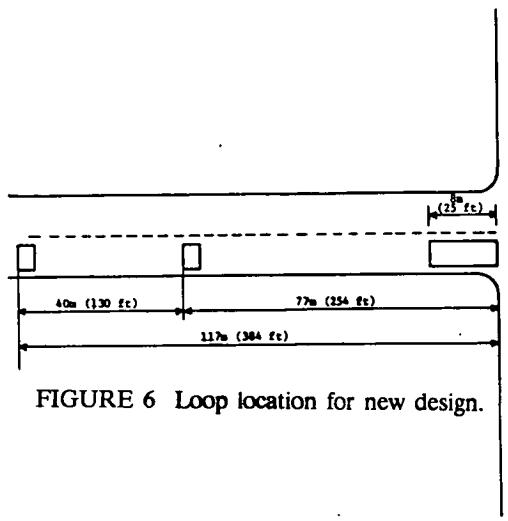

FIGURE 6 Loop location for new design.

The following explanation is given (*9*):

> The loop at the stopline is 8 m (25 ft) in length, which is intended to be long enough to bridge the gap between waiting vehicles, thereby assuring a call from a queue. The detector is a novel "EC-DC" unit that is able to change from an extended-call model to a delayed-call unit at the strategic moment during the green interval. Each mode of operation has its own adjustable timer.
>
> A description of the operation of the configuration begins with a start of green. As the waiting vehicles discharge over the stopline loop, its EC-DC detector functions as an extended call model. The controller, meanwhile, is timing a Minimum Green that Clark has found may need to be as long as 12 to 18 seconds in order to meet the expectations of truck drivers [J.D. Clark, personal communication, 1979]. Upon expiration of the Minimum Green probably only five or six vehicles have discharged and there is still no motion over either of the upstream loops. A "stretch" setting of approximately two seconds on the EC-DC detector is intended to produce an unbroken actuation, and an extension of the green, until motion is assured over the middle loop. By this time discharging traffic is up to speed. A two-second gap between vehicles appears. The extended-call detector at the stopline in effect "gaps out" and becomes a delayed-call unit with approximately five seconds of time-delay. The full-speed vehicles in transit over the stopline loop do not produce a call. This loop has in effect become disconnected, and the continued extension of the green is controlled by the upstream loops and the Unit Extension setting of the controller.
>
> If the upstream loops utilize an "amplifier" that produces a short pulse when the vehicle enters the loop, then the Unit Extension setting of the controller is selected to be 2.2 seconds. It is simple to show that this setting will "carry" vehicles approaching within the speed range of 64 km/h (40 mph) to 89 km/h (55 mph) through their respective option zones.

A vehicle approaching at 35 mph (56 kph) is also protected because the yellow will appear *before* the vehicle reaches its option zone.

With a unit extension of 2.2 sec, the allowable gap produced by the two upstream loops is calculated as the travel time from the first loop to the middle loop. The allowable gap is therefore 3.8 sec for traffic at the design speed of 55 mph (89 kph) and 4.4 sec for traffic at 40 mph (64 kph). Although these values are not much lower than those discussed previously for the extended-call design, actually the difference is greater. The ability of the EC-DC design to disconnect its stop line loops gives it a decided superiority to the extended-call design in real-world operation.

Volume-Density Controllers

Grimm (*12*) devised a high-speed design using a nonlocking density controller, a long loop at the stop line, and one extended-call detector upstream. The design was of limited application, however, because it assumed an emergency stop on a dry pavement. Lepic (*13*) modified Grimm's work to use multiple detectors upstream. His objective was to accommodate a wider range of design speeds and to consider real-world deceleration rates. Figure 7 shows his design for a 50 mph (80 kpb) approach speed. The stretch detectors are set to 1.5 to 1.8 sec, resulting in an allowable gap of 4.1 sec, according to Lepic.

According to Table 2 herein, the upstream-detector set-back of 300 ft (91 m) gives this scheme a design speed of 43 mph (69 kph), not 50 mpb (80 kph). It appears that his configuration needs further work to increase its design speed.

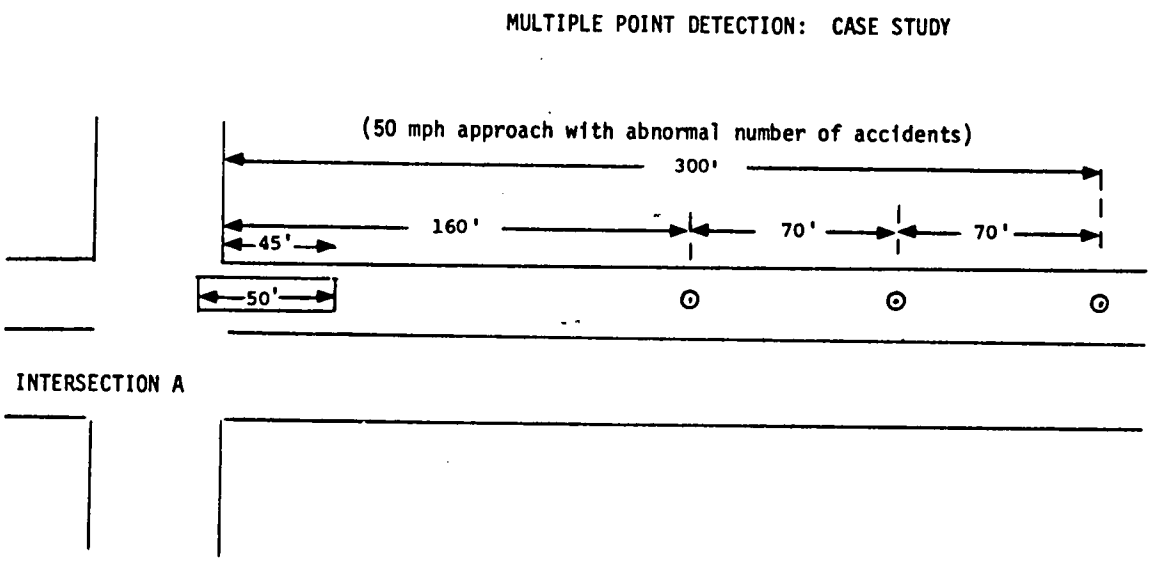

FIGURE 7 Lepic's detection for nonlocking density controllers (*13*).

APPENDIX B
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3. Zegeer, C.V., Effectiveness of Green-Extension Systems at High-Speed Intersections," Research Report 472, Kentucky D.O.T., Bureau of Highways, Division of Research, Lexington, Ky. (May 1977).
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5. "Operation of Actuated Traffic Signals at Local Intersections; Part IV: Loop-Occupancy Control," 30-min film, 16 mm, available on loan from the Center for Media-Based Instruction, ESM Building, Georgia Institute of Technology, Atlanta, Georgia 30332.
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12. Grimm, R.P., "Traffic Signal Operation Design: Efficiency and Safety," Newsletter of Western ITE (October/November 1974) pp. 2-4. [Reprinted in *Technical Notes,* Vol. 1, No. 1, ITE (Summer 1975) pp. 2-4.]
13. Sackman, H. et al., "Vehicle Detector Placement for High-Speed, Isolated, Traffic-Actuated Intersection Control," Vol. 2, Manual of Theory and Practice, Report No. FHWA-RD-77-32, Federal Highway Administration, Washington, D.C. (May 1977) pp. 67-71.

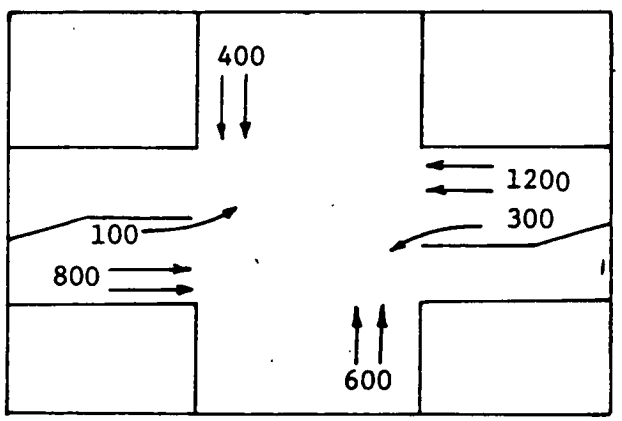
APPENDIX C
SELECTED CASE STUDIES

The three case studies that follow present approaches that have been used to improve traffic signal timing. The first case study is taken from a bulletin prepared by Automatic Signal about 1965. The second and third case studies were submitted in response to the questionnaire used in the synthesis (Appendix A).

Case Study 1. Simplified Method to Determine Capacity of Alternative Signal Phasings
(Adapted from a bulletin prepared by Automatic Signal circa 1965)

The following case study begins with simplified calculations to determine the optimal phasing. Although that area is beyond the scope of this synthesis, the phasing work is included because it supports the timing calculations immediately following. The simplified phasing procedure builds on the fundamentals introduced in Chapter 2. The procedure is of value in itself but also provides a foundation to understand the more complex methods presented in Chapter 9 of the *1985 Highway Capacity Manual* (HCM). The procedure introduces the Planning Analysis beginning on page 9-21 and the Appendix II on Signal Design beginning on page 9-64 of the HCM.

Peak Hour Traffic Count



Notes:

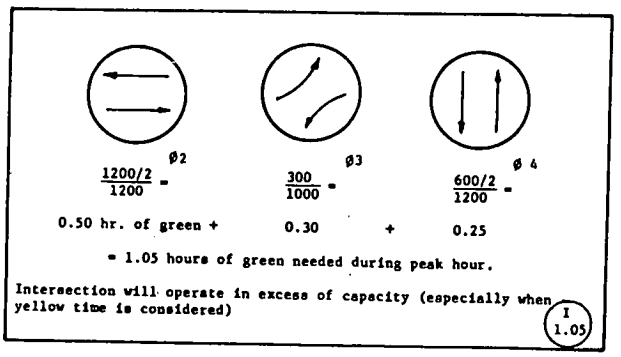
(1) Capacity of a through lane = 1200 veh/hr of green, not including yellow.

(2) Method ignores yellow-time effect on capacity, for simplicity. If all trial phasings have the same number of yellow intervals per cycle, then this simplification affects all trials equally.

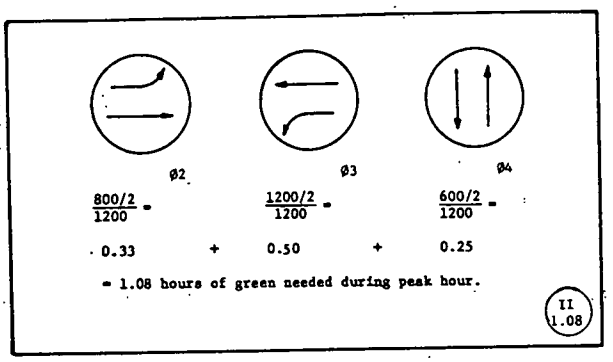
(3) Capacity of a left-turn lane = 1000 veh/hr of green, assuming a separate turning lane and separate signal control.

