HIGHWAY RESEARCH RECORD

Number 445 | Traffic Signals

5 reports
prepared for the
52nd Annual Meeting

Subject Areas

51 Highway Safety
52 Road User Characteristics
53 Traffic Control and Operations

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

Washington, D.C. 1973

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ISBN 0-309-02176-6
Library of Congress Catalog Card No. 73-11611
Price: $2.00

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CONTENTS

FOREWORD

The papers in this RECORD deal generally with traffic control devices with particular emphasis on traffic signals.

In the first paper, Dare and Jomini describe a signalized intersection suitable for speed signal funnel installation. A speed signal funnel incorporating three variable-message speed signals was designed for each of two major approaches at the study intersection. It was determined that the speed signal funnel yielded benefit-cost ratios ranging from 2.0:1.0 to 12.0:1.0 depending on the assumptions underlying the benefit-cost computation.

Gartner discusses an optimal scheme for coordination of consecutive signals along arterial routes or networks, which requires a microscopic analysis of the traffic flow patterns on every link of the system. Analysis of such a scheme was carried out for two-way links on a major artery in downtown Toronto. Accurate platoon profiles were obtained via the digital computer system controlling the traffic lights throughout the metropolitan area and its associated vehicle-detector system.

Messer, Whitson, Dudek, and Romano review a traffic signal progression program that can maximize progression along a facility having multiphase signals. A copy of the progression program was adopted in the real-time control of an arterial pilot control system in Dallas.

Plummer and King report on a special study related to the meaning and application of color and arrow indications for traffic signals. A controlled laboratory study, using both color movies and color slides, investigated 19 signal indications for their effectiveness in conveying their intended message to the driver. Based on the analysis of driver performance data recorded in a field study, a single indicator was recommended.

In the final paper, Hanscom reports on an analytic survey made of the highway signing and sign-maintenance inventory systems in each of the districts of the Virginia Department of Highways. The survey revealed a diversity of engineering opinions regarding the need for and the application of sign inventories.

BENEFIT-COST ANALYSIS OF A SPEED SIGNAL FUNNEL

Charles E. Dare, Department of Civil Engineering, Iowa State University; and
Pierre-Andre Jomini, Traffic Engineer, Billings, Montana

The objective of this research was to establish an estimate of the economic feasibility of modifying an intersection traffic control system to incorporate a speed signal funnel. An appropriate high-speed intersection currently under traffic-actuated control was selected for this evaluation. Data on traffic volumes, delays, approach speed profiles, and accident experience were gathered for the study site so future costs of retaining the present control system could be estimated. A speed signal funnel incorporating three variable-message speed signals was then designed for each of the two major approaches at the intersection. Estimates specifying equipment costs, maintenance costs, vehicle operation costs, time costs, and accident costs were developed for the proposed speed signal funnel. The economic desirability of the speed signal funnel was determined by means of an incremental benefit-cost ratio. It was found that the speed signal funnel yielded benefit-cost ratios ranging from 1.5:1.0 to as high as 12.0:1.0 depending on the assumptions underlying the computation.

•ALTHOUGH experiments have been conducted with the speed signal funnel concept in the United States, only cursory investigation (7) has been performed to determine the economic feasibility of this type of intersection control system. It is the purpose of this study to establish an index, in the form of an incremental benefit-cost ratio, that will be appropriate for comparing a speed signal funnel with other potential highway improvement projects. The analyses reported relate to the expected costs and benefits associated with retaining the existing standard control devices at a signalized intersection and the expected costs and benefits if the intersection is converted to speed signal funnel control.

PREVIOUS STUDIES

An extensive report of experiments concerning modification of vehicle approach speeds at signalized intersections is given by von Stein (11). As early as 1954, he had installed various combinations of presignals and variable-message speed signals for intersection traffic control in Germany. He recommended that a series of three speed-advisory messages be used along each intersection approach of roadways with an approach speed of about 45 mph. With this traffic control system, a driver approaching a signalized intersection encounters several signals advising him of the correct speed to assume in order to arrive at the intersection during the green phase.

The only significant installation involving the speed signal concept in the United States was the traffic pacer installed in Warren, Michigan, by the General Motors Research Laboratories in 1961 (1, 9). The traffic pacer incorporated 33 speed signals, 11 presignals, and nine intersection traffic signals located throughout a 4-mile length of divided four-lane expressway.

It was reported that the traffic pacer reduced the average trip time and the average number of stops of a vehicle traveling through the test section as compared to a past system and a progressive system. A detailed accident comparison was not presented; however, a general comparison with the accident trend within the county and for a

Publication of this paper sponsored by Committee on Traffic Control Devices.

similar parallel roadway indicated a substantial improvement in the accident experience for the traffic pacer route.

A more detailed economic analysis of the traffic pacer installation was subsequently performed by Hulbert (7) in 1964. His study evaluated road user benefits for the main route northbound traffic only, without consideration of accident costs. It was assumed that side road traffic was unaffected by the traffic pacer. Hulbert found that the rate of return offered by investing in the traffic pacer installation was as high as 1,350 percent when compared to the past system. The data from this study are summarized in Appendix A, where it is shown that the incremental benefit-cost ratio for the traffic pacer may have reached a value of 72.2:1.0.

Computer simulation studies of the speed signal funnel were performed by Dare (3, 4) in 1968. In these studies the feasibility of combining variable-message speed signals with a semi-actuated controller was explored. It was found that a speed signal funnel could function successfully with a semi-actuated controller provided proper vehicle detection devices were utilized on the minor approach. These studies showed that the signal funnel could theoretically eliminate vehicle stoppages at the intersection. An economic analysis of this system was not performed.

STUDY SITE

To determine the benefits and costs to be expected from a signal funnel installation, we selected an isolated high-speed signalized intersection for detailed evaluation. The intersection is a four-leg intersection formed by Colo-121 and West 80th Avenue at the north city limit of Arvada, Colorado. It is located in a rapidly developing rural-urban transition area with gently rolling topography.

Colo-121 is a divided four-lane highway with separate left- and right-turn lanes at the intersection. The north approach has a posted 60-mph speed limit to a point approximately ¼ mile north of the intersection, where the speed limit is reduced to 50 mph. The south approach has a limit of 50 mph for more than 1 mile preceding the intersection.

West 80th Avenue is a two-lane, two-way arterial street with 45-mph speed limits decreasing to 25 mph near the intersection. Its approaches are widened at the intersection to facilitate right-turn vehicle movements. Current signalization is a two-phase, fully actuated controller with adjustments as given in Table 1.

The 1971 ADT and peak-hour volumes obtained by field studies are shown in Figure 1. Truck traffic was found to range from 2 to 5 percent, 4 percent being a typical value during daytime periods.

A summary of the accident experience during the years 1961-1964 and 1967-1971 is given in Table 2. During the former time period, traffic was controlled by a two-way stop; after 1964 the intersection was regulated by two-phase signalization. The broadside collision was the predominate accident type in 1961-1964, while the turning movement and broadside collisions were most frequent in 1967-1971.

SIGNAL FUNNEL DESIGN

Planning and Design

Numerous interrelated factors must be considered in planning and installing a speed signal funnel. It is essential that the advance variable-message speed signals be properly located in advance of the intersection. This problem has been explored to a certain extent by Breuning (2), and he has shown total funnel length to be primarily dependent on the following:

1. The unimpeded approach speed of the vehicles,
2. The slower approach speed advised to vehicles, and
3. The red phase duration on approaches with funneled traffic.

In practice, one must also consider the traffic volumes and capacity of the intersection; the sequence of the phases at the intersection; and speed signal and intersection signal visibility limitations arising from roadway alignment, driver response, and vehicular deceleration characteristics.

The first phase of the design process in this study was to evaluate the ability of the study intersection to accommodate the anticipated traffic demands. Figure 2 shows the projected 1972 ADT values and the peak-hour volumes as determined by extrapolating volume trends for the location and applying adjustment factors to recognize additional traffic generated by proposed nearby shopping centers.

An intersection capacity study indicated that, with the current intersection geometrics and signal phasing, several traffic movements would be operating at level of service D or E during peak hours in 1972. It was concluded that additional turning lanes on both the east and west approaches would achieve smoother operation during peak hours. Furthermore, capacity and safety factors necessitated the introduction of protected left-turn signal phases for the north, south, and east approaches.

The selection of a cycle length and optimum phasing sequence for the intersection controller was then considered. After cycle lengths ranging from 60 to 90 sec and several potential phasing sequences were explored, it was determined that a 70-sec cycle incorporating the phase sequence shown in Figure 3 would provide operation at level of service B or C for all movements.

Variable-Message Speed Signals

The number and placement of the variable-message speed signals and the speeds displayed are critical aspects of the speed signal funnel design. An approximate location for the outermost speed signal may be determined from the relationship developed by Breuning (2):

$$S = T_r V_1 \frac{V_2}{V_1 - V_2}$$

where

S = length of speed signal funnel, ft;
T_r = red phase length, sec;
V_1 = free-flow speed, fps; and
V_2 = slowest advised speed, fps.

This relationship assumes that drivers will adopt the slower speed at the speed signal location and progress toward the intersection at a constant velocity. In reality, the adjustment to the slower speed may not occur exactly at the point of the speed signal. It is more likely that a driver will react to a slower advised speed at some distance prior to the variable-message speed signal and then continue to decelerate to the slower speed for a considerable distance after he has passed the speed signal location. This gradual deceleration to the slower speed would result in his arriving at the intersection several seconds prior to appearance of the green phase. To compensate for the gradual vehicle deceleration pattern, we increased the speed signal funnel lengths given by Breuning's equation as necessary to prevent the early arrival from occurring.

The number of speed signals installed on an approach is a problem remaining to be explored in further detail. It is recognized that drivers must receive sufficient information to properly regulate their progress, but they must not be overwhelmed by the advisory speed messages to the extent that a confusing siutation is created. For lengthy funnels at high-speed intersections, von Stein (11) has illustrated and recommended the installation of three speed signals on an approach. It was therefore decided that, for this preliminary analysis, selective placement of three speed signals on the north and south approaches at the study site would be appropriate.

Figure 4 shows the profile of Colo-121, the location of the speed signals, and the sequence of speeds to be displayed on the speed signals. The intermediate speed signals are located according to a somewhat irregular spacing to permit better visibility as drivers travel through the system and to provide speed information to drivers entering from minor side streets.

Table 1. Present fully actuated controller timing.

Route	Adjustment	Time (sec)
Colo-121	Minimum green	15
	Maximum green	40
	Extension	15
	Amber	5
	All red	2
West 80th Avenue	Minimum green	14
	Maximum green	30
	Extension	7
	Amber	4
	All red	2

Table 2. Accident experience summary.

Period	Months	Accidents	Persons Injured	Fatalities
1961-1964	41	13	23	1
1967-1971	44	21	9	5

Figure 1. 1971 peak-hour volumes and ADT.

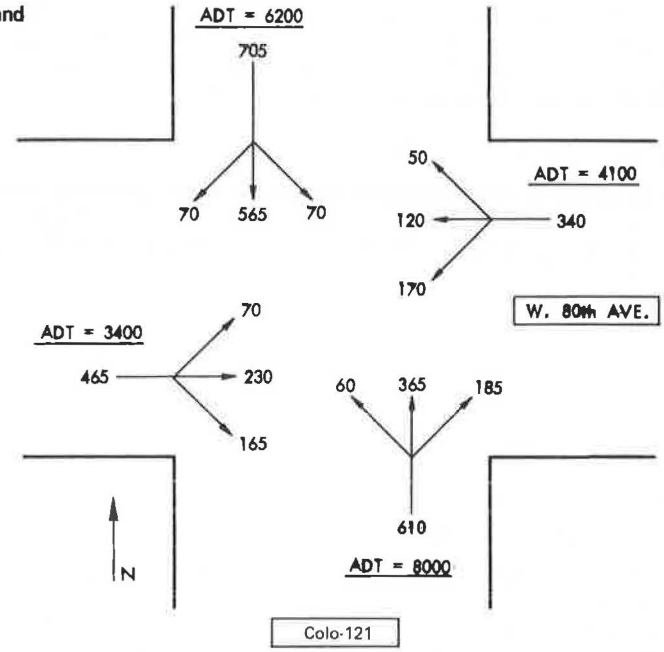
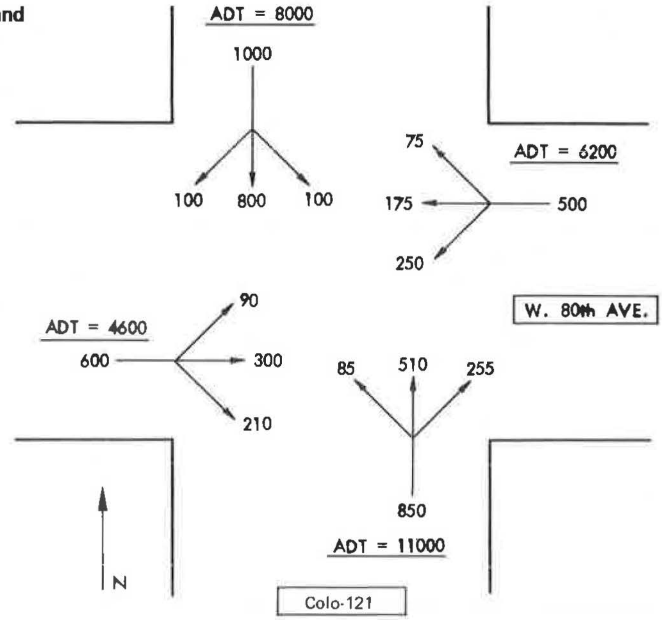


Figure 2. 1972 peak-hour volumes and ADT.



BENEFIT-COST EVALUATION

A detailed economic analysis was performed to determine the feasibility of installing the proposed speed signal funnel as compared to continuing with the existing two-phase, fully actuated control at the intersection. An interest rate of 6 percent was selected, and the following factors were evaluated for both control systems:

1. Highway costs—capital expenditures, maintenance cost, and equipment operation cost; and
2. Road user costs—motor vehicle running cost, motor vehicle idling cost, travel time costs, and accident costs.

Due to the controversial nature of certain cost factors, such as the value of a driver's time and the actual cost of an automobile accident, several benefit-cost ratios were calculated.

Speed Signal Funnel Installation Costs

The initial expenditures (in 1971 costs) required for the multibulb variable-message speed signals, the poles and mast arm mountings, a new pretimed multiphase controller, installation, and roadway widening were as follows:

Item	Cost (dollars)
Pretimed signalization	8,500
Poles and mast arms (six required)	2,400
Speed signals (six required)	10,200
Installation	2,400
Supplementary signs	1,000
Widening W. 80th Avenue	18,000
Total	42,500

The equivalent annual costs corresponding to the initial investments and the necessary roadway widening on West 80th Avenue and maintenance and operation are given in Table 3. Data in Table 3 represent 1971 costs.

Road User Cost Estimates

To formulate an incremental benefit-cost ratio required that road user costs for both the present system and the proposed speed signal funnel be predicted. Field studies of the existing fully actuated signal system served as the basis for estimating future road user costs associated with continued use of the present equipment. Field data were taken by sampling procedures to estimate vehicular delay at the intersection, and car-following studies were conducted to determine vehicular deceleration patterns and travel times. Vehicle running costs and travel time costs of commercial vehicles were estimated for all possible movements against all possible signal indications for a distance equal to 1 mile before the intersection on Colo-121 and ¼ mile before the intersection on West 80th Avenue. A total daily road user expense was determined for each movement with the existing situation by utilizing vehicle operation and time costs as tabulated by Winfrey (12). The annual total costs of vehicular operation and driver time were estimated for retaining the present control system in 1972 by applying a factor of 365 days/year and a ratio of 1972 ADT to 1971 ADT to the 1971 daily costs and summing for all possible traffic movements at the intersection. The 1972 accident costs were estimated according to a potential conflict model developed in Appendix B. The results of these road user cost analyses as well as the annual maintenance cost are as follows:

Figure 3. Proposed signal phasing for intersection controller (70-sec cycle).

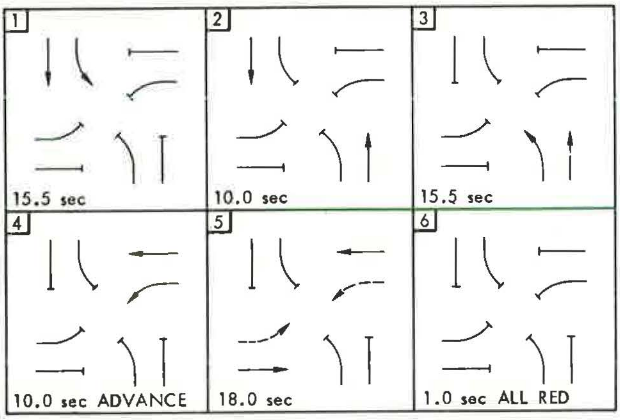


Figure 4. Profile view and speed signal location on Colo-121.

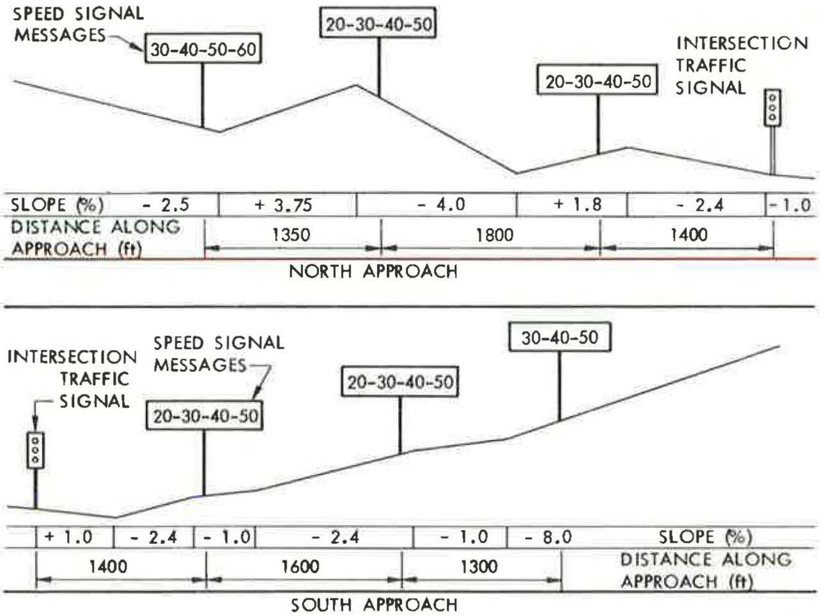


Table 3. Equivalent annual cost of investment and operation at an interest rate of 6 percent.

Item	Service Life (years)	Salvage Value (dollars)	Annual Cost (dollars)
Widening West 80th Avenue	20	0	1,570
Equipment and installation	8	0	3,950
Annual operation and maintenance	–	–	2,350
Total			7,870

Table 4. Incremental benefit-cost evaluation.

Cost Sources Included	Benefit-Cost Ratio
Investment, maintenance, vehicle operation	4.1
Investment, maintenance, vehicle operation, travel time	1.5
Investment, maintenance, vehicle operation, travel time, direct accident cost	2.6
Investment, maintenance, vehicle operation, travel time, direct and indirect accident cost	9.3
Investment, maintenance, vehicle operation, direct and indirect accident cost	12.0

Cost Source	Amount (dollars)
Vehicle operation	1,031,600
Travel time	258,000
Accident direct cost	16,800
Accident direct and indirect cost	108,800
Operation and maintenance	1,000

The road user costs for the proposed speed signal funnel were developed in the same manner as for the present system although field data were not available for an actual installation. Each traffic movement was analyzed, and the costs were tabulated on a daily basis. It was assumed that drivers on Colo-121 would accept the speed advisory messages, thus eliminating stops on the high-speed route. An accident prediction model was developed for the speed signal system and this is described in Appendix B. The estimated annual road user costs for the speed signal funnel are given below:

Cost source	Amount (dollars)
Vehicle operation	1,003,600
Travel time	276,000
Accident direct cost	8,700
Accident direct and indirect cost	55,000
Operation and maintenance	2,350

A comparison of costs for the present system and for the funnel reveals that the funnel could be expected to reduce vehicle operation and accident costs; however, it would cause a slight increase in maintenance and travel time costs.

Incremental Benefit-Cost Evaluation

It is possible to compute several different benefit-cost ratios for any highway improvement project, depending on certain assumptions such as which road user costs to include, the placement of the maintenance costs (numerator versus denominator) in the computation, and the interest rate that is selected. The benefit-cost ratios given in Table 4 were calculated according to the AASHO (10) procedure where an interest rate of 6 percent has been assumed. This table shows that, for all types of cost combinations commonly used in calculating the incremental benefit-cost ratio, the speed signal funnel installation is economically justified when compared to continued operation with the present system. As expected, the largest benefit-cost ratio is obtained when vehicle operation, direct and indirect accident costs, maintenance, and investment costs are included in the analysis. The effect of including travel time costs in any of the computations is to slightly reduce the ratio.

CONCLUSIONS AND DISCUSSION

This investigation has demonstrated the economic feasibility of installing speed signal funnels on the two major approaches at a specific signalized intersection. It was found that the benefit-cost ratio for the proposed change would be no lower than 1.5 to 1.0 and may be as high as 12.0 to 1.0 depending on the factors included in the computations.

In support of the change to the speed signal funnel it seems worthwhile to mention several other environmental factors that tend to favor the installation but that were not rigorously evaluated. Specifically, it would seem reasonable to anticipate a reduction in traffic noise level if the funnel is installed, inasmuch as main route vehicles would not be forced to stop and then completely regain speed at the intersection. It would also seem reasonable to anticipate a reduction in vehicle exhaust emissions inasmuch as the speed signal funnel would facilitate smoother vehicular operation.

This study has, of necessity, relied on estimates of equipment costs, installation costs, maintenance costs, and road user costs for a specific location. A different location may not yield similar benefit-cost ratios due to several possible sources of variation. It is obvious that different traffic volumes could cause noticeable changes in road user benefits. Another factor that could cause considerable variability is the adequacy of the intersection capacity and the necessity for widening the approaches at the intersection. In this study it was thought advisable to widen the two minor route approaches to gain needed intersection capacity and to permit smoother flow during peak hours. These additional construction costs for the roadway widening penalized the speed signal funnel in this evaluation, and this may not be a pertinent cost at other locations. Additional costs were also assessed against the speed signal funnel due to the change to a new multiphase pretimed traffic controller, without the recognition of any salvage value for the currently used equipment. If this change had not been required, substantially higher benefit-cost ratios would have been obtained favoring the speed signal funnel.

ACKNOWLEDGMENTS

This research project was conducted with the financial support of the National Science Foundation and partial support from the Engineering Research Institute at Iowa State University. The cooperation of the Colorado Division of Highways and the Arvada City Traffic Division in performing this study is sincerely appreciated.

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APPENDIX A

ECONOMIC EVALUATION OF TRAFFIC PACER

In 1964 Hulbert (7) reported an economic evaluation of the northbound flow through the General Motors traffic pacer installation. The analysis determined the rate of return on the extra investment for the traffic pacer compared to a past system and for the traffic pacer compared to a progressive system.

The additional initial investment required for the traffic pacer when compared to the past system is $36,600, whereas the additional initial investment for the traffic pacer when compared to the progressive system is $32,000. Additional annual maintenance and operation costs would be $2,000 and $1,200 more than for the past and progressive systems respectively. The reported amounts are 50 percent of the actual expenditures because the road user benefits were estimated for the northbound flow only. Annual road user costs are given in Table 5. Separate analyses were performed for time costs estimated for commercial traffic only and time costs estimated for all main route traffic. Accident costs were not included in the evaluation.

The rate of return on the extra investment was calculated on a 10-year equipment life with negligible salvage values. Excluding extra-market costs, the rate of return on the traffic pacer was 1,000 and 380 percent over the past and progressive systems respectively. Including extra-market costs, the rate of return on the pacer was 1,350 and 360 percent over the other systems. It is apparent that the extra investment in the traffic pacer yields a high rate of return for all reported comparisons.

To perform a benefit-cost evaluation using traffic pacer data required that an interest rate be assumed so that all costs may be expressed in terms of equivalent annual costs. The equivalent annual cost for extra investment in the traffic pacer (interest rate = 6 percent) was $4,970 and $4,350 when compared to the past and progressive systems. Table 6 gives the benefit-cost rates determined according to the procedure recommended by AASHO (10). These computations yield high benefit-cost ratios, indicating that the traffic pacer system is definitely the preferred system of the three evaluated.

APPENDIX B

ACCIDENT PREDICTION MODEL

To predict the number and severity of accidents expected in 1972, it was decided to develop an accident exposure model based on 1969-1970 volume and accident records for the study site. The information included in the modeling process was (a) the number of accidents of each type and severity reported in the base period, 1969-1970; (b) the average annual number of vehicle exposures corresponding to each accident type in the base period; and (c) the expected annual number of vehicle exposures for each type of accident with the two control systems for the year 1972, based on 1972 traffic projections. The expected number of 1972 accidents with each system was estimated by multiplying the base period accidents by the ratio of 1972 vehicle exposures to the corresponding base period vehicle exposures.

In estimating the number of accident exposures, we evaluated each traffic movement on each approach to determine the average number of exposures per signal cycle. This was converted to an annual number of exposures by estimating the number of signal cycles to occur within a 1-year period. The results of the accident exposure modeling process are given in Table 7 by accident type for the intersection control systems being compared. The data given in Table 7 indicate a general reduction in accident exposure with the speed signal funnel, with the exception of the sideswipe collision. The increase in potential for sideswipe collisions reflects the assumption that, with the speed signal funnel, traffic will flow in more tightly grouped platoons, and there may be a greater opportunity for a sideswipe or lane-change accident with vehicles in platoons than with random flow. The projected 1972 accident experience by type and severity of accident is given in Table 8. Miscellaneous accidents were estimated by applying a factor of 20 percent to the subtotal, inasmuch as miscellaneous accidents were 20 percent of the subtotal in the 1969-1970 base period.

Two separate evaluations of the cost of the projected 1972 accidents were then performed. The first evaluation utilized only the direct costs for each type and severity of accident according to the recent findings in Texas (6). The results of the direct cost

Table 5. Annual road user costs.

System	Market Cost[a] (dollars)	Market Plus Extra-Market Cost[b] (dollars)
Past	1,267,500	2,051,500
Progressive	992,200	1,665,000
Pacer	869,500	1,548,500

[a]Includes vehicle operation costs for all vehicles and time costs for commercial vehicles.
[b]Includes vehicle operation costs and time costs for all vehicles.

Table 6. Traffic pacer incremental benefit-cost evaluation.

	Benefit-Cost Ratio	
Systems Compared	Excluding Extra-Market Costs	Including Extra-Market Costs
Pacer and past	57.1	72.3
Pacer and progressive	22.1	21.0

Table 7. Accident exposures.

Accident Type	Average Exposures per Year During 1969 and 1970	Projected 1972 Exposures: Present System	Projected 1972 Exposures: Signal Funnel
Rear end	1,770,000	2,910,000	960,000
Right angle	890,000	1,180,000	840,000
Sideswipe	430,000	840,000	2,270,000
Turning	2,610,000	5,030,000	2,340,000
Total	5,700,000	9,960,000	6,410,000

Table 8. 1972 accident experience prediction.

	Projected 1972 Accidents	
Accident Type and Severity	Present System	Signal Funnel
Rear end, PDO	1.5	0.5
Right angle, PDO	0.5	0.5
Sideswipe, PDO	1.0	2.0
Sideswipe, INJ		0.5
Turning, PDO	4.5	2.0
Turning, INJ	1.5	0.5
Turning, FAT	1.0	0.5
Subtotal	10.0	6.5
Misc., PDO	1.5	1.5
Misc., INJ	0.5	—
Total	12.0	8.0

Note: PDO = property damage only, INJ = injury, and FAT = fatal accident.

Table 9. Estimated 1972 accident costs.

	Direct Costs (dollars)		Direct and Indirect Costs (dollars)	
Accident Type and Severity	Present System	Signal Funnel	Present System	Signal Funnel
Rear end, PDO	450	150	450	150
Right angle, PDO	200	200	200	200
Sideswipe, PDO	250	500	250	500
Sideswipe, INJ		650		4,000
Turning, PDO	1,350	600	1,350	600
Turning, INJ	2,850	950	12,000	4,000
Turning, FAT	10,200	5,100	90,000	45,000
Misc., PDO	550	550	550	550
Misc., INJ	950	—	4,000	—
Total	16,800	8,700	108,800	55,000

evaluation are given in Table 9. Both direct and indirect costs were considered in the second evaluation, where values of $8,000 for each injury and $90,000 for each fatality were applied as drawn from the recent U. S. Department of Transportation Automobile Insurance and Compensation Study (5). The data reported in Table 9 were included in several of the incremental benefit-cost analyses of this investigation.

MICROSCOPIC ANALYSIS OF TRAFFIC FLOW PATTERNS FOR MINIMIZING DELAY ON SIGNAL-CONTROLLED LINKS

Nathan Gartner*, Massachusetts Institute of Technology

An optimal scheme for coordination of consecutive signals along arterial routes or networks requires a microscopic analysis of the traffic flow patterns on every link of the system. Such an analysis was carried out for two-way links on a major artery in downtown Toronto. Accurate platoon profiles were obtained via the digital computer system controlling traffic lights throughout the metropolitan area and its associated vehicle-detector system. Individual link delay functions were calculated subject to the particular characteristics of each signalized traffic link. These functions were then combined in parallel according to the principles of the British TRRL combination method. The optimal settings derived are shown to deviate substantially from those established by conventional coordination methods. The resultant improvement in delay to traffic was confirmed by direct field observations.