The capacity values of 1200 and 1000 vphg are not saturation capacity flows; they reflect start-up losses.

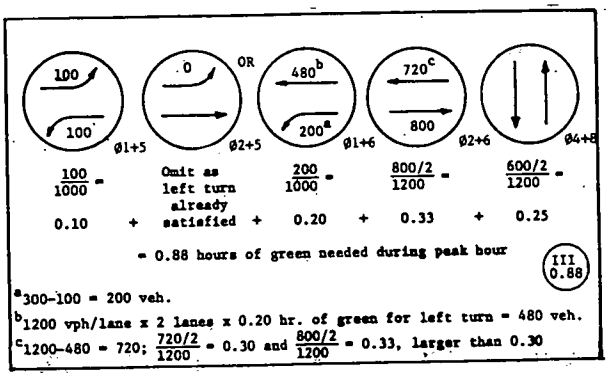
Each arrow in the diagram is a lane. The 1200 vph flow westbound is carried on two lanes, so the per-lane flow is 1200/2



TRIAL PHASING I–THREE-PHASE FULL-ACTUATED CONTROLLER WITH SIMULTANEOUS LEFT TURNS ON Ø 3



TRIAL PHASING II–THREE-PHASE FULL-ACTUATED CONTROLLER WITH OPPOSITE PHASING ON ARTERY

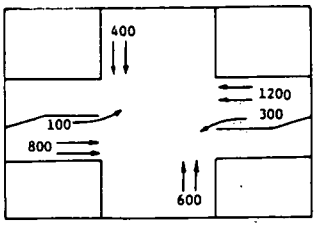


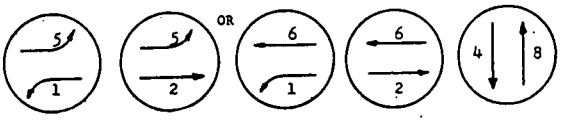
TRIAL PHASING III–FIVE-PHASE FULL-ACTUATED CONTROLLER WITH DUAL LEFT TURNS FROM ARTERY

Trial III, with its dual left turns from the artery, is seen to be the most attractive of the alternatives examined.

TIMING THE FIVE-PHASE FULL-ACTUATED ATERIAL INTERSECTION

Peak-Hour Traffic Count





NEMA Movement No.	1	2	5	6	3	4	7	8
Lane Volumes	300	800/2	100	1200/2	0	400/2	0	600/2
Conflicting Sums	700		700		200		300	
Larger Major-Street Sum		700						
Larger Minor-Street Sum						300		
Sum of Critical Lane Volumes				1000				

Although five controller phases are needed for this design--l, 2, 4, 5, and 6--it is considered a three-phase design for purposes of determining cycle length. That is, the five phases do not time sequentially; concurrent timings add efficiency through overlap movements. So, for purposes of determining cycle length, we would consider that there are start-up losses and yellow losses associated with the dual lefts, the artery throughs, and the cross street--hence, three phases. Now, assume that the given volumes were obtained by taking the peak 15-min volumes and multiplying each by 4 to give an equivalent hourly volume--a "flow rate." Our goal for level of service is that during the peak 15 min of the peak hour there be no queue at the end of the period; in other words, all the vehicles that arrive during that period will clear during it. (However, a vehicle may not clear on the cycle in which it arrives.) That is, over the peak 15-min period the greens are long enough to pass the **average** rate of arrivals, but the random peaks and surges above the average rate will create short-lived queues. This is the same as saying that we can accept a "50 percent probability of performance," where "performance" means success in passing all the vehicles that arrive during that cycle. Half the time we will clear all the vehicles that arrive in that cycle. The cycle length can be obtained using an iterative procedure with Davidson's curves, or can be obtained from the following table.

Sum of Critical Lane Volumes at Intersection	Minimum Cycle Length in Seconds		
	Two-Phase 2	Three-Phase 3	Multi-Phase 4
800	30	40	60
900	35	50	70
1000	40	60	80
1100	45	70	90
1200	50	80	105
1300	60	100	120
1400	80	125	-
1500	110	-	-

Entering the table with 1000 vph and three phases, we obtain a cycle length of 60 sec.

If our given volumes were actually **hourly** ground counts--not flow rates--then we would require a higher probability of performance than 50 percent, such as 75 percent, so that during the peak 15 min we would preserve our desired probability of 50 percent. One way to do this is to use the three phase volumes of 300, 800/2, and 600/2 to enter Davidson's curves. By trial and error, obtain a cycle length of 90 sec. An easier procedure begins by dividing the sum of the critical lane volumes (1000 vph) by the peak-hour factor (PHF), which might be 0.8. The 1000 is an hourly rate, an average for the hour. Dividing it by the PHF gives the flow rate during the peak 15 min, or 1250 vph. As explained previously, a probability of performance of 50 percent is required for our signal timing during the peak 15 min, so the previous table applies. Entering the table with 1250 and three phases yields a cycle length of 90 sec, the same as obtained by trial and error with Davidson's curves. This example makes it clear that cycle-length determination is sensitive to the consideration given to peaking within the peak hour. and to the engineer's goals for level of service.

Now we will return to the original solution that produced a cycle length of 60 sec. To obtain the green time, the 60 is now reduced by three clearance periods averaging approximately 4 sec each:

$$60 - 12 = 48 \text{ seconds of green}$$

The 48 is now divided among the three phases according to relative volume (or relative volume x headway if headways differ because of upgrade, trucks, etc.):

Left Turn,	Phase 1	$\frac{300}{1000}$	x 48 =	14.4
Through	Phase 2	$\frac{800/2}{1000}$	x 48 =	19.2
Cross-Street	Phase 4	$\frac{600/2}{1000}$	x 48 =	14.4
				48.0

The five controller phases need to have maximum interval settings of 1.25 to 1.5 times the "fixed time" intervals calculated, so that brief surges in arrivals will be cleared. The following table uses 1.5:

Controller Phase	Lane Volume	Fixed-Time Green		Surge Factor		Setting of Maximum Interval
1	300	14.4	x	1.5	=	21
2	$\frac{800}{2}$	19.2	x	1.5	=	29
4[a]	$\frac{600}{2}$	14.4	x	1.5	=	21
5	100	5	x	1.5	=	7
6[b]	$\frac{1200}{2}$	27	x	1.5	=	41

[a]Movements 4 and 8 would be timed using a single controller phase 4.

Note that at another time of day, movements 2, 4 and 5 may well be larger in volume than at the time of day represented by this problem. An external call to Max II could be used to change to a longer maximum interval at that time.

[b]Phase 6 is not a critical movement. One way to determine the timing is to note in the Trial Phasing III diagram that its timing is determined by phase 1+6 followed by phase 2+6. Phase 1+6 is controlled by the 200 vehicles on phase 1, and phase 2+6 is controlled by the 800 on phase 2. Then,

$$\frac{200}{1000} \times 48 = 9.6$$

$$\frac{720/2}{1000} \times 48 = 17.3$$

$$26.9 = 27 \text{ sec}$$

Case Study 2. Delaware Bureau of Traffic Detector Switching

Traffic-engineering agencies learn to adapt standard signal-control hardware to the solution of complex problems. The Delaware Bureau of Traffic uses various loop locations, loop delay and extension features, and detector omit circuits to solve specific problems. Chief traffic engineer Raymond S. Pusey finds that these are the challenges that distinguish the true traffic engineers. "This is where the fun is," he says. In the pages that follow, Pusey explains how Delaware switches detectors among controller phases. His reference to the "Dilemma zone" pertains to the zone within which high-speed drivers have difficulty deciding whether to stop or clear. New York State and some others call it an "option zone," as explained in the body of this synthesis. He does not refer to the dilemma facing drivers when the clearance period is so short that they can neither stop nor clear.

As both traffic signal controllers and vehicle detectors evolved through the 1960s and 1970s, maximizing the efficiency of an actuated intersection sometimes required switching detectors between phases as well as turning detectors on and off. The advent of the multi-ring controller solved many of these switching problems that had required extra timers and relays.

Typical of our control plans using external detector switching logic was the one used at a T-intersection with a left-turn lane.

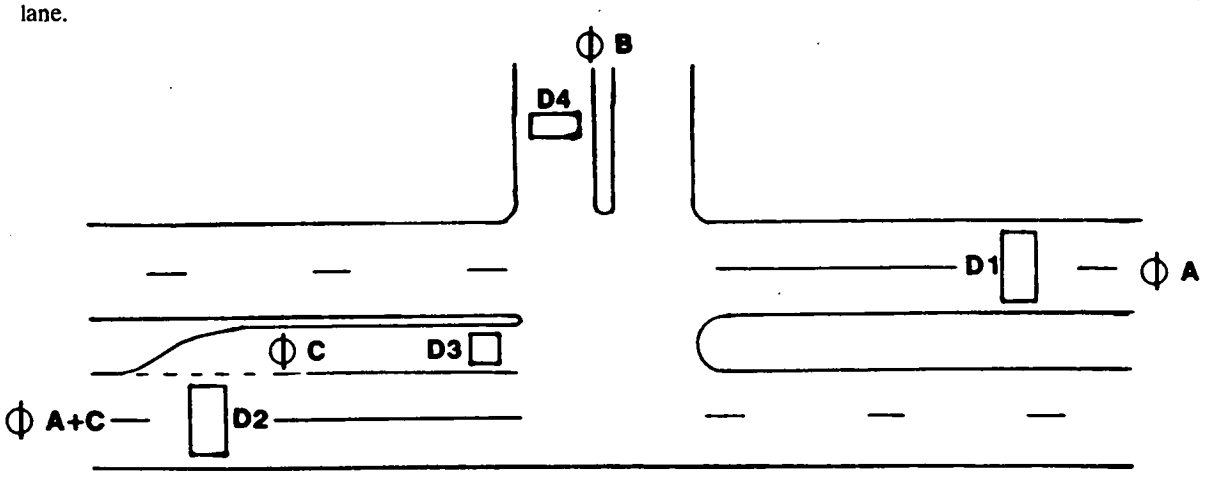


FIG. 1

A three-phase controller (pre-NEMA) was set up as shown in Figure 1. Detectors 1 and 2 were connected to phase A.

Even if a protected-only left-turn display was used and the controller did not have added initial or time waiting or gap reduction features, detector switching could reduce delay time.

If there is a call for the left turn (phase C, detector 3), and none for the stem of the T (phase B, detector 4), the directional split on the top of the T becomes important. Should detector 1 have a light traffic volume and detector 2 a heavy volume, the signal would remain on Phase A, thus delaying the left turn for no valid reason, unless detector 2 is switched off.

The logic is:

All detectors are active when:

1. No call is active for phase B or phase C or
2. Phase A is yellow or red (not green).

Detector 2 is turned off only when:

1. Phase B has no call and phase C has a call.

For controllers with added initial or time waiting or gap reduction features, detector 2 must be switched off during phase C green and yellow and all-red. Having detector 2 operational during phase C will run up the phase A added initial. Switching detector 2 on and off depending on the call status of phases B and C can disrupt other features.

A two-ring quad left-turn NEMA logic machine accomplishes this purpose automatically when phased (Figure 2):

Pre-NEMA	NEMA
Phase A	Phase 2
Phase (A+C)	Phase 6
Phase B	Phase 3
Phase C	Phase 1

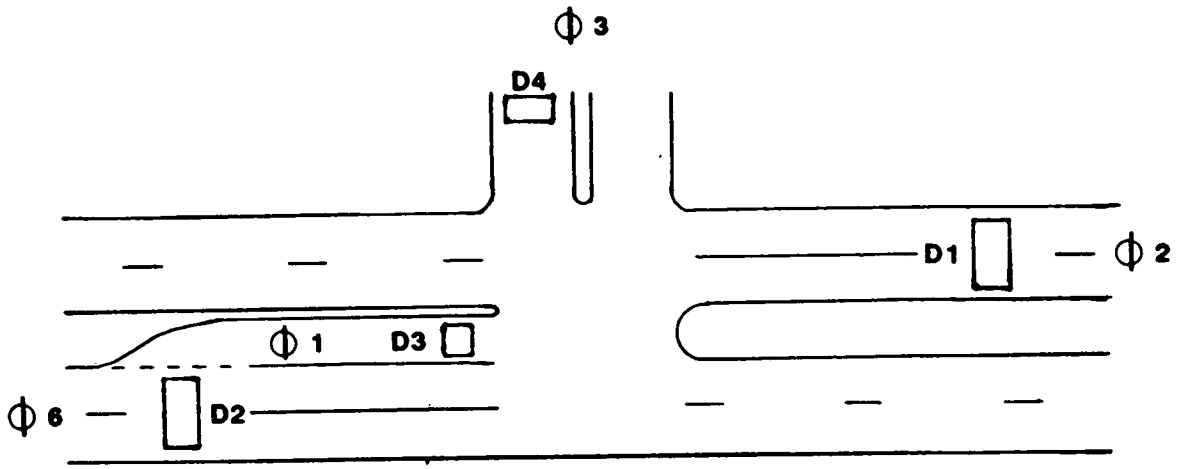


FIG. 2

Assigning each approach a separate phase and placing them by ring has eliminated many external logic packages, much to the joy of our signal techs. Although it is entirely possible to signalize this intersection with a single-ring controller under actuated control, we find it impractical.

Changing the left-turn display to protected/permissive requires detector occupancy timing, as well, for either NEMA or pre-NEMA machines.

If a vehicle turning left on a permissive green activates detector 3 (Figure 2) while the controller is at rest, the controller will begin to cycle even if the vehicle clears without so much as a rolling stop. One solution is to use a presence detector and to delay the call during phase 2 green. Some jurisdictions simply turn off the left-turn detector until the controller responds to a phase 3 call. This may cause queueing problems in the left-turn lane.

The logic is:

The left-turn lane detector, detector 3, is time-delayed during phase 2 green only.

This particular logic permits a lagging left-turn display whenever there is left-turn traffic and no side-street (stem of the T) traffic. For those jurisdictions that do not permit lagging green displays, full response to a left-turn call requires that the controller display a cross-street green before the left-turn green. That can be accomplished by placing a call in the cross-street phase when a call is placed in the turn lane phase.

To ensure the desired phase order of controller response, detector 3 is connected to phase 3 and is switched to phase 1 by phase 3 green.

The logic is:

The left-turn lane detector, detector 3, is time-delayed except during phase 3 green, yellow, and red (phase 3 on) and during phase 1 green.

Detector 3 is connected to phase 3 except during phase 3 green, yellow, and red (phase 3 on) and during phase 1 green.

By delaying the response of Detector 3, needless stops for Phase 1 and unnecessary delays to phase 3 traffic are prevented. The time delay must be removed during Phase 1 green when prompt detector response is necessary and during phase 3 until the "next phase" decision is made. If a vehicle covers detector 3 just as phase 3 is about to end, phase 1 must not be skipped.

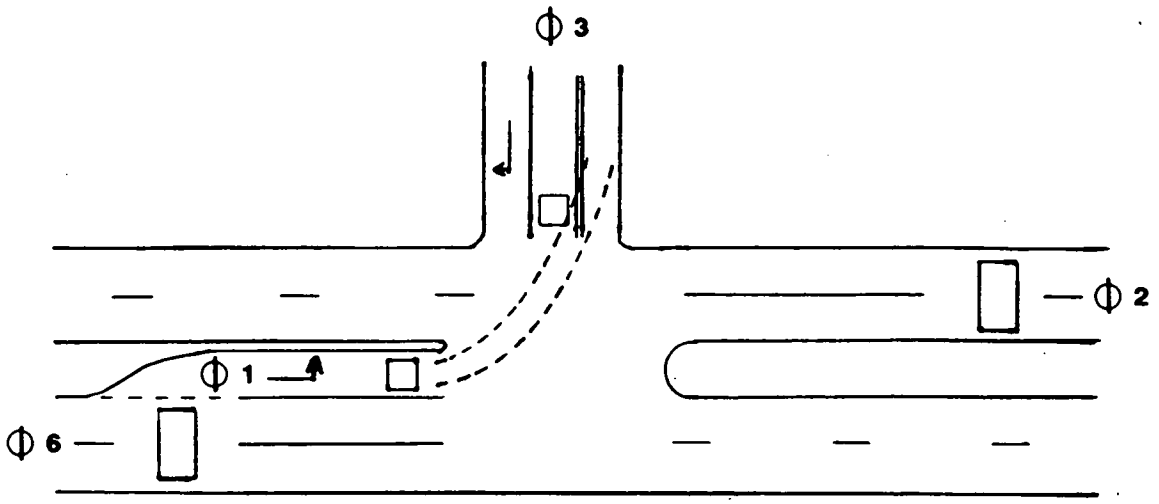


FIG. 3

A vehicle turning left will often catch the edge of a cross-street left-turn loop if the lane widths or radii are substandard. A presence detector with a 3-sec delay will prevent the departing vehicle call from being recognized. The delay must be turned off when the offending left-turn lane has a red.

Right turn on red (RTOR) (Figure 4) introduced an interesting problem at many of our intersections where separate right-turn lanes did not exist. A presence detector at the stop line with time delay ended the problem of signal cycling for no apparent reason during periods of light traffic. Unfortunately, variable initial timing and gap reduction do not respond as well to a stop line detector.

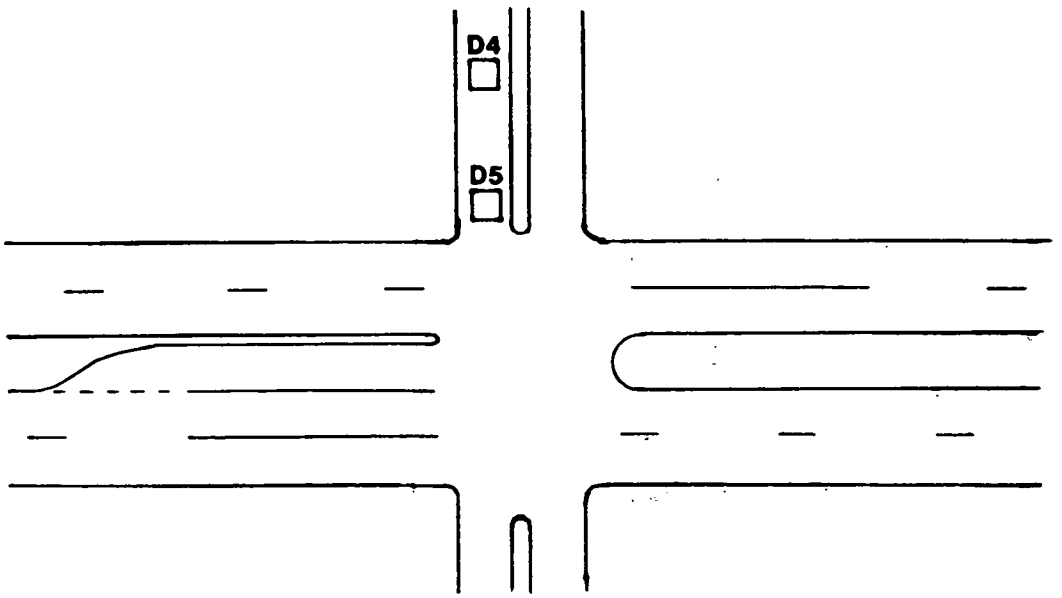


FIG. 4

Retaining the approach detector and switching it and the stop line detectors retains most of the best of both. When a vehicle is detected at the stop line (Detector 5, Figure 4) for a period exceeding the delay timer, a call is sent to the controller, the approach detector (Detector 4) is turned on, and the stop line detector is turned off. The obvious flaw in this system is that a platoon of traffic may arrive and fill the space between the detectors before the switching takes place.

Under light traffic that is unlikely. Under heavy traffic, a call usually remains at the end of the green so the stop line detector never switches on. A time clock can be used to eliminate the RTOR detection during specified periods as well.

The solution that we favor is to forgo the variable initial feature for the gap reduction feature. The phase green then is the switching key.

Although the multi-ring machines with assignable phasing allowed most of the external detector switching logic to be eliminated, there are still cases where the techniques originated for the single-ring machines have merit. For most cases, better geometrics is the correct solution. That does not absolve the traffic engineer from doing his best until reconstruction can be accomplished.

One of the more interesting situations now in place is illustrated in Figure 5.