A PAIR of adjacent signalized intersections along an urban arterial street is connected by two-way traffic links. The signals are synchronized, i.e., they operate with a common cycle. The common cycle time is determined by the requirements of the most heavily loaded intersection along the arterial. The green-time splits at each intersection are determined by local demand so that equal degrees of saturation on the conflicting approaches are achieved. The pattern of traffic flow on each of the links is assumed to remain steady during the time period considered. No network constraints are imposed on the phasing of the signals under consideration. The problem is to determine the relative phase (offset) between the linked pair of signals that will cause minimal delay to traffic on the connecting links.

This is a subproblem of the more general problem concerning optimal coordination of linear signal systems. The traditional tool of the traffic engineer in coordinating signals along urban arterials has been the progressive signal system (1, 2, 3). Its objective is to maintain maximal throughbands proportionate to the traffic volumes along both directions of the artery. This objective is intuitively associated with the reduction of stops and delays to traffic. However, it has long been recognized that this design maximizes essentially a geometric quantity (the bandwidth) without explicitly taking into account the actual traffic flow patterns using the system (4, 5).

An advance on this method has been the combination method (6, 7), originally conceived by the British Transport and Road Research Laboratory (TRRL). The salient feature of this technique is that delay incurred by traffic is taken under direct consideration and is systematically minimized. The greatest difficulty in applying the technique is to determine the basic delay-offset relationship for each link. A simplified model for calculation or a crude simulation procedure is usually employed (8).

It is the purpose of this paper to describe a study in which a microscopic analysis of the actual traffic flow patterns on the signal-controlled links is undertaken. The analysis is carried out by direct measurement of traffic flows via vehicle loop detectors

*The research reported was performed while the author was a research fellow at the University of Toronto.

Publication of this paper sponsored by Committee on Traffic Control Devices.

relating their data to the computer control center of Metropolitan Toronto. The data are then processed by the computer to obtain accurate delay-offset relationships for the links under consideration, based on the characteristics of these links. Optimal settings for the controlling signals are derived subsequently.

THEORETICAL BACKGROUND

Link Delay Functions

A traffic link is defined as a section of street carrying traffic in one direction between two signalized intersections, as shown in Figure 1. Delay is incurred at the downstream signal of the link, i.e., where traffic exits the link. The offset across any link is defined as the time difference between the starting point of a green phase at the upstream signal of the link and the starting point of the next green phase at the downstream signal. It is a directional quantity, assuming the direction of traffic flow along that link. This section describes the transition process of traffic through the link's exit signal and the computational procedure for obtaining a delay-offset relationship, given the cyclic flow pattern on the link.

For the purpose of the present discussion, a zero value is assigned to the beginning of the green time at the exit signal of the link in order to establish it as a reference point. Thus, the time interval (-r, g) consists of an effective red period (-r, 0) and an effective green period (0, g) so that

$$r + g = C \tag{1}$$

where C denotes the cycle time. The following notations are also used:

$q_a(t)$ = arrival rate (vehicles/second),
$q_d(t)$ = departure rate (vehicles/second),
$A(t)$ = cumulative number of arrivals,
$D(t)$ = cumulative number of departures,
$s(t)$ = possible departure rate (should there be a nonexhaustive lineup of cars at the signal's stop line), and
s_o = saturation flow rate during green period.

Starting with the beginning of any red period at the exit signal, we have the following basic relations:

$$A(t) = \int_{-r}^{t} q_a(\tau)d\tau \tag{2}$$

$$D(t) = \int_{-r}^{t} q_d(\tau)d\tau \tag{3}$$

$$s(t) = \begin{cases} 0 & \text{if } -r < t \le 0 \\ s_o & \text{if } \ 0 < t \le g \end{cases} \tag{4}$$

The following assumptions are made:

1. Arrivals are periodic, i.e., for any integer number n,

$$q_a(t) = q_a(t - nC) \tag{5}$$

2. The signal is undersaturated, i.e.,

$$A_p < g\ s_o \tag{6}$$

where the total number of cars arriving during one cycle (the platoon size) is

$$A_p = \int_{-r}^{g} q_a(t)dt \tag{7}$$

3. The arrival rate during the green time of the signal does not exceed the saturation flow rate,

$$q_a(t) < s_o \qquad \text{if } 0 < t \le g \tag{8}$$

This implies that, once a queue has vanished during the green period, it cannot rebuild before the next red period commences.

According to these assumptions, all vehicles arriving during a cycle in which the red period precedes the green can be accommodated in that cycle. It follows that the queue is always empty at the end of the green period, and delay time calculations can be confined to a single interval (-r, g).

The queue length Q(t) at any time $-r < t \le g$ is given by the difference between the cumulative number of arrivals and the cumulative number of departures.

$$Q(t) = A(t) - D(t) = \begin{cases} A(t) & \text{if } -r < t \le 0 \\ A(t) - ts_o & \text{if } 0 < t \le t_o \\ 0 & \text{if } t_o < t \le g \end{cases} \tag{9}$$

t_o denotes the time when the queue disappears $(0 < t_o < g)$. By definition, $t = t_o$ when

$$Q(t_o) = A(t_o) - t_o s_o = 0 \tag{10}$$

If we follow this analysis, the departure rate is described by

$$q_d(t) = \begin{cases} 0 & \text{if } -r < t \le 0 \\ s_o & \text{if } 0 < t \le t_o \\ q_a(t) & \text{if } t_o < t \le g \end{cases} \tag{11}$$

An illustration of the traffic transition process through the link's exit signal is shown in Figure 2.

The delay incurred by Q(t) queuing vehicles during an interval dt is Q(t)dt. Therefore, the total delay time d incurred by traffic during a complete cycle (-r, g) is represented by the area under the queue-length curve.

$$d(\theta) = \int_{-r}^{g} Q(t)dt = \int_{-r}^{t_o} Q(t)dt \tag{12}$$

Obviously, the size of this area depends on the relative phase of the exit signal, i.e., on the offset θ. The average delay per car (per cycle) $\delta(\theta)$ is obtained by dividing by the total number of arrivals during one cycle.

$$\delta(\theta) = \frac{1}{A_p} d(\theta) \tag{13}$$

The procedure described yields only one point on the delay-offset curve. To obtain the complete relationship requires that this procedure be repeated while the relative phasing between the exit signal settings and the arrivals is altered so that all possible offsets across the link under consideration are examined.

Parallel Combination Procedure

According to the principles of the combination method, where two or more links occur in parallel, joining two nodes, the delay functions of the individual links can be combined, with reference to the same offset, to yield a total delay function. Referring to Figure 1, $d_1(\theta_{ij})$ and $d_2(\theta_{ji})$ are calculated for $0 \le \theta_{ij} \le C$ and $0 \le \theta_{ji} \le C$ respectively. The two offset variables in this case are constrained by the following relationship:

$$\theta_{ij} + \theta_{ji} = C \tag{14}$$

Consequently, only one of the two offsets can be determined independently. Relating the total delay D to offset θ_{ij}, we obtain

$$D(\theta_{ij}) = d_1(\theta_{ij}) + d_2(C - \theta_{ij}) \tag{15}$$

To obtain the average combined delay function $\Delta(\theta_{ij})$ from the individual average delay functions, we use the following formula:

$$\Delta(\theta_{ij}) = \frac{1}{A_{p1} + A_{p2}} [A_{p1}\,\delta_1(\theta_{ij}) + A_{p2}\,\delta_2(C - \theta_{ij})] \tag{16}$$

An optimal offset θ^*_{ij}, between the adjacent pair of signals, is readily obtainable by searching for the minimal value of the combined function.

SYSTEM CHARACTERISTICS

Links

The site where the study was carried out is located on Bloor Street, one of the major east-west arterials in downtown Toronto. The two links selected for analysis connect the junctions of Bloor-Yonge and Bloor-Church. The links consist of two traffic lanes in each direction. Geometrical details are shown in Figure 3. Parking and stopping are prohibited during rush-hour periods, whereas, otherwise, stopping alone is permitted. All turning movements are prohibited at the Bloor-Yonge intersection. Only turning-in movements from Church Street onto Bloor Street are allowed at the Bloor-Church intersection. Thus, relatively compact platoons are formed, characterized by a high degree of coherence.

To calculate the expected delay incurred by traffic at the signal stop lines requires that an estimate of the saturation flows on these links be made. Inasmuch as traffic is confined to well-marked lanes, the Australian method for capacity calculations was adopted (9, 10). The results in through car units (tcu's) are given in Table 1.

Detectors

In the Toronto signal system, vehicle detectors are installed on all approaches to signals that have been designated to operate in the TR2 responsive control mode (11). Under this scheme green-time splits are adapted dynamically to traffic demand, while a constant cycle length and a fixed offset relationship in the major flow direction are maintained. Both signals at the study site shown in Figure 3 operate in the TR2 control mode and thus are equipped with detectors. Each link has one 4-ft wide magnetic loop detector across both traffic lanes, located approximately 300 ft in advance of the downstream intersection. Information on vehicular presence within each detection zone is transmitted via telephone lines to the traffic control center by means of tone-telemetering devices. At the center, the state of each detector circuit is scanned at a rate of 16 times a second. The data are then fed into a Univac 418 computer to be reduced and relayed to a Univac 1107 computer for further processing.

DATA COLLECTION AND ANALYSIS

Platoon Profiles

The traffic data reported by the detectors to the central computer are used for calculations in the on-line responsive green-time allocation algorithm. They are also

Figure 1. Two-way traffic links connecting pair of adjacent signalized intersections.

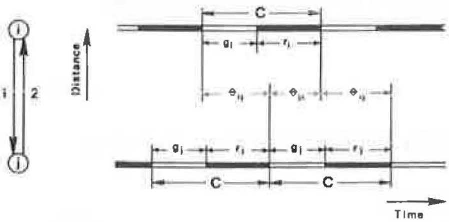


Figure 2. Traffic transition process through a link's exit signal. (U and V in arriving and departing flow patterns are equal in magnitude. W represents the estimated delay per cycle.)

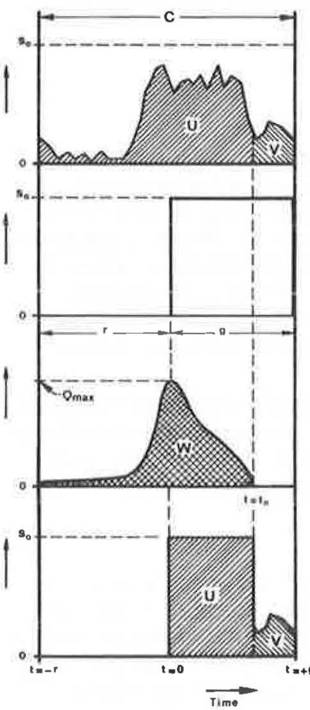


Figure 3. Geometrics of study site: link 1—eastbound; link 2—westbound.

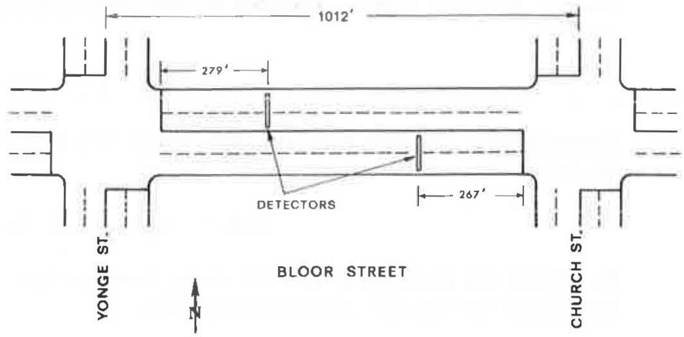
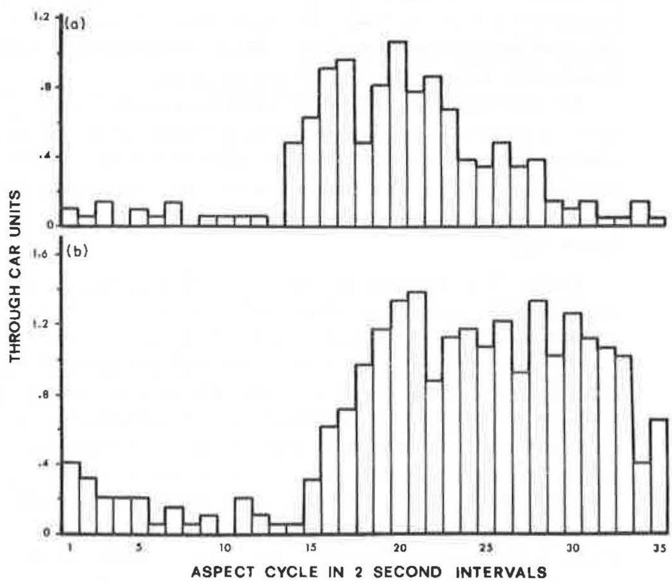


Table 1. Saturation flows by link and lane.

Link	Curb Lane (tcu/hour)	Center Lane (tcu/hour)	Total for Approach (tcu/hour)	Total for Approach (tcu per 2-sec interval)
1 (eastbound)	1,270	1,580	2,850	1.56
2 (westbound)	1,350	1,650	3,000	1.67

Figure 4. Platoon profiles and delay functions for 70-sec cycle length: (a) link 1 and (b) link 2.



recorded on magnetic tape in conjunction with the prevailing signal aspects for off-line analysis and evaluation. Thus, if the signal upstream from a detector is being controlled with a fixed cycle length, the platoon profile at the detector can be calculated from the data recorded by the computer. The cycle is divided into short subintervals of time (usually 2 sec). The number of vehicles crossing the detector in each interval is averaged over a number of cycles during which the average flow intensity is assumed to remain steady (e.g., 25 cycles). The resulting curve shows how the vehicle density varies throughout the average cycle. Congested cycles, i.e., cycles during which the approach queue on the downstream end of the link extends to the detector or beyond, must be excluded; otherwise, the actual shape of the platoon profile will be distorted.

The method used to detect congestion is based on an exponential smoothing process. Average values are continuously generated by assigning relative weights to new versus old information according to the following generalized formula: (present value of average) = A (new data) + (1 - A) (previous value of average). The smoothing factor A in this formula has to be a positive number less than 1. Exponential smoothing is used to calculate both the average volume level EDGEX and the average pulse length PULSEX. These values are calculated for each approach as follows:

$$\mathrm{EDGEX}_m = (\alpha)\ \mathrm{count}_m + (1 - \alpha)\ \mathrm{EDGEX}_{m-1}$$

$$\mathrm{PULSEX}_n = (\beta)\ \mathrm{pulse}_n + (1 - \beta)\ \mathrm{PULSEX}_{n-1}$$

α and β are smoothing factors that have been assigned the values 0.01 and 0.2 respectively. Count_m denotes the number of counts recorded by the detector during the time interval m. Pulse_n denotes the n th pulse length, i.e., the time duration for which the induction loop senses vehicle presence; it is inversely related to speed. The PULSEX calculation is performed only when a new pulse occurs, whereas the EDGEX calculation is repeated for each time interval. Congestion is assumed to exist when both of the smoothed quantities pass specified threshold values. The values are chosen so that only the existence of a real queue is detected, while disturbances such as single vehicles stopping or moving slowly over the detector are ignored. The values used by the traffic program are 0.07 for EDGEX and 0.7 for PULSEX (12).

The analysis for each link is terminated when the required number of cycles has accumulated or if a program change has occurred and the cycle time of the upstream signal was altered. In this case the flow pattern over the detector changes, and a new profile has to be generated.

Shift and Calibration Procedures

The platoon profile obtained through averaging of the magnetic tape vehicle count recordings, which was described in the previous section, has to undergo two transformations, one in time and the other in magnitude.

Time Transformation—The profile describes the arrival pattern observed at the detector location during an average cycle starting with the upstream signal's green phase. If the link delay function is to be calculated, this pattern has to be extrapolated (in time) to the stop line at the downstream signal. With well-defined platoons, such as those shown in Figures 4 and 5, travel times from the previous intersection to the detector can be estimated. The point at which the platoon profile rises steeply represents the average arrival time of the leading edge of the platoon. Given the length of the link and if allowances are made for the fact that the leading edge of the platoon usually accelerates from a standing position, a good estimate of speed can be made. The average speed that was detected on the links under consideration varied in the range of 25 to 28 mph. The arrival pattern is then shifted backward by the expected travel time from the detector to the downstream stop line.

Magnitude Transformation—The two factors involved are the necessity for count calibration due to errors in detector countings and the conversion of vehicle counts into equivalent tcu's. Extensive field observations have shown that, due to a multitude of error possibilities, the detectors tend to overcount for volumes below 600 vph per two-lane approach, whereas they undercount for volumes above this level (13). A calibration curve obtained by regression analysis is available to take account of these errors. Because the saturation flows are specified in tcu/hour, the arrival flow has to be stated in the same units. If the average composition of traffic is known as well as the proportions of turning vehicles, the counts can be scaled up to equivalent tcu's by using appropriate conversion factors.

Random Variations

Once the platoon profile is obtained, the delay-offset relationship can be readily established by repeated use of Eqs. 1 to 16. However, this profile represents the expected pattern of arrivals at the signal's stop line. It is an average component of a periodic process that includes also a random component arising from variations in driving speeds, marginal friction, and turns. The latter component may cause additional delay because of the possible occurrence of an overflow queue Q_o at the end of the green period. Although this effect is negligible at low degrees of saturation, its importance increases at values exceeding 0.80 (14). According to our notation (see Eq. 6), the degree of saturation x is defined as

$$x = A_p/gs_o \tag{17}$$

Account of this factor is taken by estimating the expected overflow queue according to the value of the saturation flow at the signal's approach and the expected degree of saturation. Such an estimate was given by Wormleighton (15), who considered traffic behavior along the link as a nonhomogeneous Poisson process with a periodic intensity function represented by the platoon profile obtained from the detector countings.

Results

Data were collected during the morning rush hours while an inbound plan (70-sec cycle) and a heavy inbound plan (80-sec cycle) were in operation. The resultant platoon profiles for each of the links involved are shown in Figures 4 and 5 respectively. The ratio of flows among the links is approximately 2:1 in favor of the inbound direction (westbound direction in this case). The conventional procedure in such circumstances is to provide the best possible progression to accommodate the heavy flow. In some cases this includes provision of advance clearance times for queues that might develop during the signal's red period owing to overflows, turning movements, or intralink sources (such as parking lots and garages) and obstruct the major flow emanating from the previous signal. Given the length of the link l and the desired speed of travel along the link v, the desired offset between the signals in that direction, θ_d, would have to be

$$\theta_d = \frac{l}{v} - \frac{Q_g}{s_o} \tag{18}$$

Q_g denotes the expected queue length at the start of downstream green and combines the overflow queue Q_o contributed by the random fluctuations and the expected number of arrivals during the red period: $\int_{-r}^{0} q_a \, dt$. The value of Q_g is determined either by estimation or by direct observation.

According to this design, the starting of downstream green is advanced Q_g/s_o sec prior to the regular progression time along the link of l/v sec, so that the platoon released from the upstream signal will be able to pass unhindered. In fact, the advance time might have to be larger to account for possible additional arrivals joining the queue while it is being dissolved. The basic philosophy underlying this design is one of smooth flow control. However, no direct quantitative analysis in terms of delays or other costs is attempted to support this supposition. Two deficiencies are apparent:

1. The coordination policy is concerned only with the leading edge of the arriving platoon, while the remainder of it is neglected; and

2. Traffic on the opposing link (on a two-way arterial), which carries a much lighter load, is often disregarded.

The effects of these deficiencies are illustrated in the following analysis.

Based on the observed platoon profiles, a delay-offset relationship is calculated for each link. Each point on the curve represents the average delay per vehicle the platoon would incur should the downstream green start at the corresponding time of the arriving pattern. Because the origin coincides with the starting of upstream green, the horizontal axis describes the relative phase between the signals (the offset). The individual link delay functions are depicted in Figure 6 for the 70-sec cycle plan and in Figure 7 for the 80-sec cycle plan. For each of these pairs a combined relation is established from Eq. 16 and is shown illustrated in Figures 6c and 7c respectively. A range of offsets for which the expected combined delay is within 5 percent of the minimum has been designated as the "minimum range." Point A indicates the offset advocated by the progression method according to Eq. 18. Evidently, a considerable reduction in delay time, and hence in travel time, can be achieved by selecting one of the offsets within the minimum range: up to 40 percent for the 70-sec cycle plan and 45 percent for the 80-sec cycle plan.

Field Observations

To check the validity of the link delay functions, we conducted several field observations at the test site. The performance of the traffic signal control system was compared for different offset settings while an "inbound" plan (70-sec cycle) was in operation. The comparison was done by manual measurement of stopped-time delays. A detailed description of the observations is given elsewhere (16). The compiled results of measurements for offsets corresponding to points A and B in Figure 7c are given in Table 2.

Westbound Link (link 2)—This link carries the major flow during the morning rush hours. While traveling along the link, the platoon formed by the upstream signal disperses in time. As it arrives at the downstream signal, it has spread wider than the available green service period, and some of the cars must be delayed. With an offset corresponding to point A, the leading edge of the platoon arrives just after the queuing cars have cleared the approach (or at the beginning of green should there be no waiting cars) and can pass unimpeded. However, a substantial portion of the platoon is cut off when the green time terminates and has to wait for the next phase to be served. As was shown in a theoretical study by Newell (17), under such conditions delay in one-directional traffic is minimized if the trailing edge of the platoon arrives at the light just before it turns red and suffers no delay, whereas the leading edge arrives too early and is stopped; it is released at saturation flow when the next green commences. Obviously, an offset setting corresponding to point B comes close to this policy.

Eastbound Link (link 1)—This link carries the minor flow during the morning rush hours. The signals on this link operate at a much lower degree of saturation (approximately 0.4 as opposed to 0.8 on the westbound link). The width of the arriving platoon does not usually exceed the available green time. With an offset located at point A, the platoon arrives at the downstream signal shortly after the beginning of red and consequently suffers a long waiting time and a high percentage of stoppages. When the offset is shifted to point B, the beginning of downstream green is advanced (the offset for this link is shortened). The platoon still arrives during the red aspect but at a later instant and therefore suffers a shorter waiting time before being served. The proportion of stopped cars is also reduced slightly. Minimal delay for travel on that link cannot be achieved because of the constraint imposed on its phasing by the requirements of the parallel link, carrying traffic in the opposing direction between the same pair of signals. However, the minimum range of offsets in the combined delay function is largely due to the possibility of trade-offs in the allocation of delays among the competing links.

Figure 5. Platoon profiles and delay functions for 80-sec cycle length: (a) link 1 and (b) link 2.

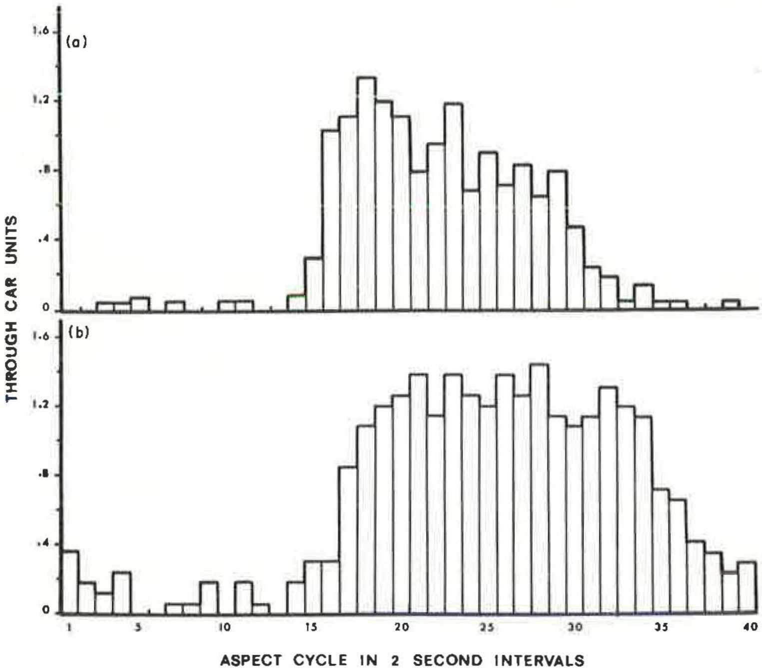


Figure 6. Delay per vehicle on (a) link 1, (b) link 2, and (c) links 1 and 2.

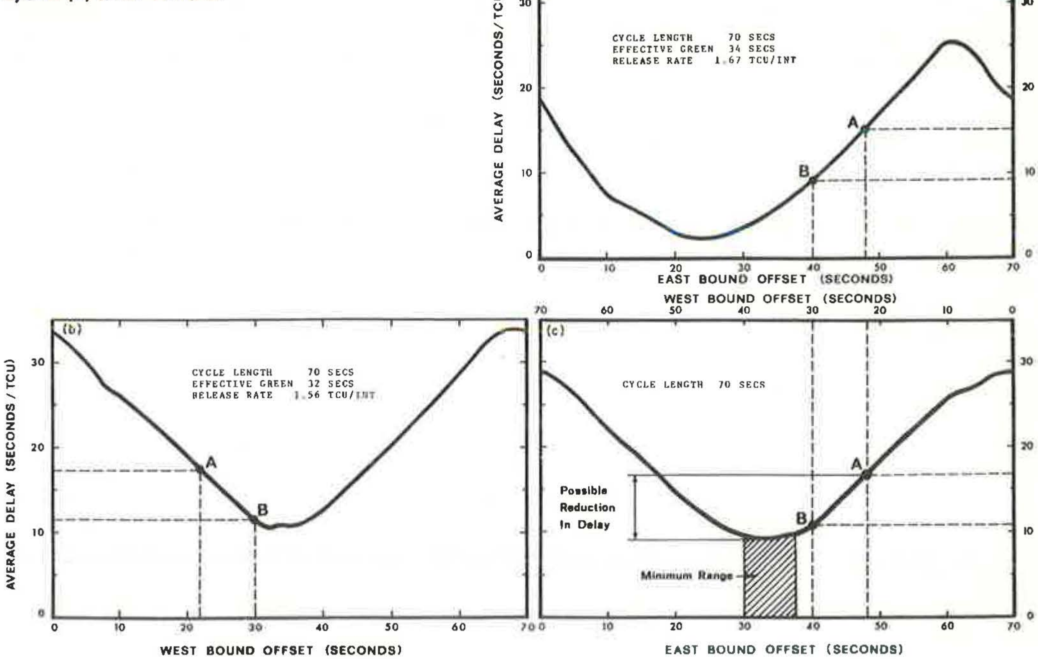


Figure 7. Delay per vehicle for (a) link 1, (b) link 2, and (c) links 1 and 2.

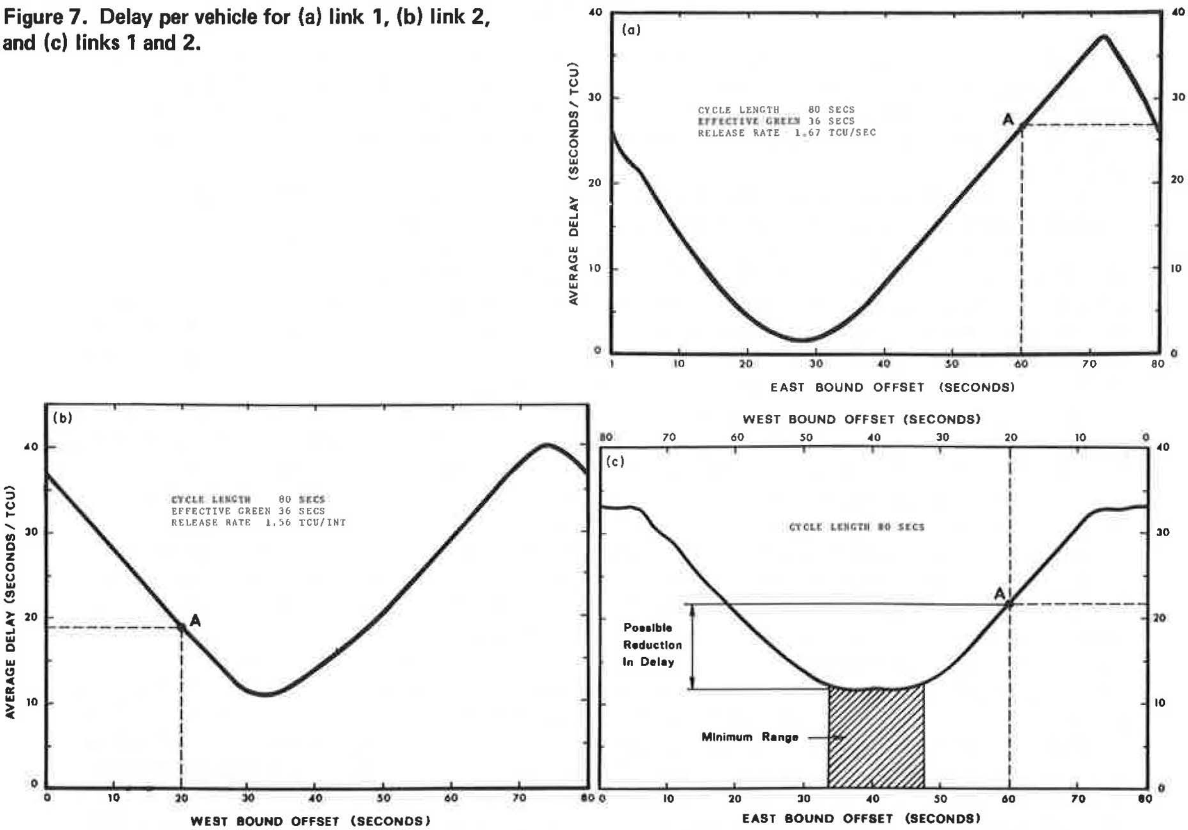


Table 2. Comparison of coordination policies by field observations of stopped-time delays.