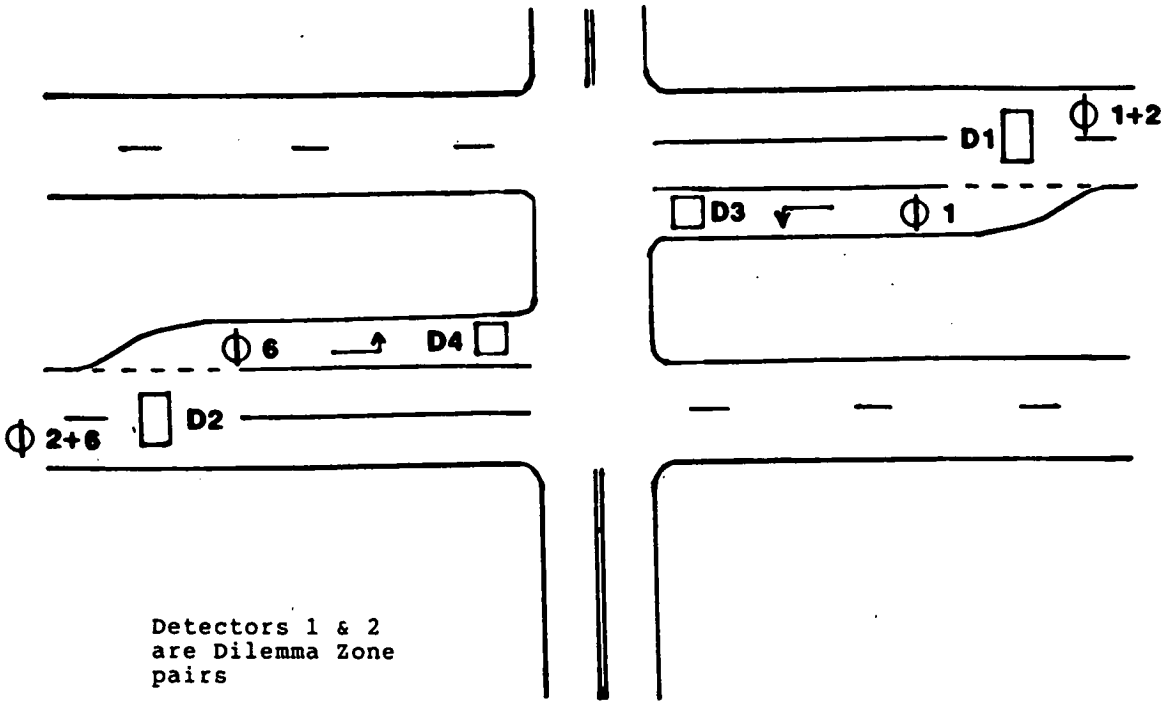


FIG. 5

The figure illustrates a situation where leading green left-turn indications are not possible because of the wide median. These intersections are signalized using a leading turn for one direction and a lagging turn for the other.

NEMA Phasing Leading Green	Lead/Lag Phasing
1 5	1 5
2 6	2
---	6
3 7	---
4 8	3 7
	4 8

The through signal displays shown in Figure 5 are actually overlaps (1+2) and (2+6), but the through lane timing for both directions is done on controller phase 2. Detectors 1 and 2 are connected to phase 2.

Because one through direction moves with the adjacent left turn (leading turn) before phase 2 is green and the other after it has been green (lagging turn), a variable initial on phase 2 becomes inaccurate. Our choice is to forgo this interval. It is possible to make limited use of the variable initial interval by switching the phase 2 detectors off during phases 1 and 3 as well as whenever phase 1 has a call. This is not a valid measure of need.

In our case, these intersections often occur in high-speed areas where it is our practice to use dilemma zone approach detection. Dilemma zone detection for both directions connected to phase 2 has no meaning if the signal has an unanswered call for the lagging green. Furthermore, to get full benefit for the lagging direction, the dilemma zone detector must switch from phase 2 to phase 6 when an unanswered lagging call exists during phase 2 green.

The question of determining when to move into the lag phase can be difficult to answer. Ideally, the operation would be such that the through movement from the lag direction would be fully served at the same moment that the lag left turn is fully served. Although that can be done, it is considerably more complex then necessary for most cases.

Our practice is to use one of the loops of the dilemma zone detector pair as an approach detector on the lag approach to ensure sufficient through green and to switch the dilemma zone system between the lag phase and the through phase, depending on left-turn calls.

The logic is:

With the controller at rest
(phase 2 green) or with phase 1 timing:

Detector	Connected to Phase	
1	2	(Dilemma zone)
2	2	(Dilemma zone)
3	1	
4	6	

With the controller timing Phase 2 and:
No call for Phase 6

Detector	Connected to Phase	
1	2	(Dilemma Zone)
2	2	(Dilemma Zone)
3	1	
4	6	

Call for Phase 6

Detector	Connected to Phase	
1	2	(Dilemma Zone)
2	OFF	(Dilemma zone)
2	2	(Approach)
3	1	
4	6	

With the controller timing Phase 6:

Detector	Connected to Phase	
1	2	(Dilemma Zone)
2	6	(Dilemma zone)
3	1	
4	6	

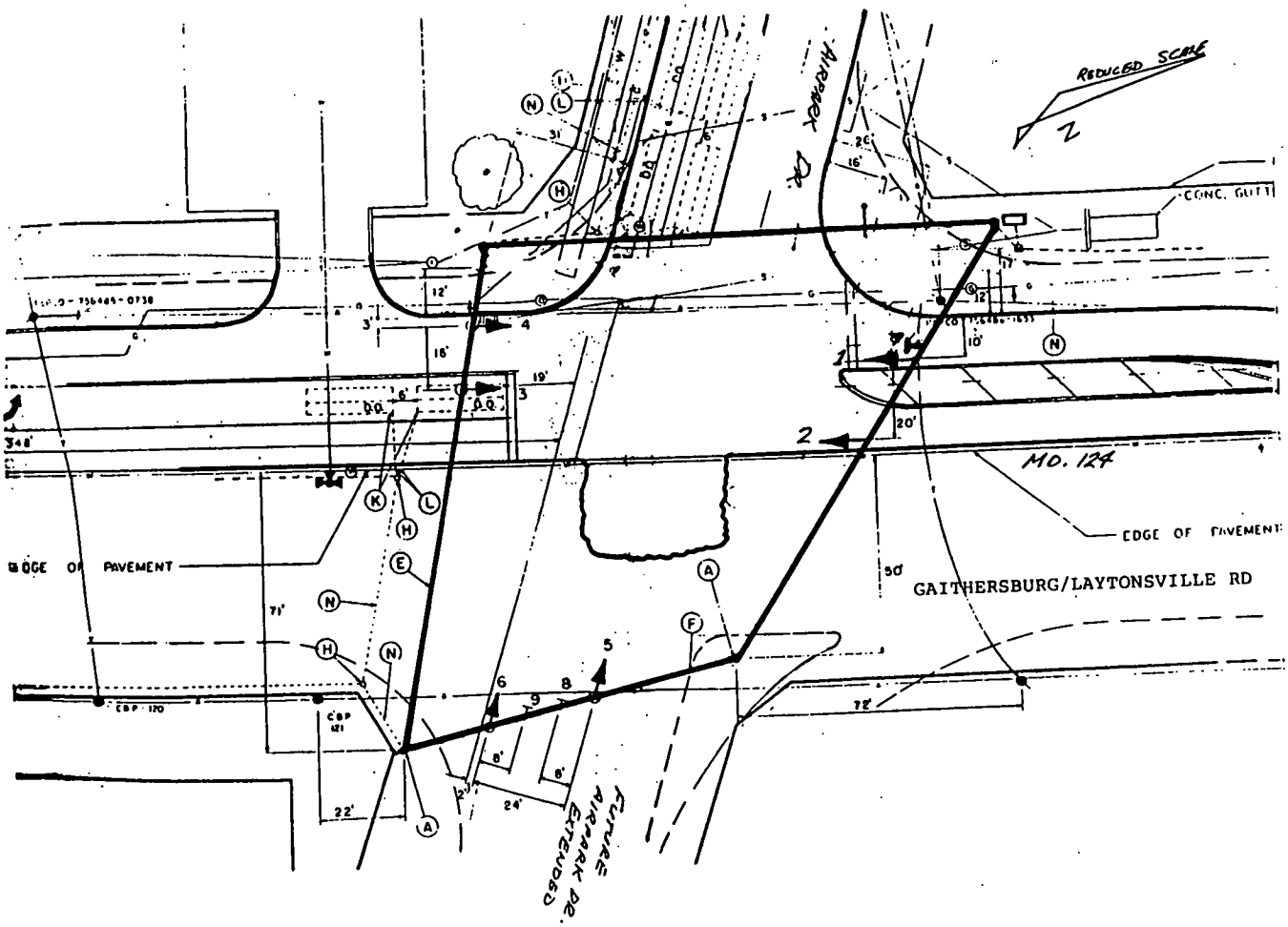
Case Study 3. Montgomery County, Maryland--Three Examples

Traffic engineers Scott Wainwright and Bruce Mangum of Montgomery County, Maryland, contributed a number of examples in their response to the questionnaire. Three of the examples follow. The first pertains to the intersection of Airpark Drive and Gaithersburg/Laytonsville Road (MD 124). It is fully actuated and isolated. The calculations show their iterative process, using trial calculations with different cycle lengths. The process uses a chart titled "Thru-Inter Signal," which is basically the Greenshields formula. Factors are used for more than one lane as in "critical-lane" types of capacity analyses (2 lanes = 0.55, 3 lanes = 0.40, and 4 lanes = 0.30).

The other two examples show "artful" fine-tuning, in response to Question 6 of the questionnaire. The intersection of Parklawn Drive and Twinbrook Parkway includes a special pedestrian phasing that is both efficient and safe when the side-street approaches are "split-phased," meaning that each moves on a phase separate from the other. The pedestrian movement, phase 8, times concurrently with *both* phase 3 (one direction of the side street) and phase 4 (the other direction). This avoids having to time either phase 3 or phase 4 alone to handle complete pedestrian timing needs.

The third example from Montgomery County shows the major arterial New Hampshire Avenue (MD 650) and its intersections with Elton Road and the westbound off-ramp from I-495. The heavy right-turn movement from the off-ramp creates the need for close attention to what detectors are assigned to which phases and overlaps in multi-phase operation.

AIRPARK DRIVE AND GAITHERSBURG/LAYTONSVILLE ROAD



Montgomery County, Md.
Department of Transportation
Division of Traffic Engineering

Thru-Inter Signal
(after Greenshields)

V/α/La.	Sec	Cum. Sec	V/α/La.	Sec.	Cum. Sec
1	3.8	3.8	24	2.1	54.1
2	3.1	6.9	25	2.1	56.2
3	2.7	9.6	26	2.1	58.3
4	2.4	12.0	27	2.1	60.4
5	2.2	14.2	28	2.1	62.5
6	2.1	16.3	29	2.1	64.6
7	2.1	18.4	30	2.1	66.7
8	2.1	20.5	31	2.1	68.8
9	2.1	22.6	32	2.1	70.9
10	2.1	24.7	33	2.1	73.0
11	2.1	26.8	34	2.1	75.1
12	2.1	28.9	35	2.1	77.2
13	2.1	31.0	36	2.1	79.3
14	2.1	33.1	37	2.1	81.4
15	2.1	35.2	38	2.1	83.5
16	2.1	37.3	39	2.1	85.6
17	2.1	39.4	40	2.1	87.7
18	2.1	41.5	41	2.1	89.8
19	2.1	43.6	42	2.1	91.9
20	2.1	45.7	43	2.1	94.0
21	2.1	47.8	44	2.1	96.1
22	2.1	49.9	45	2.1	98.2
23	2.1	52.0	46	2.1	100.3

V = no. of vehicles
α = cycle length
La. = Lane

Cum. = Cumulative
Sec = Seconds
Adj. = Adjustment

For Use with
Signal Timing
Comp. Sheet

SIGNAL TIMING COMPUTATION SHEET

Intersection Airpark Dr. - Gaith./Laytonsville Rd. (MD. 124) Signal # 365-A.O.