	Point A			Point B			Percentage of Change		
Item	Link 1	Link 2	Links 1 and 2	Link 1	Link 2	Links 1 and 2	Link 1	Link 2	Links 1 and 2
Average delay per vehicle (sec)	12.6	16.1	14.9	6.8	11.4	9.8	-46	-29.1	-34.2
Percentage of vehicles stopped	85.0	53.8	63.9	74.5	44.6	54.6	-12.5	-17.0	-14.6
Volume (vph)	542	1,100	1,624	554	1,070	1,624	+2.2	-2.7	

CONCLUSIONS

A microscopic analysis of traffic flow patterns was conducted using the facilities of the Toronto computer control system. It was shown that optimal signal settings determined by such an analysis can produce considerable savings in delays to traffic. As a result, travel times through the system are reduced, and the capacity of the system is increased.

The method of analysis in this study is applied to fixed-time signal settings. It is assumed implicitly that flow patterns are constant over a certain control period, such as morning or evening rush hours, off-peak periods, and the like. It is also assumed that these patterns are of a repetitive nature during similar days of the week. Experience with the Toronto system over a number of years, where extensive data collection and analysis have been carried out, has shown that these assumptions are valid for many links of the system. Hence, this procedure can be a useful tool in providing optimal coordination schemes both for arterial streets and for networks.

The outlined method is not limited to fixed-time programs. Conceptually, the same procedure can be applied to develop a responsive control logic by which optimal settings are determined for control periods on the order of a few minutes. Apart from the hardware requirements, the main problem is whether programs should be selected automatically according to demand or calculated on-line.

An interesting subject for further investigation would be to assess the importance of other criteria in traffic signal coordination. In some schemes, a stop penalty is superimposed on the delay cost (18). As the limited field tests conducted in this study have shown, reduction in delays was accompanied by a reduction in the number of stops. But, generally, the minima of both measures do not necessarily coincide, though they are close (19). Emphasis is placed, in these cases, on the deterministic act of a complete stop, i.e., the deceleration of a vehicle to zero velocity. The process of vehicle platoons traversing a series of traffic lights is one in which repeated decelerations and accelerations take place. Some of the vehicles have to decelerate to a complete stop. A rational objective function should take into account the disutilities associated with all vehicular maneuvers. These disutilities might eventually include the costs of time losses, wear, discomfort, accident risk, pollution, etc. The delay function proposed in the present analysis is believed to constitute a good approximation to such a generalized cost function.

ACKNOWLEDGMENT

The author gratefully acknowledges the assistance given to him by E. Hauer and R. M. Soberman of the University of Toronto in conducting this research. The author would also like to thank S. Cass, Metropolitan Toronto Commissioner of Roads and Traffic, for making it possible to use the facilities of the Toronto Traffic Control Centre and for his permission to publish the results.

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A VARIABLE-SEQUENCE MULTIPHASE PROGRESSION OPTIMIZATION PROGRAM

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A traffic signal progression program has been developed that maximizes progression along a facility having multiphase signals. The main street green phase sequences of left turns first, through movements first, leading green, and lagging green can be evaluated at each intersection. The progression program can determine which of the four phase sequences provides maximum progression. Conventional two-phase signal operation is a special case of the through-movements-first sequence. The computer program, written in FORTRAN IV, can also compute movement durations and phase splits if desired. The progression program was adapted for use in the real-time control of an arterial pilot control system in Dallas. The controllers were modified to permit variable-phase sequence operation. Good progression was obtained, and no apparent problems have occurred due to the variable-phase sequencing.

• WITH ever-increasing demands being placed on urban traffic facilities, traffic engineers need efficient traffic control systems and strategies to improve the level of service being provided. New solid-state traffic controllers, digital process control computers, and minicomputers now provide increased computational and control capabilities for use in improving traffic operations. Modern telecommunications equipment enables efficient gathering, transmission, and receiving of large quantities of traffic data. Integrated circuit design now also permits flexible signal phase implementation. These new computational and control capabilities have removed several of the hardware constraints that have restrained the implementation of more responsive and efficient traffic control strategies.

Traffic control strategies have been developed for optimizing large-network signal control such as SIGOP (1, 2) and TRANSYT (3). The analysis is done off-line on large digital computers. However, the intersection control operation is usually conventional two-phase operation. In the area of arterial optimization, researchers have concentrated on developing optimization techniques that minimize delay, such as the delay-offset technique (4), or that maximize the progression bands, such as the algorithms developed by Little (5) or Brooks (6) as investigated by Bleyl (7). A recent research study (4) recommends that Webster's method (8) for computing cycle length and splits be used in conjunction with the arterial optimization techniques.

The previously noted arterial progression programs determine the offsets that yield the maximum progression only for conventional two-phase signal operation. These programs do not analyze multiphase (greater than two-phase) signal operation or a control process having variable multiphase sequencing. With modern electronics, variations in phase sequencing are possible. For example, a lagging green phase sequence can easily follow a leading green phasing arrangement using the new hardware.

ARTERIAL PILOT STUDY CONTROL SYSTEM

For purposes of illustration, the program is discussed as applied to a pilot arterial control system operated in Dallas as a research project conducted by the Texas Trans-

Publication of this paper sponsored by Committee on Traffic Control Devices.

portation Institute for the Federal Highway Administration in cooperation with the Texas Highway Department and the city of Dallas. The general-use progression program was used with a real-time data acquisition and signal driver program to produce the real-time control program.

The arterial pilot study site is located on Mockingbird Lane, a six-lane divided major urban arterial that serves as a crosstown facility and a feeder street to the North Central Expressway. Within the 1½-mile study section (Fig. 1), there are three high types of intersections, having separate protected left-turning movements, and a diamond interchange at the expressway. These intersections have traffic-actuated controllers; however, with the installation of additional electronics, the actuated controllers were completely bypassed during computer control, thus permitting a variable selection of nonconflicting phase sequences. The diamond interchange is operated with four-phase overlap control (9, 10). Progression is provided in both directions along the arterial from the interchange through the three intersections.

DEVELOPMENT OF PHASING SEQUENCES AND PROGRESSIVE GREENS

Actuated Control Phase Sequence

Each high type of intersection has eight separate and protected movements as shown in Figure 2. When the actuated controllers are operating, traffic movements are separated into two basic phases: the A-phase for the arterial and the B-phase for the cross street. The relationships between traffic movements and resulting phase sequence are shown in Figure 3. Within the A-phase, the normal quad-left operation with all movements calling would be left turns first (movements 1 + 3), followed by a left-turn drop-out (movements 2 + 3 or 1 + 4) depending on the durations of movements 1 and 3, followed by the through movements (2 + 4). Transfer of control to the B-phase would then be made, which would result in a similar sequence.

Analysis of Four Phase Sequences

The program determines the signal phase sequence and offset at each intersection that will maximize the progression. Within the basic two-phase framework consisting of an A-phase followed by a B-phase, the following four A-phase sequences (Fig. 4) can be analyzed: (a) left turns first (e.g., dual or quad left), (b) through movements first, (c) leading main street (arterial) green, or (d) lagging main street green. The latter two sequences are with respect to the outbound direction from the diamond interchange. A single protected left-turn movement would be either a leading sequence (sequence 3) or a lagging sequence (sequence 4). Conventional two-phase signal operation would be represented by through movements first (sequence 2) with no left turns present.

The three intersections in the pilot study are permitted to have any one of the four possible A-phase sequences with a different sequence permitted with each new real-time evaluation. The diamond interchange in the progression analysis is considered as having only one possible phase sequence, a leading green on the side of the interchange connecting to the remainder of the study section.

Traffic Movement Durations

The movement green times, consisting of the green plus amber, for a given cycle length are based on the demand-capacity ratio concept as presented by Webster (8). The smallest movement green g_{im} (on movement m of intersection i) that will satisfy the present average movement demand D_{im} is computed from

$$g_{im} = \frac{D_{im}}{S_{im}} C + L_{im} \tag{1}$$

where g_{im} must be greater than or equal to a minimum permitted movement length and where S_{im} is the movement saturation or capacity flow, L_{im} is the lost time per movement (11), and C is the cycle length. The cycle length used by the progression program is the one that results in the most efficient progression as described later.

The minimum A-phase (arterial) and B-phase (cross street) lengths are then computed from

$$A_{min} = \max \begin{vmatrix} (g_{11} + g_{12}) \\ (g_{13} + g_{14}) \\ Ped_B \end{vmatrix}$$

$$B_{min} = \max \begin{vmatrix} (g_{15} + g_{16}) \\ (g_{17} + g_{18}) \\ Ped_A \end{vmatrix} \qquad (2)$$

where Ped_A and Ped_B are the minimum pedestrian crossing times when activated. The respective movements are as shown in Figure 2. Any slack, or difference between the sum of the minimum A and B phase lengths and the cycle length being analyzed, is first prorated to the two basic A- and B-phases. The corresponding phase slack is then proportioned to the related movements within the phase.

Queue Clearance Option

The objective of an arterial progressive signal system is to allow platoons of vehicles to travel through the signal system without having to stop. These vehicles are impeded when they arrive during a red signal or when they arrive during a green but are blocked by a queue of vehicles still stopped at the signal. Even for an arterial having good two-way progression, queues can form at intersections due to traffic turning onto the arterial from adjacent intersections or to parking facilities.

If the average stopped queue for each of the through movements (g_{12}, g_{14}) is known, then the queue clearance time per movement can be calculated from

$$Q_{im} = \frac{N_{im}}{\frac{3,600}{S_{im}}} + u_{im} \qquad (3)$$

where

Q_{im} = the queue clearance time in seconds required at movement m of intersection i,
N_{im} = average number in queue at start of green on movement m of intersection i,
S_{im} = capacity flow of movement m of intersection i, and
u_{im} = queue start-up time of vehicles on movement m of intersection i.

The queue clearance option is a logical addition to the progression analysis. To allow progressive movement on the arterial when stopped queues exist, we subtracted the queue clearance times Q_{im} from the two through movement green times g_{12} or g_{14} to determine the resulting progressive through green times G_{12} or G_{14}:

$$G_{im} = g_{im} - Q_{im} \quad m = 2, 4 \qquad (4)$$

where m refers to the movements shown in Figure 2. The progression program can skip this option if desired by letting $Q_{im} = 0$ or $G_{im} = g_{im}$.

MULTIPHASE PROGRESSION OPTIMIZATION

Theory

The procedure used to determine the maximum progression bands that can be found along an arterial having multiphase signal sequences is an extension of Brooks's interference algorithm (6) illustrated by Bleyl (7). Reference is also made to Little's maximum bandwidth algorithm (5). Both algorithms analyze only two-phase progression and use the half-integer synchronization technique, which does not apply for multiphase signal operation because the inbound and outbound progression greens at an intersection having multiphase signal operation are generally of unequal lengths and are offset in time relative to one another.

Figure 1. Pilot control system site.

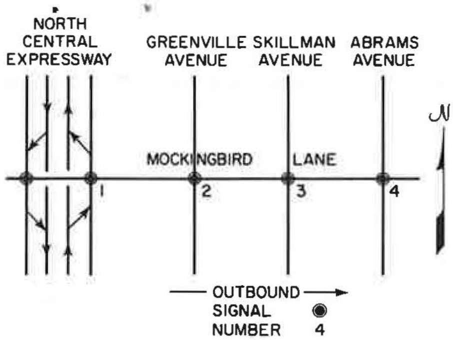


Figure 2. Traffic movements on high type of intersection.

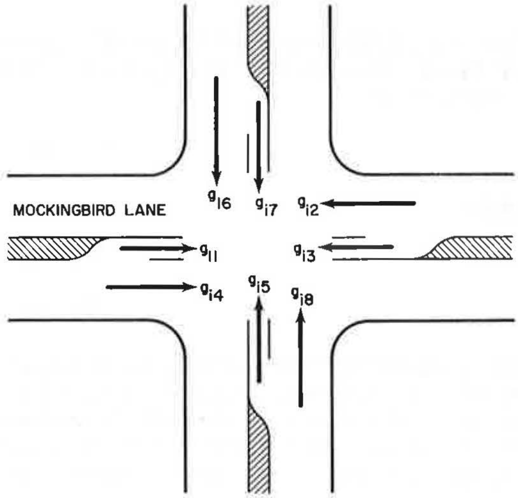


Figure 3. Relationships between traffic movement durations and resulting typical quad-left sequence.

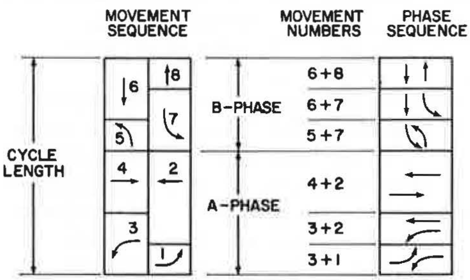


Figure 4. Four A-phase sequences.

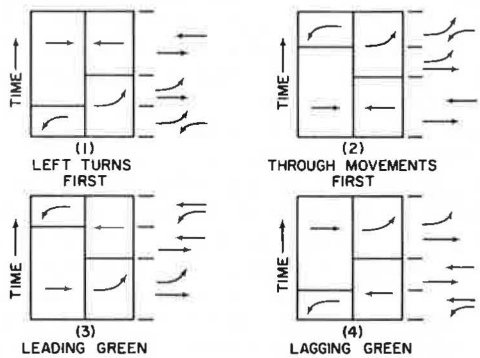
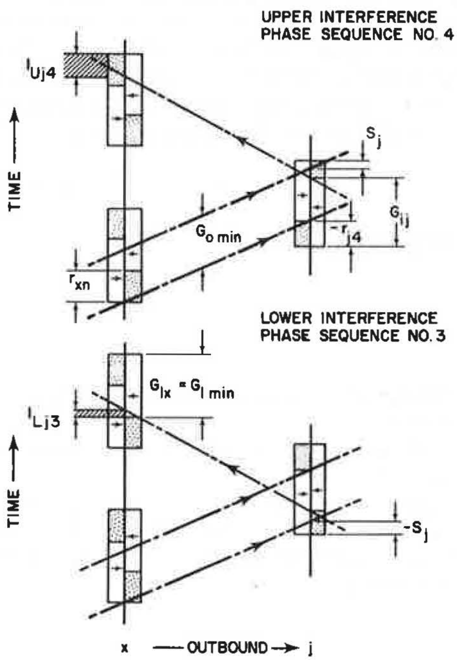


Figure 5. Example of upper and lower interferences to inbound progression band caused by an intersection.



However, Little's unequal bandwidth equation at an optimized condition does hold within given constraints. This equation is that the sum of the bandwidths at optimization is a constant, or

$$B_o + B_i = B_{max} \tag{5}$$

subject to

$$\begin{aligned} B_o &\le G_{o\ min} \\ B_i &\le G_{i\ min} \end{aligned} \tag{6}$$

where B_o equals the width of the progression band in seconds in the outbound (an arbitrary selection) direction along the arterial and $G_{o\ min}$ is the minimum outbound progressive through green. Likewise, B_i is the bandwidth and $G_{i\ min}$ the minimum progressive through green in the inbound direction.

Extending Brooks's interference theory, it can be shown that

$$B_{max} = G_{o\ min} + G_{i\ min} - I_{i\ min} \tag{7}$$

where $G_{o\ min}$ and $G_{i\ min}$ are the minimum outbound and inbound progressive greens respectively and $I_{i\ min}$ is the minimum possible inbound band interference as described subsequently. The minimization of inbound interference must be achieved without causing any outbound interference to occur. Thus, to maximize the sum of the progressive bands, the total inbound interference should be minimized.

The minimum inbound interference $I_{i\ min}$ is computed by setting the initial intersection signal phasings for outbound one-way progression as shown in Figure 5. The width of the outbound green progressive band is $B_o = G_{o\ min}$. The intersection, denoted as x, that has the minimum progressive green in the inbound direction $G_{i\ min}$ is then located. The inbound green bands of all other intersections are then projected onto this smallest inbound green to determine their interference to the inbound progression. Because these interference projections are with respect to the smallest green in the inbound direction, it is not possible for another intersection to have both upper and lower interferences simultaneously. However, it is possible for the projection of an inbound green onto the minimum green to completely cover or straddle the minimum green causing neither upper nor lower interference.

To evaluate the upper interference values I_{Ujp} for phase sequence p of intersection j, all signal phases are offset for perfect one-way progression in the outbound direction as shown in the upper section of Figure 5. The upper interference is computed by first accumulating the elapsed time from the inbound minimum green G_{ix} located at intersection x to intersection j and returning to project the upper edge of the inbound band onto G_{ix}. This total time is then scaled to modulus C and then subtracted from G_{ix}. That is,

$$I_{Ujp} = G_{ix} - (-r_{xn} + t_{xj} + r_{jp} + G_{ij} + t_{jx}) \bmod C \tag{8}$$

and after regrouping terms

$$I_{Ujp} = G_{ix} - (t_{xj} + t_{jx} - r_{xn} + r_{jp} + G_{ij}) \bmod C \tag{9}$$

where

I_{Ujp} = upper interference caused by phase sequence p of intersection j ($0 \le I_{Ujp} < C$) mod C,

G_{ix} = minimum inbound progressive green located at intersection x (i.e., $G_{ix} = G_{i\ min}$),

t_{xj} = cumulative travel time from intersection x to intersection j,

t_{jx} = cumulative travel time from intersection j to intersection x,

r_{xn} = the relative offset of G_{ix} with respect to G_{ox} with up (lag) positive for phase sequence n,

r_{jp} = the relative offset of G_{ij} with respect to G_{oj} with up (lag) positive for phase sequence p,
G_{ij} = the inbound progressive green time at intersection j, and
C = cycle length.

Travel times are considered positive if intersection j is outbound of intersection x, the intersection having the minimum green in the inbound direction. Conversely, travel times are negative if intersection j is inbound of intersection x. Travel times are computed from the directional link distances between intersections and the corresponding running speeds. Because only cumulative travel times are used, directional link speeds or distances or both between intersections may be different.

According to the lower section of Figure 5, the lower interferences are computed similarly from the following equation:

$$I_{Ljp} = (-r_{xn} + t_{xj} - S_j + r_{jp} + t_{jx}) \bmod C \tag{10}$$

or

$$I_{Ljp} = (t_{xj} + t_{jx} - r_{xn} + r_{jp} - S_j) \bmod C \tag{11}$$

where

I_{Ljp} = lower interference ($0 \leq I_{Ljp} < C$) mod C and
S_j = $G_{oj} - G_{ox}$.

Note that, in the lower interference computation, the phase sequence at intersection j is slipped down an amount S_j to reduce the lower interference as much as possible without causing any interference to the outbound band. Lower interferences are also checked for the possibility of an inbound minimum green straddle condition occurring where neither upper nor lower interference occurs. This can occur if $I_{Ljp} \geq C - (G_{ij} - G_{ix})$.

After upper and lower interferences have been computed for all of the four-phase sequences permitted at each intersection, the minimum upper I_{Uj} and lower I_{Lj} interference values are determined at each intersection j for all intersections within the progressive system.

Optimization

As noted in Eq. 7, the optimization criterion is to maximize the sum of the progression bands by minimizing the total inbound interference without causing any outbound interference to occur. The total inbound interference is the sum of the maximum upper and lower interferences that are in the solution at any time. That is,

$$I = I_{U\,max} + I_{L\,max} \tag{12}$$

where I is the total interference for a progression solution having a maximum upper interference of $I_{U\,max}$ and a maximum lower interference of $I_{L\,max}$. Either an upper or lower interference can be selected at an intersection. It is possible to select all upper or lower interferences or any combination of the two. Brooks's minimization of interference concept (6) can now be used to determine the appropriate combination of upper and lower interferences that will yield the minimum interference.

Interference Minimization

An example of the minimization of interference will be presented for the four-intersection Mockingbird Lane pilot control system. The existing intersections with their allowable phase sequences and corresponding computed upper and lower interferences are given in Table 1. These values were determined using a 70-sec cycle and a uniform speed of 40 fps in each direction for clarity of presentation.

The minimum total interference can be evaluated from Eq. 12 in the following manner. The minimum upper and lower interferences at each intersection from Table 1 are ranked in descending order according to upper interferences as given in Table 2. If all the I_{Uj} in Table 2 were used as a solution, the total interference would be the largest upper interference, 24 sec. However, if, for the first intersection that has the largest upper interference, the lower interference of 1 sec were selected, the total interference for this second trial would be $I_2 = I_{U(2)} + I_{L\,max}$ or $I_2 = 12 + 1 = 13$ sec. Continuing, if the second ranked intersection also used a lower interference, then the total interference of the third alternative would be $I_3 = I_{U(3)} + I_{L\,max}$ or $I_3 = 5 + \max(1, 5) = 10$ sec. Lastly, if all three of the possible interference intersections used lower interferences, then the total interference of the fourth trial would be $I_4 = I_{U(4)} + I_{L\,max}$ or $I_4 = 0 + \max(1, 5, 11) = 11$ sec.

Thus, the minimum possible interference for this cycle $I_{i\,min}$ is 10 sec from trial three. This minimum is obtained by selecting lower interferences for the first two listed intersections (intersections 4 and 3) and upper interferences for the remainder (intersection 2 in this case). If the upper and lower interferences that yield the minimum interference are known, the corresponding phase sequence to be used at each intersection is determined. As given in Table 2, intersections 4, 3, 2, and 1 would use the phase sequences shown in Figure 4 of 1, 4, 4, and 3 respectively.

The maximum sum of the inbound and outbound progression bands B_{max} is determined from Eq. 7.

$$B_{max} = G_{o\,min} + G_{i\,min} - I_{i\,min}$$

For this example, $G_{o\,min} = 20$ sec, $G_{i\,min} = 15$ sec, and $I_{i\,min} = 10$ sec. Thus, $B_{max} = 20 + 15 - 10 = 25$ sec.

In the example presented, the intersection having the minimum green in the inbound direction, intersection x, was the diamond interchange. It has only one possible phase sequence. Thus, only one analysis of interferences projected onto it had to be made. At the present time, the progression program has to evaluate as many total interference calculations, similar to the previous example, as are the number of phase sequences existing at the intersection having the minimum inbound green.

The diamond interchange, which had the minimum inbound green, also had the minimum outbound green. Thus, the phase sequence timing band could not be "slipped down" to reduce the initial upper interference value of 24 sec at the first or all upper interference trials. Normally, some reduction can be achieved. Because of the way the program is structured, no similar all lower interference evaluation is required.

Selection of Cycle Length

The system cycle length that the progression program will finally select is the one that will maximize progression band efficiency. The procedure is similar to the two-phase signal operation described by Bleyl (7). The percentage of efficiency E_c of an optimal progression solution for a given cycle length C is defined as

$$E_c = \frac{100 \times B_{c\,max}}{2C} \tag{13}$$

where $B_{c\,max}$ is the maximum sum of the progression bands at cycle length C. In the example being presented, $B_{70\,max} = 25$ sec and $C = 70$ sec; therefore, the percentage of progression efficiency is

$$E_{70} = \frac{100 \times 25}{2 \times 70} = 14.9$$

The upper curve in Figure 6 shows the variation in efficiency of the optimal progression solutions for the Mockingbird Lane arterial system as a function of system cycle lengths ranging from 50 to 80 sec in 1-sec increments. The most efficient cycle length is 53 sec, which results in an efficiency of 28.4 percent. The 70-sec cycle is one of the least efficient cycle lengths that could have been selected. Selecting the maximum efficiency rather than the maximum sum of the progression bands results in the selection of a shorter cycle length; this is desirable because it tends to further minimize delays at the intersections (8).

The lower curve in Figure 6 is an efficiency plot of the optimal progression solutions for the Mockingbird Lane arterial system but where the three high types of intersections were restricted to operate with only the left-turns-first sequence (phase sequence number 1) permitted. The differences between the two curves in Figure 6 clearly demonstrate the improvement in progression efficiency that may occur in variable-sequence multiphase progression analysis. These plots should not be interpreted, however, as a direct comparison of the progression efficiencies of multiphase progression analysis and conventional two-phase signal operation.

In two-phase operation, approximately 60 percent of the cycle is devoted to the arterial phase, whereas, with multiphase operation having protected left turns, perhaps only 40 percent of the cycle is devoted to arterial through movements. If the arterial has large unprotected turning movements, rather sizable stopped queues may develop that may block the through movements and, as a consequence, reduce the actual progressive green time of a two-phase system from 60 to 40 percent of the cycle or less. In this case, it may be possible to provide a multiphase protected turning movement signal operation that has as much if not more actual progression than a conventional two-phase signal system. In addition, traffic flow using multiphase control would result in smoother, more orderly, and safer traffic operations.

Attainability

Attainability is a measure of the ability of the progression strategy to utilize the available progressive greens of the intersections within the system. Attainability shows how good the progression solution is compared to the maximum possible solution for given traffic conditions and green splits. The percentage of attainability A_c for a given cycle length is defined as

$$A_c = 100 - \frac{I_{i\ min}}{G_{o\ min} + G_{i\ min}} \times 100 \tag{14}$$

Thus, if $I_{i\ min}$ is reduced to zero, the attainability would be 100 percent. An attainability plot for the 50- to 80-sec cycle lengths previously evaluated for the Mockingbird Lane control system is shown in Figure 7. This plot shows that several solutions with different cycle lengths have the largest progression bands that could have been determined or 100 percent attainability. The inbound and outbound progression bands must be equal to the minimum greens in each direction to reach 100 percent attainability.

Time-Space Diagram

It has been previously shown that the most efficient progression occurs at a 53-sec cycle length within the Mockingbird Lane pilot system for the given traffic conditions. As illustrated in the corresponding time-space diagram (Fig. 8), the solution uses phase sequences 3, 4, 4, and 1 for the four intersections 1, 2, 3, and 4 respectively. That is, the diamond interchange uses the leading phase sequence, intersections 2 and 3 the lagging phase sequence, and intersection 4 the left-turns-first sequence. Uniform speeds were used for clarity. This solution has an efficiency of 28.4 percent and an attainability of 100 percent. The relatively low efficiency is due to the low minimum bandwidth limits placed on the system in both directions by the diamond interchange. The attainability of 100 percent shows that no interference to the progression bands occurs. Thus, the phase sequences of the three high types of intersections were selected such that they did not interfere with the progression bands generated from the diamond interchange. Efficiencies on the order of 35 to 40 percent would likely have been obtained had the diamond interchange been a high type of intersection.

Testing

Although this example has only four progressive signals, the general-use progression program can analyze any practical number of signals. It has been tested against the 10-

Table 1. Upper and lower interferences by phase sequence for Mockingbird Lane arterial system.

Intersection	Intersection Number	Phase Sequence[a]	I_{UJP}	I_{LJP}
North Central Expressway	1	3	0	0
Greenville	2	1	17	20
		2	30	16
		3	9	28
		4	5[b]	11[b]
Skillman	3	1	26	8
		2	16	18
		3	12[b]	24
		4	18	5[b]
Abrams	4	1	30	1[b]
		2	24[b]	5
		3	26	13
		4	35	4

[a]Phase sequences are shown in Figure 4.
[b]Minimum upper or lower interference at intersection.

Figure 6. Optimal progression efficiency as a function of cycle length for Mockingbird Lane system.

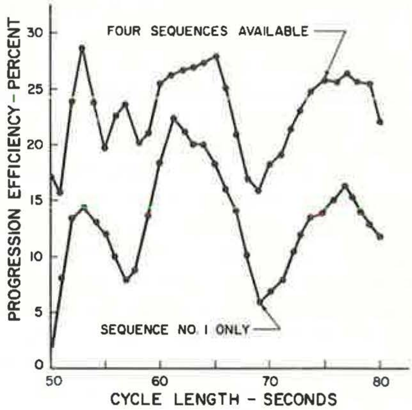


Table 2. Ranking of upper interference values for determining minimum interference for Mockingbird Lane arterial system.

Rank k	Intersection Number	Minimum I_{UJ}	Phase Sequence	Minimum I_{LJ}	Phase Sequence
1	4	24	2	1[a]	1
2	3	12	3	5[a]	4
3	2	5[a]	4	11	4
4	1	0[a]	3	0	3

[a]Interferences used in minimum interference solution.

Figure 7. Relationships between attainability and efficiency for Mockingbird Lane system.