La. = lane ; V = vehicle(s) ; H = hour ; α = cycle ; p = per ; NA = not adjusted ; Adj = adjusted

Clearance = Total of 12"					Trial #1 αL: 60" αpH: 60			Trial #2 αL: 90" αpH: 40			Trial #3 αL: 75" αpH: 45		
Hour and Approach	# of La.	Adj. factor	VpH	V/H/La.	Vpα	Time Req'd. NA	Time Req'd. Adj	Vpα	Time Req'd NA	Time Req'd Adj	Vpα	Time Req'd NA	Time Req'd Adj
AM PEAK ø1 NBLT	1	1.0	307	307	5.1	15"		7.7	20"	18"			
ø2 SB THRU	1	1.0	930	930	15.5	36"		23.3	53"	50"	78" +	12" CL. =	90"
ø4 EB LT	1	1.0	146	146	2.4	8"		3.7	11"	10"	THE 18, 50 AND THE INITIAL SETT		10 WERE INGS FOR
						+ 12" CLEARANCE 71°	> 60" (N.G.)		+ 12° CLEARANCE 96" >	90" (CLOSE)			MAX 2 (AM PEAK)
PM PEAK ø1 NBLT	1	1.0	163	163	2.7	9"					3.4	11"	15"
ø2 NB THRU - NBLT	1	1.0	714-163 = 551	551	9.2	23"					11.5	28"	33"
ø4 EBLT	1	1.0	167	167	2.8	9"					3.5	11"	15"
						+ 12" CLEARANCE 53"	< 60° (OK)					+ 12" CLEARANCE 62" <	75" (OK)
											THE 15, 33 AND THE INITIAL SETTINGS		15 WERE FOR MAX 1 (PM PEAK & OFF PEAK)
Note: Actual settings after field observations were different — See attached controller settings timesheet.													

63 + 12" = 75"

ECONOLITE
KMC

TRAFFIC SIGNAL TIMING SHEET
TRAFFIC OPERATIONS SECTION
Division of Traffic Engineering
Montgomery County, Maryland

Office_____
Shop_____
Controller___

INTERSECTION: Airpark Drive - Gaithersburg/Laytonsville Rd (MD. 124)

NO. I-365 A.0

FROM DYN. DISP. [F] ↓ Ø# ↓ [INT] ↓ INT #. ↓ VAL # ↓ [ENT]

STREET/MOVEMENT	PHASE Intervals	MINGRN 1	WALK 2	PEDCLR 3	VEHEXT 4	MAX I 5	MAX 2 (AM) 6	YELLOW 7	RED 8	RED REVT 9	ADDED INT 10	MAX INIT 11	TBR 12	TTR 13	MIN GAP 14	MODE 0
NB LEFT (MD 124)	Ø1	3	–	–	3	15	18	3.0	0	2	0	99	0	0	3	1
SB MD. 124	Ø2	15	–	–	6	40	60	5.5	1.0	2	2	30	0	0	6	3
	Ø3															
AIRPARK ROAD	Ø4	5	–	–	3	18	10	3.5	1.0	2	0	99	0	0	3	1
	Ø5															
NB MD. 124	Ø6	15	–	–	6	40	60	3.5	1.0	2	2	30	0	0	6	3
	Ø7															
	Ø8															

FROM DYN DISP [F] ↓ [Ø] ↓ [INT] ↓ INT # ↓. VAL # ↓ [ENT]

PHASE ZERO INTERVALS

Interval Number	Interval Function	Description	Range	Displayed in ØT ØN Indicators	→ Entry ← by	Key
0	Flash time	Sets power ON flash period. Zero time inhibits function	1–255 seconds	No		10
1	Initial start interval	Selects power ON or initial condition from options below: 0 = Yellow with O'LAPs green or yellow based on O'LAP programming 1 = Green with O'LAPs green 2 = Red with O'LAPs red 3 = Green with O'LAPs green 4 = Yellow with O'LAPs yellow	0–4	No		1
2	External start interval	Selects external start condition from options above	0–4	No		1
3	Initial start phase(s)	Selects power ON or initial start phase(s)	1–8	Yes		2-6
4	External start phase(s)	Selects external start phase	1–8	Yes		2-6
5	Call-to-nonactuated I phase(s)	Selects phase(s) assigned to call-to-nonactuated I input	1–8	Yes		0
6	Call-to-nonactuated II phase(s)	Selects phase(s) assigned to call-to-nonactuated II input	1–8	Yes		0
7	Guaranteed passage phase(s)	Programs a phase(s) to have the guaranteed passage option	1–8	Yes		0
8	Pedestrian only phase(s)	Programs a phase(s) to have only a pedestrian movement associated with it	1–8	Yes		0
9	Special programmable option	Selects use of programmable options below: 1 = Dual entry 2 = Simultaneous gap termination 3 = Actuated rest in walk 4 = Pedestrian clearance protection	1–4	Yes		1

Mode Programming

Setting	Function
00	0. NO PHASE (eliminates phase)
01	1. NONLOCKING (detector)
02	2. LOCK (detector)
03	3. VEH RECALL
04	4. PED RECALL
05	5. PED RECALL & MAX RECALL
06	6. MAX RECALL
07	7. NONACT
IX	FLASHING WALK (X=0-7 above)

NOTES

FULLY ACTUATED - ISOLATED

PROGRAM

FLASH (LOCAL) 00:30 - 05:30 DAILY
MAX II (LOCAL) - 06:00 - 09:00 MON-FRI
Ø1 LOOP DELAY @ 5"
Ø4 R.T. LOOP DELAY @ 10"
Ø4 L.T. LOOP & CALL LOOPS @ 4"

SUBMITTED BY TR **DATE** 3-21-88 **CHECKED BY** BCM **DATE** 3/21/88 **APPROVED BY** BCM **DATE** 3/21/88
IN SERVICE BY SHA **DATE** 4-21-88 **TIME** 10:00 A **CHECKED BY** _____ **DATE** _____ **TIME** _____

SEQUENCE OF OPERATION SHEET

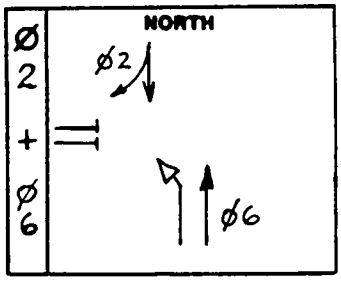
TRAFFIC OPERATIONS SECTION
DIVISION OF TRAFFIC ENGINEERING
MONTGOMERY COUNTY, MARYLAND

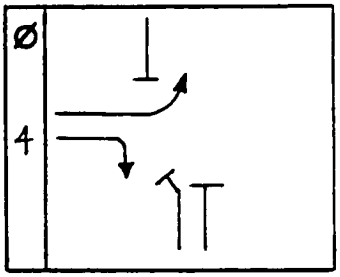
NO. 365

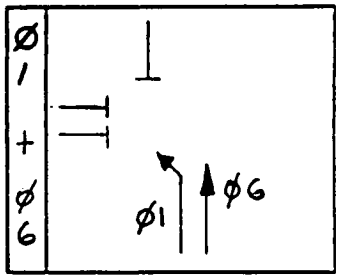
INTERSECTION: AIRPARK ROAD AND G'BURG/LAYTONSVILLE RD. (MD 124)

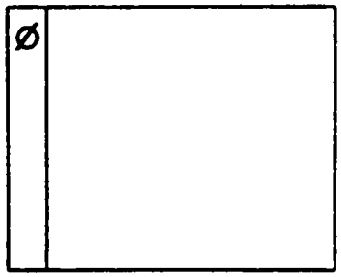
	SIGNAL HEAD INDICATIONS					
SIGNAL NO.	2-6		1			
TOTAL:	5		1			
LEGEND OPTICALLY LIMITED R - RED Y - YELLOW G - GREEN ← ARROW F - FLASHING	R Y G 12"	R Y G G 12"	R ~~Y~~ Y ~~G~~ G 12"	R Y G 8"		9" or 12"

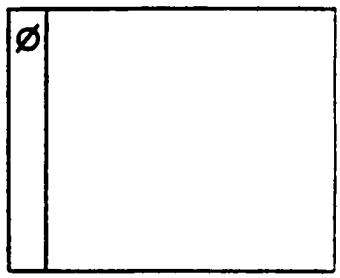
PHASING











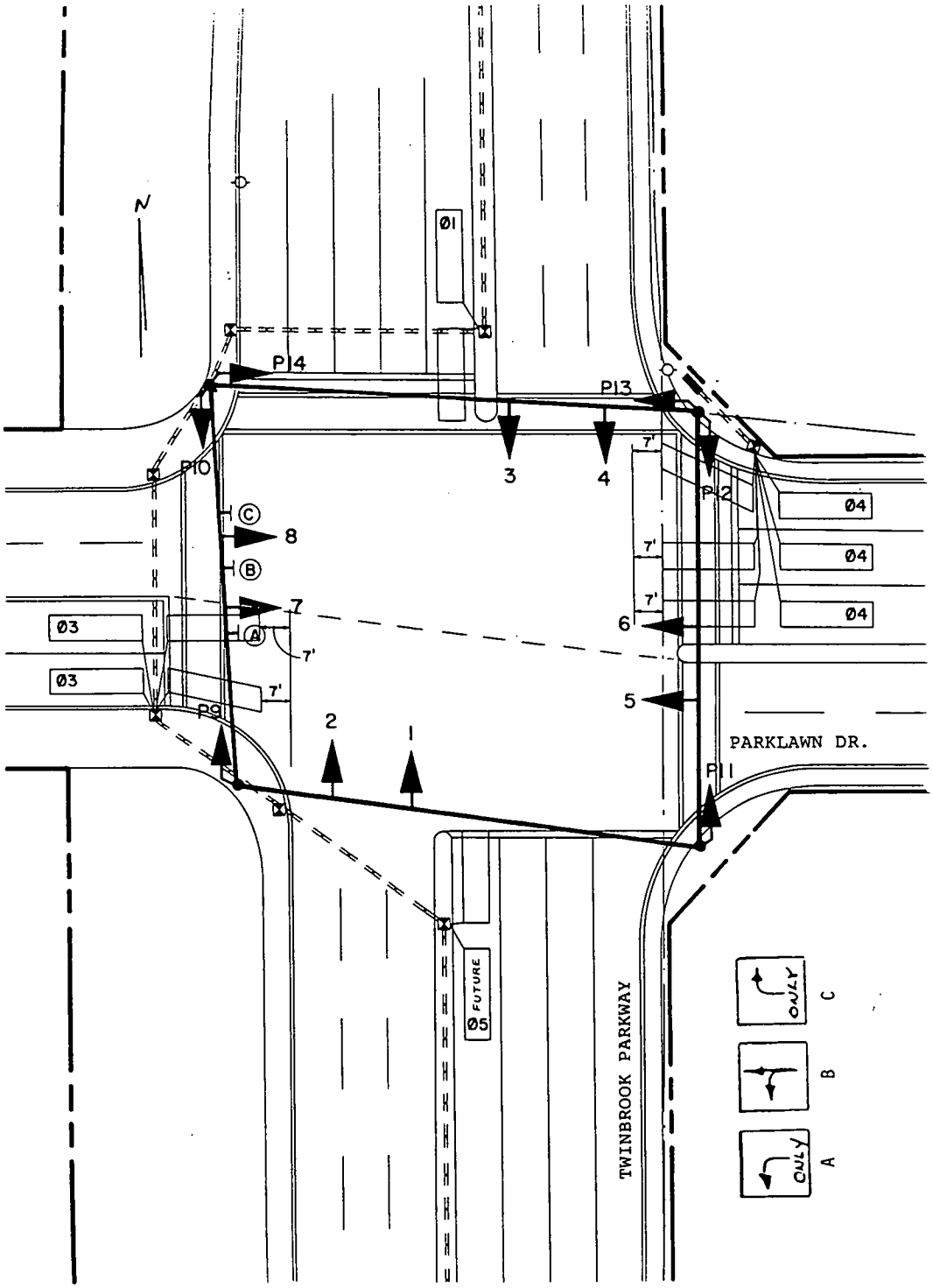
SEQUENCE OF OPERATION

SIGNAL NO.	INTERVAL 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	FLASH
1	G	Y①	R①	R	R	R	←G	←Y									
2	G	Y①	R①	R	R	R	G	G									
3	G	Y	R	R	R	R	R	R									
4	G	Y	R	R	R	R	R	R									
5	R	R	R	G	Y	R	R	R									
6	R	R	R	G	Y	R	R	R									
7																	
8																	
9																	
10																	
11																	
12																	
13																	
14																	
15																	
16																	
17																	
18																	
19																	
20																	
21																	
22																	
23																	
24																	
PHASE	Ø2+Ø6		ALL RED	Ø4		ALL RED	Ø1+Ø6										

NOTES:
① IF Ø4 IS SKIPPED SIGNALS #1 & #2 WILL BE G

SUBMITTED: AS PER S.H.A. CHECKED: _____ APPROVED: _____
IN SERVICE BY: _____ DATE: _____ TIME: _____

PARKLAWN DRIVE AND TWINBROOK PARKWAY



SEQUENCE OF OPERATION SHEET

TRAFFIC OPERATIONS SECTION
DIVISION OF TRAFFIC ENGINEERING
MONTGOMERY COUNTY, MARYLAND

NO. 4034E

INTERSECTION: Parklawn Drive & Twinbrook Parkway

SIGNAL NO.	2, 4, 5, 6	7	1, 3		8	9-14
TOTAL:	4	1	2		1	6
SIGNAL HEAD INDICATIONS	R, Y, G	R, Y, G, ←G	R, ←Y, Y, ←G, G	R, Y, G	R, Y, ←Y, G, ←G	✋, 🚶
	12"	12"	12"	8"	12"	9" or 12" ✓

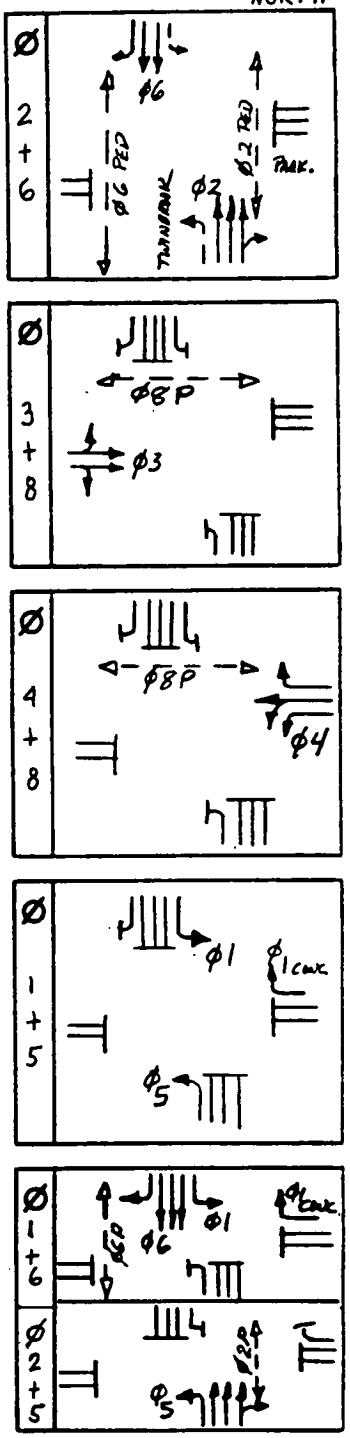
LEGEND
- ▢ OPTICALLY LIMITED
- R - RED
- Y - YELLOW
- G - GREEN
- ← ARROW
- F - FLASHING

SEQUENCE OF OPERATION

SIGNAL NO.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	FLASH
1	G	G	Y①	R①	R	R	R	R	R	R	R	R	R,←G	R,←Y,Ⓢ	G,←G	R R	Y
2	G	G	Y①	R①	R	R	R	R	R	R	R	R	R	R	G G	R R	Y
3	G	G	Y②	R②	R	R	R	R	R	R	R	R	R,←G	R,←Y,Ⓢ	R R	G,←G G,←G	Y
4	G	G	Y②	R②	R	R	R	R	R	R	R	R	R	R	R R	G G	Y
5	R	R	R	R	G	Y	R	R	R	R	R	R	R	R	R R	R R	R
6	R	R	R	R	G	Y	R	R	R	R	R	R	R	R	R R	R R	R
7	R	R	R	R	R	R	R	G,←G	G,←G	G,←G	Y	R	R	R	R R	R R	R
8	R	R	R	R	R	R	R	G	G	G	Y	R	R,←G	R,←Y,Ⓢ	R,←G R,←Y	R R	R
9	WF	DWF	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	WF WF	DW DW	—
10	WF	DWF	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	WF WF	DW DW	—
11	WF	DWF	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW DW	WF WF	—
12	WF	DWF	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW	DW DW	WF WF	—
13	DW	DW	DW	DW	WF	WF	WF	WF	DWF	DW	DW	DW	DW	DW	DW DW	DW DW	—
14	DW	DW	DW	DW	WF	WF	WF	WF	DWF	DW	DW	DW	DW	DW	DW DW	DW DW	—
15																	
16																	
17																	
18																	
19																	
20																	
21																	
22																	
23																	
24																	
PHASE	2+6			ALL RED	3+8		ALL RED	4+8				ALL RED	1+5		1+6	2+5	

PHASING

NORTH



NOTES:
1. IF Ø1+6 IS NEXT, HEADS 1 & 2 WILL REMAIN G
2. IF Ø2+5 IS NEXT, HEADS 3 & 4 WILL REMAIN G
3. IF Ø1+6 IS NEXT, HEAD 1 WILL REMAIN R←G
4. IF Ø2+5 IS NEXT, HEAD 3 WILL REMAIN R←G
5. IF Ø1+6 IS NEXT, HEAD 8 WILL REMAIN R←G

Ø8 IS FOR PEDS

SUBMITTED: G.S.D. 10/26/84 CHECKED: _____ APPROVED: JCR 11-1-84
IN SERVICE BY: GW DATE: 12/5/84 TIME: 0700

TRAFFIC SIGNAL TIMING SHEET
TRAFFIC OPERATIONS SECTION
Division of Traffic Engineering
Montgomery County, Maryland

INT. 46

INTERSECTION: PARKLAWN DR. & TWINBROOK PKWY.

NO. 4034 E.0

STEP 3 PRESS "INT"

STEP 2 PRESS "#"	PHASE STEP 1 "PRESS Ø"	MEMORY	PED SELECT	RECALL	"MGR" MIN GRN	"PSG"	"YEL"	"RED"	"Mx 1"	"Mx 2"	"WLK"	"PCL" DONT WALK	"S/A" SEC ACT	"TBR" TIME BEF RED	"TTR" TIME TO RED	"MGP" MIN GAP	"Mx I" MAX VAR INIT
					TIME IN SECONDS												
(1)	SB TWIN LT	OFF	—	OFF	2	2	3	0	40	40	0	0	0	0	0	0	99
(2)	NB TWIN	PED ON	WF	EXT	10	50	4	1	99	99	7	12	0	0	0	0	99
(3)	EB PARK	OFF	—	OFF	3	3	4	0	30	30	3	0	0	0	0	0	99
(4)	WB PARK	OFF	—	OFF	3	3	4	0	40	40	0	0	0	0	0	0	99
(5)	NB TWIN LT	OFF	—	OFF	2	2	3	0	30	30	0	0	0	0	0	0	99
(6)	SB TWIN	PED ON	WF	EXT	10	50	4	1	99	99	7	12	0	0	0	0	99
(7)																	
(8)	PED	OFF	WF	OFF	0	0	4	0	0	0	7	18	0	0	0	0	99

STEP 4 PRESS

STEP 5 PRESS "VAL"

STEP 6 PRESS

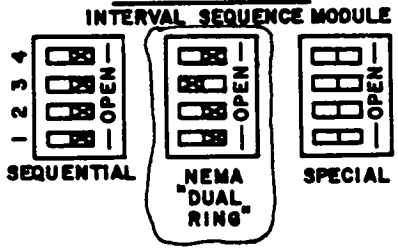
STEP 7 PRESS "LOAD"

STEP 2 PRESS

STEP 1 PRESS "Ø" → "Ø" → "INT"	"Mx I" START-UP	"Mx 2" NON ACT PHASE I	"WLK" NON ACT PHASE II	"PCL" RED REVERT (SEC)	"S/A" LAST CAR PAS	"TBR" OVER LAP SEQ	"RED" DUAL ENTRY
CODE	89	34	2	0	0	0	1
DESCRIPT	2+6	2+6	2	0	0	1+4	ON

STEP 3 PRESS "VAL" +

CROUSE-HINDS DMK2 CONTROLLER
PHASE SEQUENCE
INTERVAL SEQUENCE MODULE



DUAL ENTRY			
STEP 1 PRESS "CLR" → "Ø"			
STEP 2 PRESS	STEP 3 PRESS "VAL"	(PHASE)	(ASSOCIATED PHASE)
"1"		"5"	
"2"		"6"	
"3"		"7"	
"4"		"8"	