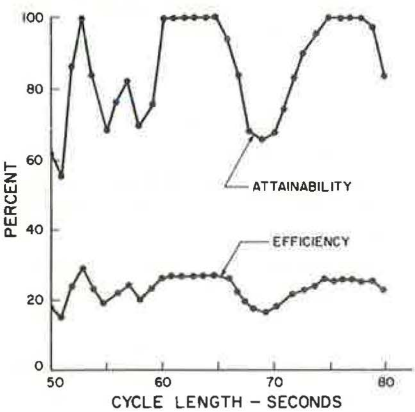
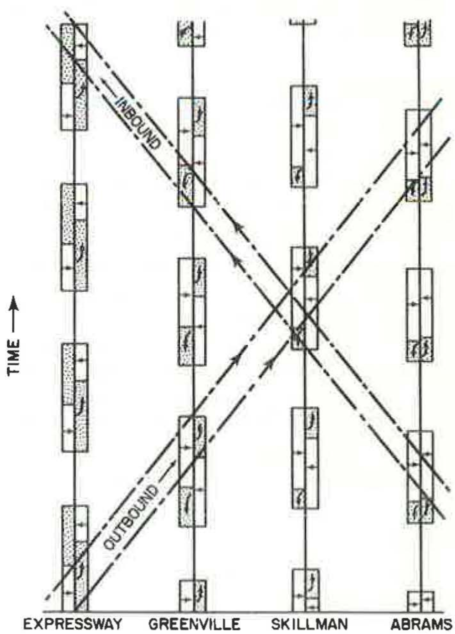


Figure 8. Optimal time-space diagram and selected phase sequences for Mockingbird Lane system.



intersection conventional two-phase progression solution presented by Little (5) with identical optimal results. The real-time control version has been implemented on the Mockingbird Lane computer control system. Preliminary travel time studies reveal that the expected high-quality two-way progression is obtained. Variable-phase sequencing within the arterial A-phase has not caused the motorists traveling the arterial any noticeable difficulty.

SUMMARY

A highly flexible general-use computer program has been developed that can be used to determine the most efficient optimal progression along an arterial where the signal phasing can range from the conventional two-phase operation to the flexible selection of multiphase sequences. The program can also compute the initial phase splits if desired. Any practical number of intersections can be analyzed on most digital computers. Intersection types can include normal, jogged, high type, and diamond having three-phase or four-phase with overlap operation. Speeds or distances or both between intersections can also be different in each direction.

The program was developed with the objective of providing a new progression analysis technique that could be used to provide more efficient traffic operations and a higher level of service on signalized traffic facilities. The traffic operations characteristics of the program appear to be well suited for computer control. However, the program can also serve to analyze more typical progression problems such as the desirability of adding leading or lagging left turns at signalized intersections currently having two-phase operation.

ACKNOWLEDGMENT

This research was developed within the Dallas urban corridor project conducted by the Texas Transportation Institute for the Federal Highway Administration in cooperation with the Texas Highway Department and the city of Dallas. The contents of this paper reflect the views of the authors who are responsible for the facts and the accuracy of the data. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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MEANING AND APPLICATION OF COLOR AND ARROW INDICATIONS FOR TRAFFIC SIGNALS

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The Manual on Uniform Traffic Control Devices for Streets and Highways sets forth the conditions under which combinations of signal colors and turn arrows should be used at signal-controlled intersections. These recommendations have been generally accepted and complied with except in the case where a left-turning movement at an intersection is to be terminated while the through movement continues. In many instances, due to the physical limitations of an intersection or for reasons of economy, it is difficult to comply with this standard. Present practice at such locations is to mount a fourth lens displaying a turn arrow either on the through face or adjacent to it. Installations such as this result in a wide variety of clearance interval indications. This research project was designed to develop a standard for this situation. A literature review and mailed questionnaire were employed to determine present practices. A controlled laboratory study, utilizing both color movies and color slides, investigated 19 signal indications for their effectiveness in conveying their intended message to the driver. The collection of accuracy and reaction time data was analyzed by an analysis of variance and the Newton-Keuls test. Four of the indications proved superior and were tested further under actual field conditions. Based on the analysis of the driver performance data recorded in the field study, a single indication was recommended.

•SATISFACTORY RESULTS from traffic signal operation require a uniform understanding of signal color and arrow indications. The Manual on Uniform Traffic Control Devices for Streets and Highways (17) sets forth the conditions under which combinations of signal colors and turn arrows should be used at signal-controlled intersections. Recommendations contained in the Manual have been generally accepted and complied with; however, there is one notable exception, the case in which a turning movement at an intersection is to be cut off while the through movement continues. Present practice at such locations is to mount a fourth lens displaying a turn arrow either on the through face or adjacent to it. Installations such as this result in a diversity of clearance interval indications. Although this practice is widespread, it does not conform to the existing standard and has been a matter of some concern to various interested persons and groups. In this regard it was recommended as early as September 1966 in the report from the Traffic Control Devices Workshops sponsored by the Institute of Traffic Engineers that a standard be developed for this situation. Further concern has been shown by members of the Traffic Signal Committee of the National Joint Committee on Uniform Traffic Control Devices, who noted the present inconsistency in the use of signals for controlling separate turning movements and offered several suggestions for improving the situation. However, the majority of the suggested solutions appear to be based on the personal experiences of the committee members rather than on any verified research.

In brief, the problem is as follows: There is no uniform treatment of clearance interval indications at intersections where a through phase (no turns) follows a phase

Publication of this paper sponsored by Committee on Motorist Information Systems.

when turns are permitted, and separate signal faces cannot be provided for the approaches.

PRESENT PRACTICE QUESTIONNAIRE

A total of 1,302 questionnaires were sent to traffic engineers throughout the United States and Canada. Of these, 20 percent were completed and returned. To analyze the data, we classified the questionnaires into 11 methods of left-turn signal indications. Five geographical regions were also established in an effort to determine what methods, if any, might be of a regional nature. Figure 1 shows the left-turn signal indications and phasing for each of the following 11 methods.

Method 1 is generally referred to as the exclusive left-turn method and is covered in the Manual regulations. It involves the use of a separate left-turn lane and a separate left-turn signal face consisting of circular red and yellow lenses and a green left-turn arrow lens. This type of arrangement is usually found at channelized intersections. Separate phases are usually provided for in this situation, and left turns may be made from both approaches simultaneously.

Method 2 is a leading left-turn split-phase signal indication. It involves the use of a signal face consisting of circular red, yellow, and green lenses and a green left-turn arrow lens. This signal face is used by both through traffic and left-turning traffic both using the same traffic lane. This arrangement is generally found at unchannelized intersections.

Method 3 is essentially the same as method 2 except in this case the type of intersection and the location of the signal are different. This method is also a leading left-turn split-phase indication. It involves the use of a separate signal face for left-turning traffic and is usually used in conjunction with a separate left-turn lane.

Method 4 is also a leading left-turn split-phase indication. It generally involves the use of a signal face consisting of red, yellow, and green lenses and a green left-turn arrow lens. The clearance interval for the termination of the protected left-turn movement is indicated by the disappearance of the left-turn arrow leaving only the red indication.

Method 5 is also a leading left-turn split-phase indication. It generally involves the use of a signal face consisting of circular red, yellow, and green lenses. The signal face controlling left-turning traffic may be a separate one or one that is used jointly by left-turning and through traffic. For the most part, however, this type of arrangement is used at channelized intersections with separate left-turn lanes.

Methods 6 and 7 have a lagging left-turn phase. In contrast to a leading left-turn phase, these sequences of indicating a protected left-turn movement take place at the end of the interval provided for the through traffic on the same approach as the left-turning traffic. They generally involve use of circular red and yellow and sometimes green lenses and a green left-turn arrow lens. The signal face controlling the left-turn movement may or may not be used by the through traffic.

Method 8 is a leading split-phase left-turn indication. The results of the questionnaire indicated that, with but one exception, its use is primarily limited to Canada. It involves the use of a signal face consisting of circular red, yellow, and green lenses and a green left-turn arrow lens. For the cases reported it has primarily been used at intersections where through traffic and left-turning traffic use the same lane.

Method 9 is also a leading split-phase operation. The method involves the use of a separate lane and separate signal face to control the movement of left-turning vehicles. The signal face consists of circular red, yellow, and green lenses and an additional green left-turn arrow lens.

Method 10 is also a leading left-turn split-phase indication. However, this method differs from the ones previously described in that permissive turns are not allowed. This is accomplished by the use of a vertical arrow lens for through traffic movement. With this method, through and left-turning traffic use the same traffic lane and signal face.

Method 11 is also a leading split-phase indication. In this arrangement, through and left-turning traffic utilize the same traffic lane and signal face. The signal face consists of circular red, yellow, and green lenses together with yellow and green left-turn lenses placed below the above-mentioned lenses.

Summary

Thirty-three percent of the respondents reported using left-turn signalization method 1, 20.4 percent method 2, and 13.1 percent method 3. Methods 4 and 6 accounted for 8 and 9 percent respectively of the replies. The remainder of the replies were almost evenly divided among all the other left-turn methods.

Analysis of the replies on a regional basis, according to the 11 left-turning methods, indicates the following trends in their usage. The northeastern region reported the highest use of method 1 with the western region next. California was the chief user of this method. Method 2 seemed to be used exclusively in the northeastern, southern, and central regions.

Method 3 was found to be most prevalent in the southern region; however, with the exception of Canada, all other regions reported some limited use of this method. Method 4 is predominantly used in the central region. Method 5 is used exclusively in Canada, particularly in the northeastern provinces. Methods 6 and 7 are most used in the northeastern and southern regions, with the southern region reporting the greatest usage of these methods.

With the exception of Philadelphia, use of method 8 is limited exclusively to Canada, most of the installations being in the western provinces. Methods 9, 10, and 11 were used in all regions except Canada.

Discussion of Questionnaire Replies

It may be concluded from the questionnaires that a variety of left-turn signal indications are being used across the country. Although the analysis indicated certain general trends in the use of the various methods, it may also be concluded that the differences are by no means restricted to the limits of the established geographical regions. Within a single state, as many as five methods may be found in use.

The analysis also shows that at the present time the decision to install a particular left-turn method is often not greatly influenced by the Manual. Different agencies place different emphasis on safety, efficiency, uniformity, and economy. The result is that each agency seeks a method that best fits its individual needs, and the general tendency is for each agency to say that its signal indication method performs satisfactorily.

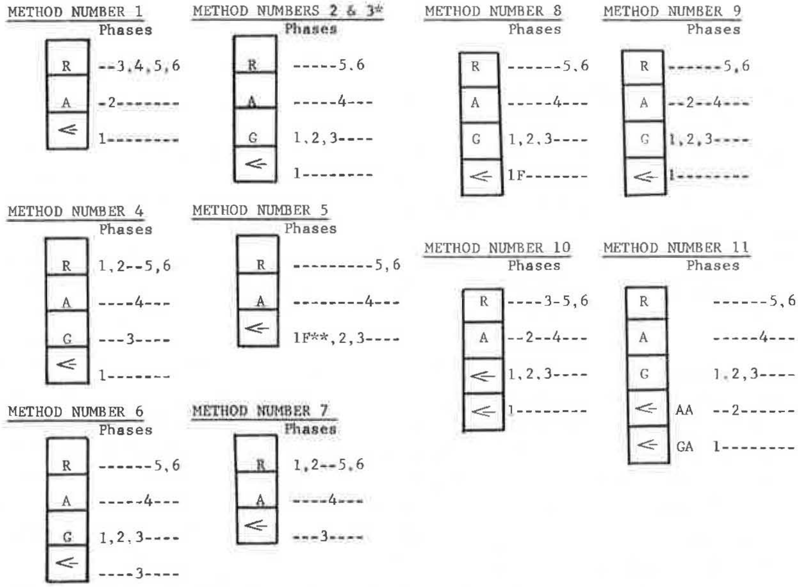
LABORATORY STUDY

The laboratory study was an in-depth investigation of the clearance interval indications at intersections where a through phase (no turns) follows a phase when turns are permitted and separate signal faces cannot be provided for each of the approach lanes. The purpose was to determine what type of signal indication, shown to the left-turning traffic, would best convey to the driver that (a) he may make a left turn, (b) the left turn is about to terminate, and (c) he must yield to opposing traffic. The final product of this portion of the study was a list of several sequences of signal indications that proved to be superior in achieving these three objectives. The basis for evaluation was provided by an information processing model.

Method

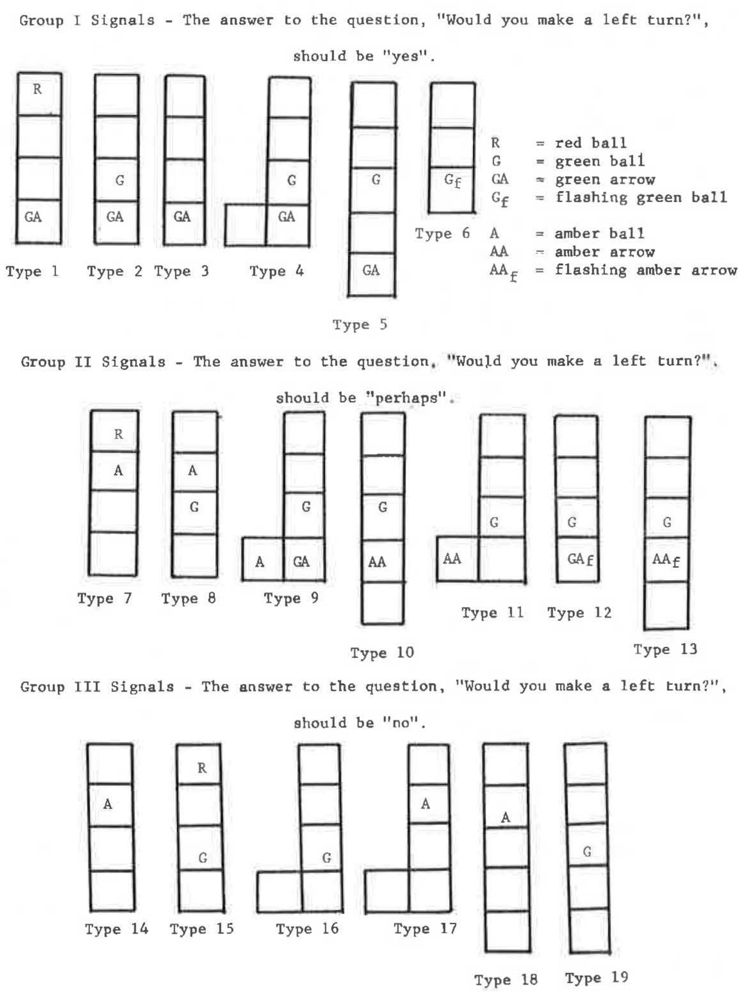
Subjects—A total of 49 male and female subjects were used in the experiment. These subjects were divided into two groups. The main study group consisted of 40 persons whose ages ranged from 18 to 30 and whose driving experience ranged from 0 to 13 years. The second group contained older drivers for the purpose of comparing their understanding of protected left-turn signals with that of the main study group. This group varied in age from 31 to 64 years, and their driving experience ranged from 15 to 44 years.

Figure 1. Methods of left-turn signal indication.



* Difference is the location of the signal faces and the type of intersection.
** F indicates flashing signal phase.

Figure 2. Signal configurations.



The subjects included students and staff of West Virginia University and housewives. The total 49 subjects had some significant driving experience in 16 states and the District of Columbia.

Stimulus Material—Nineteen left-turn signal indications in 14 signal sequences were used in this experiment. These were selected based on the results of the present practice questionnaire and the researcher's judgments. Figure 2 shows the 19 types of signal indications.