PROGRAM

1. REMOTE FLASH: 0030 TO 0600 (7 DAYS)
2. NON-ACTUATED OPERATION
3. INHIBIT MAX OFF – WALK REST ON
4. A PED CALL FOR Ø8 WILL PLACE A MINIMUM CALL ON Ø3 WITH ANY REMAINDER OF Ø8 W-DW TO TIME CONCURRENTLY WITH Ø4

NOTES

5. HOLD ON Ø2

SUBMITTED BY GSD DATE 10/26/84 CHECKED BY _____ DATE _____ APPROVED BY JCR DATE 11-1-84
IN SERVICE BY GW DATE 12/5/84 TIME 0700 CHECKED BY _____ DATE _____ TIME _____

TRAFFIC SIGNAL TIMING SHEET
TRAFFIC OPERATIONS SECTION
Division of Traffic Engineering
Montgomery County, Maryland

Crouse-Hinds DMK
Sheet 2

INTERSECTION: PARKLAWN DR. & TWINBROOK PKWY

NO. 4034 E

CYCLE MONITOR
PRESS 'CORD' +('CYC' OR 'SPLT' OR 'OFST')

RING 1 DISPLAY | a | b

a = blank (free)
a = # (SYSTEM)

b = cycle/split/offset in effect

STEP 1 PRESS 'CLR' + 'CORD' + 'CYC'

CYCLE LENGTH STEP 2 PRESS →	CYC '1' + 'VAL' + '90'			CYC '2' + 'VAL' + '120'			CYC '3' + 'VAL' + '120'			CYC '4' + 'VAL' + '120'		
OFFSETS → STEP 2 PRESS '#' + 'CORD'	OFST 1 VAL 70	OFST 2 VAL __	OFST 3 VAL __	OFST 1 VAL 20	OFST 2 VAL __	OFST 3 VAL __	OFST 1 VAL 2	OFST 2 VAL __	OFST 3 VAL __	OFST 1 VAL 2	OFST 2 VAL __	OFST 3 VAL __

'CLR' + COORD +	#	+VAL
CYCLE SELECTION	1	0
SPLIT SELECTION	2	0
FREE/SYS SELECTION	3	1
OFFSET SELECTION	4	0
OFFSET SEEKING MODE	5	2

'CLR' + COORD +	#	+VAL
SYS TO FREE SELECTION	6	0
1ST COORD PHASE	7	2
2ND COORD PHASE	8	6
PED PERMISSIVE	PERM	1

PHASE ASSOCIATION

STEP 3 PRESS →

STEP '1' PRESS 'CLR' + 'CORD'		'Ø' '1'	'Ø' '2'	'Ø' '3'	'Ø' '4'	'Ø' '5'	'Ø' '6'	'Ø' '7'	'Ø' '8'
STEP 2	'FRC' + '1'	0	0	1	0	0	0	0	0
0 = NO FORCE	'FRC' + '2'	0	0	0	1	0	0	0	0
1 = FORCE	'FRC' + '3'	1	0	0	0	1	0	0	0
	'FRC' + '4'	0	0	0	0	0	0	0	0
	'FRC' + '5'	0	0	0	0	0	0	0	0
	'FRC' + '6'	0	0	0	0	0	0	0	0

OR

STEP 2	'PERM' '1'	0	0	0	0	0	0	1	0
0 = ALLOW	'PERM' '2'	1	1	1	1	1	1	1	1
1 = OMIT	'PERM' '3'	1	1	1	1	1	1	1	1
	'PERM' '4'	1	1	1	1	1	1	1	1
	'PERM' '6'	1	1	1	1	1	1	1	1

STEP 4 PRESS 'VAL' +
STEP 5 'LOAD'

PHASE TIME INFORMATION

STEP 1 PRESS 'CLR' + 'CORD'	'FRC' '1'	'FRC' '2'	'FRC' '3'	'FRC' '4'	'FRC' '5'	'FRC' '6'
'CYC' '1' + 'CORD' 'SPLT' '1' + 'CORD'	'VAL' 32	'VAL' 52	'VAL' 68	'VAL' __	'VAL' __	'VAL' __
STEP 2 PRESS → 'CYC' '2' + 'CORD' 'SPLT' '1' + 'CORD'	37	51	85	__	__	__
'CYC' '3' + 'CORD' 'SPLT' '1' + 'CORD'	32	60	78	__	__	__
'CYC' '3' + 'CORD' 'SPLT' '2' + 'CORD'	32	60	78	__	__	__

(SECONDS)

'PERM' '1' ST	'PERM' '1' END	'PERM' '2' ST	'PERM' '2' END	'PERM' '3' ST	'PERM' '3' END	← STEP 3 PRESS
'VAL' 00	'LOAD' 01	'VAL' __	'LOAD' __	__	__	← STEP 4 PRESS
00	01	__	__	__	__	
00	01	__	__	__	__	STEP 5 LOAD
00	01	__	__	__	__	

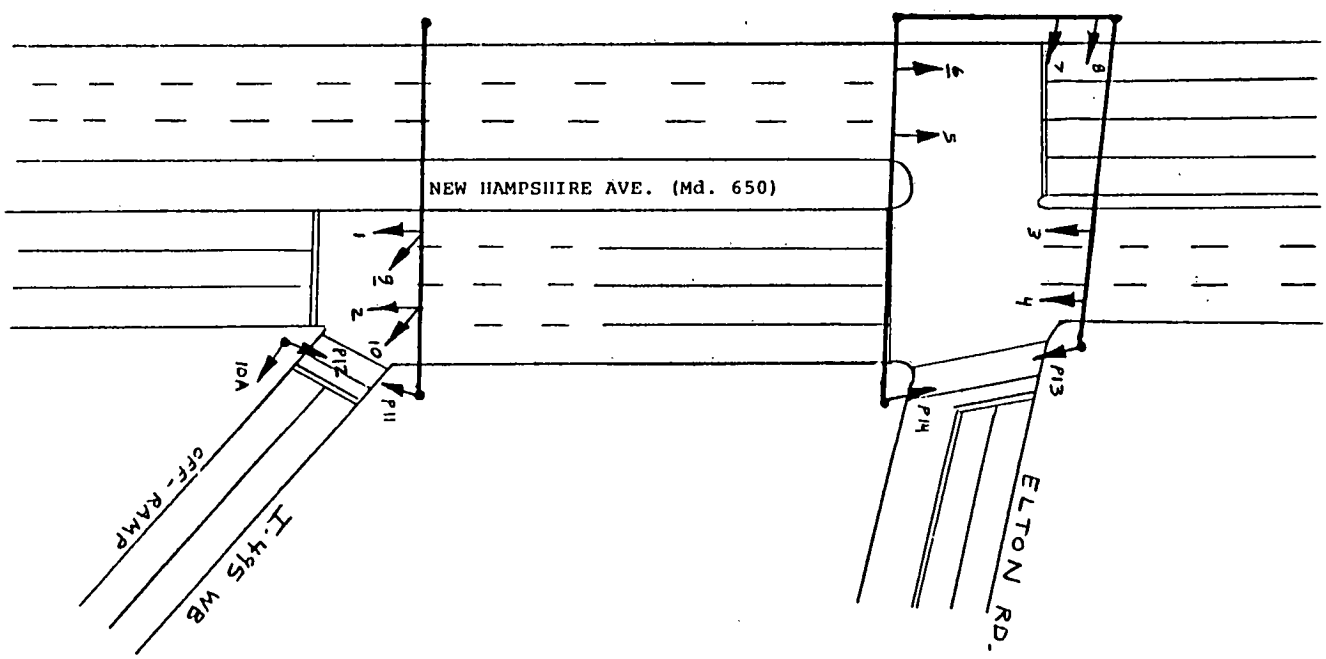
PROGRAM

NOTE: THESE COORDINATION AND SPLIT SETTINGS ARE OFF-LINE BACK-UP ONLY. NORMALLY, THE COMTRAC SYSTEM WILL GENERATE CYCLE, SPLIT, & OFFSET.

SUBMITTED BY GSD DATE 10/31/84 CHECKED BY ____ DATE ____ APPROVED BY JCR DATE 11-1-84
IN SERVICE BY GW DATE 12/5/84 TIME 0700 CHECKED BY ____ DATE ____ TIME ____

```
OPEN CODE:

INITIALS: BCM

CONSOLE INPUT OPENED AT 03/03/87  10:28:26  BCM
INAM 46
 46       PARKLAWN-TWINBROOK

03/03/87  10:28:37  BCM
RLOG ITPR 46
03/03/87  10:28:44  BCM


03/03/87  10:28:47  INTERSECTION TIMING PROFILE

INT                                                  % INCLUDING
46                                                   CLEARANCE FROM
                                                     PREVIOUS PHASE
LEVEL-1 BAL   OFFSET-1  73 %  SPLIT-1  INDEX-1   SPLIT 33/31/18/18
LEVEL-2 BAL   OFFSET-2  73 %  SPLIT-2  INDEX-1   SPLIT 33/31/18/18
LEVEL-2 INB   OFFSET-3  73 %  SPLIT-3  INDEX-1   SPLIT 33/31/18/18
LEVEL-2 OUTB  OFFSET-4  73 %  SPLIT-4  INDEX-1   SPLIT 33/31/18/18
LEVEL-3 BAL   OFFSET-5  80 %  SPLIT-5  INDEX-8   SPLIT 42/25/18/15
LEVEL-3 INB   OFFSET-6  80 %  SPLIT-6  INDEX-8   SPLIT 42/25/18/15
LEVEL-3 OUTB  OFFSET-7  80 %  SPLIT-7  INDEX-8   SPLIT 42/25/18/15
LEVEL-4 BAL   OFFSET-8  48 %  SPLIT-8  INDEX-8   SPLIT 42/25/18/15
LEVEL-4 INB   OFFSET-9  78 %  SPLIT-9  INDEX-2   SPLIT 30/20/14/36
LEVEL-4 OUTB  OFFSET-10 65 %  SPLIT-10 INDEX-11  SPLIT 47/21/18/14
LEVEL-5 BAL   OFFSET-11 45 %  SPLIT-11 INDEX-11  SPLIT 47/21/18/14
LEVEL-5 INB   OFFSET-12 63 %  SPLIT-12 INDEX-2   SPLIT 30/20/14/36
LEVEL-5 OUTB  OFFSET-13 60 %  SPLIT-13 INDEX-11  SPLIT 47/21/18/14
LEVEL-6 BAL   OFFSET-14 39 %  SPLIT-14 INDEX-11  SPLIT 47/21/18/14
LEVEL-6 INB   OFFSET-15 66 %  SPLIT-15 INDEX-2   SPLIT 30/20/14/36
LEVEL-6 OUTB  OFFSET-16 52 %  SPLIT-16 INDEX-11  SPLIT 47/21/18/14


RLOG IHWT 46
03/03/87  10:29:33  BCM
```

ELTON ROAD-I-495 RAMP AND NEW HAMPSHIRE AVENUE (MD 650)



SEQUENCE OF OPERATION SHEET

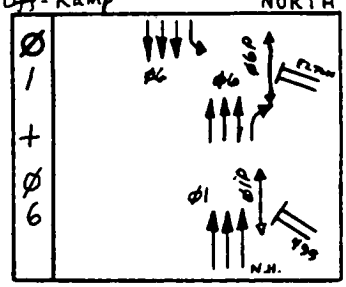
TRAFFIC OPERATIONS SECTION
DIVISION OF TRAFFIC ENGINEERING
MONTGOMERY COUNTY, MARYLAND

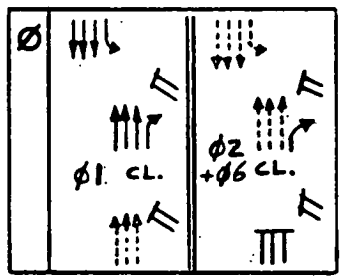
NO. 3069-A

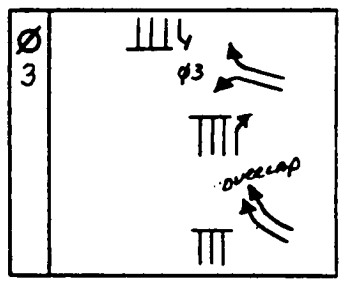
INTERSECTION: Elton Rd. - New Hampshire Ave. (Md. 650) - I-495 Westbound Off-Ramp

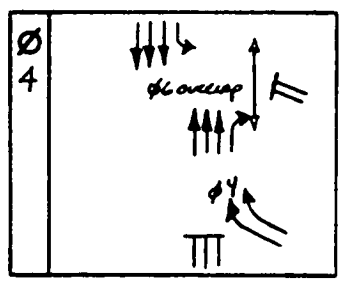
SIGNAL NO.	1, 2, 5-10A	4		3		P 11-14
TOTAL:	9	1		1		4
LEGEND (O OPTICALLY LIMITED; R - RED; Y - YELLOW; G - GREEN; ← ARROW; F - FLASHING)	R Y G 12"	R Y G (G←) 12"	R Y Y G G 12"	R Y G 12"		9" or 12"

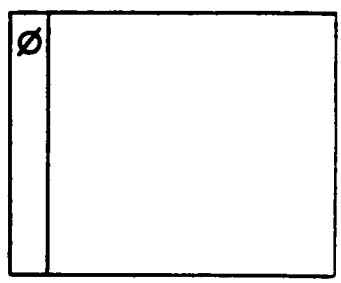
PHASING











SEQUENCE OF OPERATION

SIGNAL NO.	INTERVAL 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	FLASH
1	G	G	G	Y	R	R	R	R	R				Y				
2	G	G	G	Y	R	R	R	R	R				Y				
3	G	G	G	G	Y	R	R	G	G				Y				
4	G	G	G	G	G→ Y	G→ R	G→ R	G	G				Y				
5	G	G	G	G	Y	R	R	G	G				Y				
6	G	G	G	G	Y	R	R	G	G				Y				
7	R	R	R	R	R	G	Y	R	R				R				
8	R	R	R	R	R	G	Y	R	R				R				
9	R	R	R	R	R	G	G (3)	G	Y				R				
10,10A	R	R	R	R	R	G	G (3)	G	Y				R				
11	W	DWF	DW	DW	DW	DW	DW	DW	DW				—				
12	W	DWF	DW	DW	DW	DW	DW	DW	DW				—				
13	WF	DWF	DW	DW	DW	DW	DW	WF	WF				—				
14	WF	DWF	DW	DW	DW	DW	DW	WF	WF				—				
15																	
16																	
17																	
18																	
19																	
20																	
21																	
22																	
23																	
24																	
PHASE	Ø1 + Ø6				Ø2 & Ø6 (2)	Ø3		Ø4					FLASH				

NOTES:

(1) SIGNALS 4 AND 3 ARE OPTICALLY LIMITING

(2) IF Ø3 IS SKIPPED, "Ø2+Ø6" SHALL BE DELETED FROM THE SEQUENCE
IF Ø3 IS NOT SKIPPED, "Ø2+Ø6" SHALL PRECEDE Ø3

(3) IF Ø4 IS SKIPPED, SIGNALS 9 & 10 WILL BE Y.

SUBMITTED: WSW 10/4/77 CHECKED: _____ APPROVED: _____
IN SERVICE BY: CR DATE: 10/18/79 TIME: 9:30A

TRAFFIC SIGNAL TIMING SHEET
TRAFFIC OPERATIONS SECTION
Division of Traffic Engineering
Montgomery County, Md.

Eagle DP 900
Crouse-Hinds DM 800

INTERSECTION: Elton Rd. – I-495 Ramp-
New Hampshire Ave (Md. 650) NO. A.1 (299)

	PHASE	TIME IN SECONDS														
		MIN. INIT.	PASS.	SEC. ACT.	MAX. INIT.	SLOPE	MIN. GAP	MAX. 1	MAX. 2	YEL-LOW	ALL RED	WALK	DON'T WALK	NON COOK CALL MEM	VEH. RE-CALL	PED. RE-CALL
1	NB Md. 650 at Ramp (w/P11 + P12)	10	9.9					50	50	4.0	1.0	7	10	ON	EXT	ON
2	Special Clearance (NB + SB Md. 650 at Elton)	0	0					0	0	4.0	1.0	0	0	OFF	OFF	OFF
3	Elton (w/Ramp + NBRT OL)	3	2					30	30	4.0	0	0	0	OFF	OFF	OFF
4	I-495 Ramp (w/Ø6 OL)	3	3					60	60	4.0	0	0	0	ON	OFF	OFF
5	OMIT															
6	NB + SB Md. 650 at Elton (w/P13 + P14)	10	9.9					50	50	4.0	1.0	7	20	ON	EXT	ON
7	OMIT															
8	OMIT															

DIAL COORDINATING UNIT

PHASE	FO NO	FORCE-OFF KEYS, DIAL (%) D1	D2	D3	SPLIT, SECONDS D1	D2	D3
Ø1	Ø1	01"	01"	01"	50"	77"	62"
Ø2+6		(Not Forced Off- Termination by controller)					
Ø3	FO 1	25"	25"	30"	15"	15"	20"
	FO 2	(Not used – future phase)					
Ø4	FO 3	36"	39"	54"	7"	10"	20"

	OFFSET KEYS, DIAL (%) D1	D2	D3	OFFSET, SECONDS D1	D2	D3
OFFSET 1				21"	22"	15"
OFFSET 2						
OFFSET 3						
CYCLE				90	120	120

PROGRAM

C.H. DM 800 + DIGITAL COORD. UNIT
REMOTE FLASH: 0030-0600 (7 DAYS)

1. Ø2 SPECIAL CLEARANCE ONLY COMES IN WHEN Ø3 IS NEXT. IT IS SKIPPED IF Ø3 IS SKIPPED.
2. Ø2 IS CALLED BY DETECTORS ON Ø3 (ELTON RD.)
3. NON ACT I - OFF - BACKUP
   NON ACT II - ON - COMPUTER
4. ALL WALK INTERVALS TO BE STEADY
5. WALK REST MODIFIER ON
6. Ø1 + 6 DETECTOR GROUNDED
7. HOLD Ø1 ONLY
8. WHEN ON-LINE WITH COMPUTER THERE IS A RELAY THAT DOES NOT ALLOW THE Ø4-RETURN BIT TO BE SEEN BY COMPUTER WHILE Ø3 IS ON. IMPLEMENTED 8/4/86

SUBMITTED BY BC DATE 7/30/86 CHECKED BY BCM DATE 8/6/86 APPROVED BY [signature] DATE 8/7/86
IN SERVICE BY BC DATE 8/4/86 TIME 2:00p.m. CHECKED BY _____ DATE _____ TIME _____

```
INAM 299
 299        ELTON-I495-MD650

08/06/86  09:17:41  WWC
RLOG IHWT 299
08/06/86  09:17:52  WWC


08/06/86  09:17:55  INTERSECTION H/W TIMING REPORT

INT                GG'S    CL'S

299  PHASE-1        10      10
299  PHASE-2        10      4
299  PHASE-3        3       4

RLOG ITPR 299
08/06/86  09:18:13  WWC


08/06/86  09:18:15  INTERSECTION TIMING PROFILE

INT
299
LEVEL-1 BAL   OFFSET-1  92 %  SPLIT-1  INDEX-1  SPLIT 48/39/13
LEVEL-2 BAL   OFFSET-2  92 %  SPLIT-2  INDEX-1  SPLIT 48/39/13
LEVEL-2 INB   OFFSET-3  92 %  SPLIT-3  INDEX-1  SPLIT 48/39/13
LEVEL-2 OUTB  OFFSET-4  92 %  SPLIT-4  INDEX-1  SPLIT 48/39/13
LEVEL-3 BAL   OFFSET-5  92 %  SPLIT-5  INDEX-1  SPLIT 48/39/13
LEVEL-3 INB   OFFSET-6  92 %  SPLIT-6  INDEX-1  SPLIT 48/39/13
LEVEL-3 OUTB  OFFSET-7  92 %  SPLIT-7  INDEX-1  SPLIT 48/39/13
LEVEL-4 BAL   OFFSET-8  92 %  SPLIT-8  INDEX-3  SPLIT 60/21/19
LEVEL-4 INB   OFFSET-9  75 %  SPLIT-9  INDEX-2  SPLIT 59/29/12
LEVEL-4 OUTB  OFFSET-10 82 %  SPLIT-10 INDEX-3  SPLIT 60/21/19
LEVEL-5 BAL   OFFSET-11 0  %  SPLIT-11 INDEX-1  SPLIT 48/39/13
LEVEL-5 INB   OFFSET-12 0  %  SPLIT-12 INDEX-1  SPLIT 48/39/13
LEVEL-5 OUTB  OFFSET-13 0  %  SPLIT-13 INDEX-1  SPLIT 48/39/13
LEVEL-6 BAL   OFFSET-14 0  %  SPLIT-14 INDEX-1  SPLIT 48/39/13
LEVEL-6 INB   OFFSET-15 0  %  SPLIT-15 INDEX-1  SPLIT 48/39/13
LEVEL-6 OUTB  OFFSET-16 0  %  SPLIT-16 INDEX-1  SPLIT 48/39/13
```

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The National Academy of Engineering was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. Robert M. White is president of the National Academy of Engineering.

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