The subject's total stimulus contained three sections that were presented simultaneously on two movie screens. The first part of the stimulus was a 35-mm color slide of an intersection, which presented the subject with a visual reference on the type and layout of the intersection he was attempting to maneuver through. The second part of the stimulus was the question, "Would you make a left turn?" The first and second parts of the stimulus remained constant during the entire study period. The final portion of the stimulus consisted of a traffic signal indication, which was the changing portion of the stimulus, and was presented by a 35-mm color slide or Super 8-mm color film for the flashing indications. With each change of signal indication, the subject was to answer the question "Would you make a left turn?" by depressing one of three response buttons, yes, perhaps, and no. The signal indications were divided into group I signals (yes answer), group II signals (perhaps answer), and group III signals (no answer) based on the meanings that practicing traffic engineers intended the signals to convey to drivers.

Procedure—In designing the experimental procedure for this study, the information processing concept of the human operator was employed. The information processing model has provided the fundamentals to much of our present understanding of the factors determining speed and accuracy of human performance. Because statistical evaluation of this laboratory investigation depended on the measures of reaction time and accuracy, it was considered appropriate to apply this proven concept.

After completing a color discrimination and visual acuity test, the subjects were seated before the projection screens. The experimenter then briefly described the purpose of the study, gave instructions to the subjects, and answered questions concerning the subjects' participation. Following a short practice period, the main study period began, during which the subjects' reaction times and accuracies were recorded for each signal presentation.

The investigation was conducted in two parts. Part one tested the response of the subjects to individual signal indications, and in part two the subjects viewed an entire signal sequence or cycle that contained four or five signal indications and responded to these.

For the final portion of the experiment, the subjects were asked to complete a questionnaire that included their personal opinions regarding the signal indications they understood best.

Results

The results indicate a difference in the ability of the signal indications tested to convey a given message to the subject participating in the experiment. The following results were verified by application of the analysis of variance (ANOVA) and the comparison of statistical differences. (Results of the ANOVA are given in Table 1.)

1. The four-signal-head configurations used in the laboratory study did not influence the accuracy or the reaction time of the subjects. An example of this was the green arrow indication, which was easily and correctly understood regardless of its position or accompanying signals.
2. The solid red stop signal had a high degree of population expectancy, which means that the driving public has established a habit or clear logical relationship between the stimulus (red signal) and the response (stop). This was confirmed by part two of the research in which the red light obtained a near perfect accuracy (one miss out of 560 responses) and a low reaction time.
3. None of the three flashing signals tested in the experiment proved to be effective inasmuch as their meanings were not comprehended by the subjects so readily as the competing nonflashing indications.

4. The zone of uncertainty was greatest for the signal indications in group II (perhaps), which shows that the concept of the clearance interval is the most difficult for the subjects to understand.

5. After parts one and two of the experiment had been evaluated, it was concluded that sequences 7, 12, and 13, shown in Figure 3, should be field tested along with sequence cycle 2, which was not tested here because it uses a time offset to accomplish the equivalent of the clearance interval.

6. The data collected from the older study group supported the finding of the main study group. This suggests that age differences do not affect the meaning conveyed by the signal indications.

Discussion of Laboratory Study

The main objective of the laboratory investigation was accomplished in that the number of signal sequences to be field tested was reduced from 14 to three. The argument could be posed that all the signal sequences should be field tested. However, this was not practical because money and manpower were limited. Also it would require a minimum period of 14 months to field test the signal sequences, which would mean working through several different seasons of the year.

The results of this laboratory investigation needed to be field tested to determine whether field differences exist between the signal sequences inasmuch as the subjects in the laboratory were required to perform only one task with no outside distractions. The actual driving task is not so simple because many distractions may be present, particularly when the driver is approaching a busy intersection.

FIELD STUDY

The purpose of the field study was to evaluate the effectiveness of the four left-turn signal indication sequences (Fig. 3), recommended in the laboratory study, under actual operating conditions. The field test was designed to investigate driver response to different signal indication sequences during a protected left turn. A study of the driver's response to the display of the signal indication would explain, within certain limitations, whether the intended meaning of the signal was conveyed and fully understood. On the basis of field observations, it was possible to rate the signal indication sequences according to accuracy of response by the driver, time of driver response, use and misuse of lanes, driver hesitation, number of signal violations, apparent indecision of the driver, and conflicts that occurred during the investigation.

Method

The intersection of Patteson Drive and University Avenue (Fig. 4) was chosen as the study location for the following reasons:

1. Proximity to West Virginia University,
2. Geometric configuration,
3. High percentage of local, repeat users, and
4. Fluctuating traffic volumes throughout the day.

Magnetic loop vehicle detectors were embedded in the roadway, their existence concealed from passing motorists. The detectors were actuated by left-turning vehicles as each approached the intersection and exited from the intersection and by vehicles proceeding through the intersection in either direction. The response of the detectors and the traffic signal phasing were recorded on a constant-speed strip chart recorder. This established a timetable of activity for the study area. Times were recorded for individual vehicle movements.

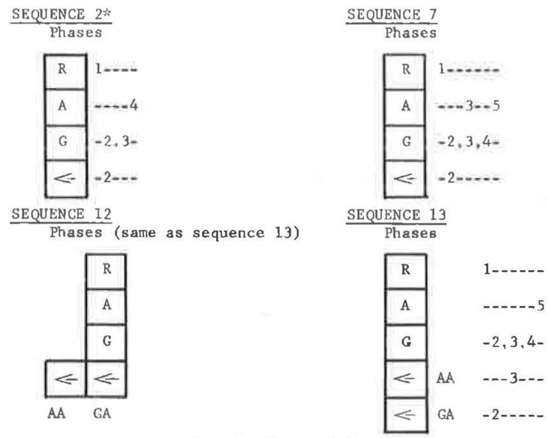
An observer was on duty during each data collection period. From a concealed vantage point, his view of the traffic was unobscured. The observer noted any unusual traffic movements, potential accident situations, and the like. He made a note of the situation and by remote control activated an event marker on the chart recorder to indicate when the situation occurred.

Table 1. Calculated F-ratio from ANOVA tables.

Comparison	Accuracy			Reaction Time		
	Group I	Group II	Group III	Group I	Group II	Group III
Mean squares for subjects versus experimental error	10.141	55.866	21.825	10.798	24.968	25.943
Mean squares for signal indications versus experimental error	1,059.38	256.89	31.99	113.60	20.22	24.00
Mean squares for interaction term versus experimental error	6.384	11.946	3.819	1.638	1.549	1.109[a]

[a]No significant difference; for all others there was significant difference.

Figure 3. Signal sequences tested in field study.

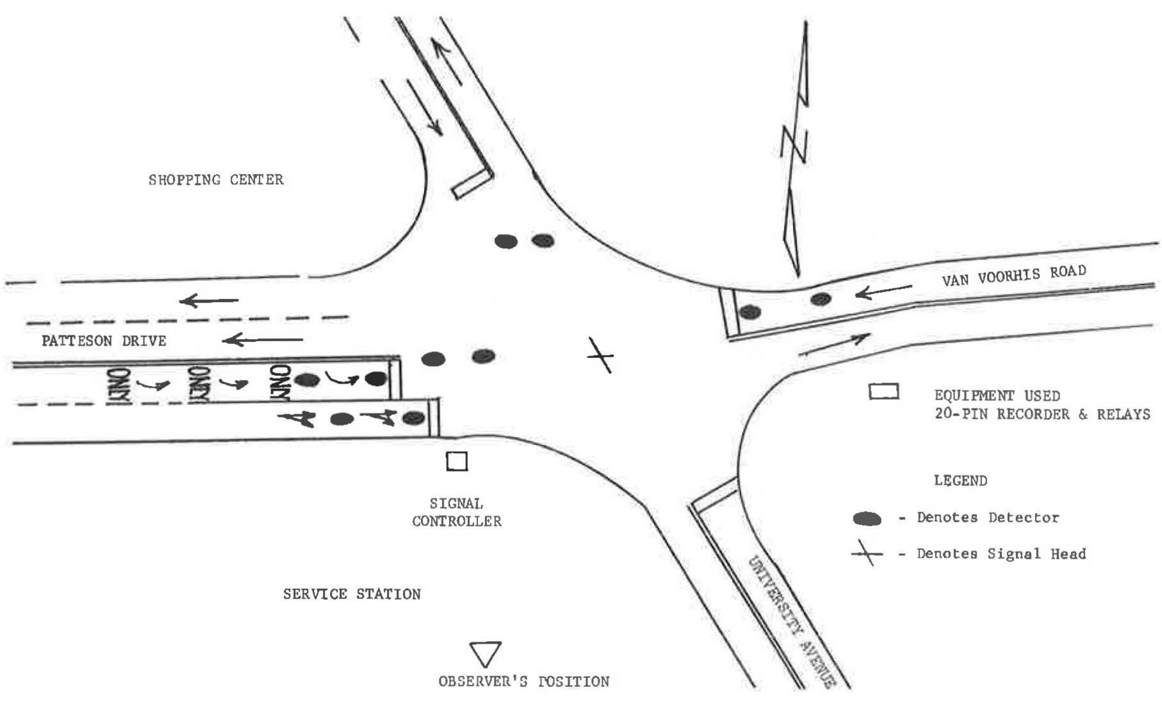


Length of Signal Presentations
(in seconds)

Phases	1	2	3	4	5	Total
Sequence 2	36.4	7.0	23.4	3.2		70.0
Sequences 7, 12, 13	36.4	7.0	3.2	20.2	3.2	70.0

* Uses a time delay for the opposing traffic's green phase to accomplish the equivalent of the clearance interval.

Figure 4. Field study intersection.



The signal indication sequences were presented in random order. Following the installation of each sequence, a 7-day adjustment period was allowed for the traffic to become familiar with the new condition. The data collection phase then began and covered the next 2-week period.

To determine the driver's understanding of the different traffic signal indication sequences, we recorded the following data:

1. The starting-up times of left-turning, through, and right-turning vehicles entering the intersection at the start of left-turn indication;
2. The termination of each indication phase in the four signal sequences;
3. The start of each indication phase for the four signal sequences; and
4. The time interval between the termination of the left-turn movement and the start of the opposing through movement.

In addition to this, the starting-up times for the opposing through movement (westbound) were recorded. For the purpose of this study, starting-up time for eastbound traffic is defined as the time interval between the start of the left-turn and green ball signal indication and the beginning of the first queued vehicle at the intersection. The starting-up time for westbound traffic began with the start of the green ball signal indication and ended with the movement of the first vehicle.

Environmental factors affecting driver performance at the study area were also considered. Data were collected only on those days that had ideal weather conditions. On rainy days, when the road was wet, or when visiblity was poor due to heavy fog, no data were collected. All data collection at the intersection was done during daylight hours.

Following these considerations, the field study was conducted to determine which signal indication sequence best conveyed its intended message to the driver. The following equipment, the location of which is shown in Figure 4, was used to record the data:

1. Magnetic loop vehicle detectors,
2. Marblelite traffic signals,
3. Amplifiers and radio receiving equipment,
4. Esterline Angus 20-pen inklers recorder, and
5. Signal control box.

With this equipment it was possible to record the start of each signal phase, the starting-up times of the first vehicles in the left- and right-turning lanes, road violations made by the motorists, number of cars passing through the intersection during the study periods, and times at which left-turn vehicles started their movement (entered the conflict area) and completed their movement (cleared the conflict area). The starting-up times of the opposing through traffic and the times when the vehicles entered the intersection were also recorded.

Results

To analyze the data collected in the field study required that, first, the data be converted from the 20-pen recorder tape to a more convenient form. This was accomplished by designating the start of each green arrow indication as the zero time base and recording the time for vehicular events with respect to this datum. The data were transferred to prepared forms and then punched into computer cards for processing. Data were analyzed both graphically and statistically.

Data were tabulated to show the number of vehicles entering the intersection after the start of the green arrow indication for each signal sequence. Included were all vehicles entering the intersection after the start of the green arrow until the time when the green arrow went out and left-turning vehicles began to yield the right-of-way to the opposing through traffic movement. The utilization of the left-turn interval also shows the extent to which drivers are preempting the right-of-way from the opposing through traffic movements. Inspection of the data shows that left-turning drivers tended to yield the right-of-way more often with signal indications 12 and 13 than with signal indications 2 and 7.

Vehicle starting-up times were extracted from the data. These starting-up times indicate the extent of driver perception of the left-turn signal display. The starting-up times would indicate whether the meaning of the signal display was understood by the motorist. The starting-up times for the first vehicle and the second vehicle in the left-turn lane were subjected to an ANOVA. The results of these tests are given in Table 2. Both showed significance at the 5 percent level. This meant that differences in starting-up times did exist between the types of signal sequences. Inspection of Figure 5 shows that signal sequences 12 and 13 have lower starting-up times for first and second vehicles than signal sequences 2 and 7.

An ANOVA was also performed using the starting-up time data from the through or right-turn lane vehicles. These data proved statistically significant for both the first and second vehicles. This showed that a difference exists in the ability of the four signal sequences to encourage quicker starting times. Bar graphs (Fig. 5) show that signal sequences 12 and 13 have shorter starting-up times.

The conclusion drawn from these tests was that the starting-up times for left-turning and right-turning and through vehicles were significantly longer for signal indications 2 and 7 than for signal indications 12 and 13. However, there is no significant difference between the mean starting-up times for signal indications 12 and 13.

A comparison of the total time required for a left-turning vehicle to travel through the intersection was conducted. The results are shown in Figure 5. This time was measured from the point the car entered the intersection until its rear bumper exited the conflict area. The results of the ANOVA given in Table 3 show that statistically significant differences exist among the signal sequences. Analysis of the mean times shows that signal sequences 2 and 7 have longer times than sequences 12 and 13.

The establishment of the critical difference was based on the length of time that would be required for a vehicle to clear a 12-ft lane at an average speed of approximately 24 fps. It was postulated that a vehicle starting ½ sec later than normal would allow one vehicle less to clear the intersection during each signal cycle. The result of this would be a reduction in the capacity of the intersection. The lower starting-up time test results indicate that motorists understood the displays of signal sequences 12 and 13 better than signal sequences 2 and 7. Furthermore, the results also indicate that the efficiency of the intersection was increased when signal sequences 12 and 13 were employed.

For the purpose of this investigation, a violation was defined as the movement of a vehicle into the conflict area after the start of the left-turn clearance interval. Crossing into the conflict area after the start of the clearance interval means that the vehicle could be trapped in the intersection and thus interfere with the right-of-way of the opposing through traffic. The investigation showed that the resulting preempting was more likely to occur at times when the ratio of opposing through traffic to left-turn traffic was low. This general trend was observed during the testing of all four traffic signal indications. A greater number of violations were recorded for sequences 2 and 7 than for sequences 12 and 13.

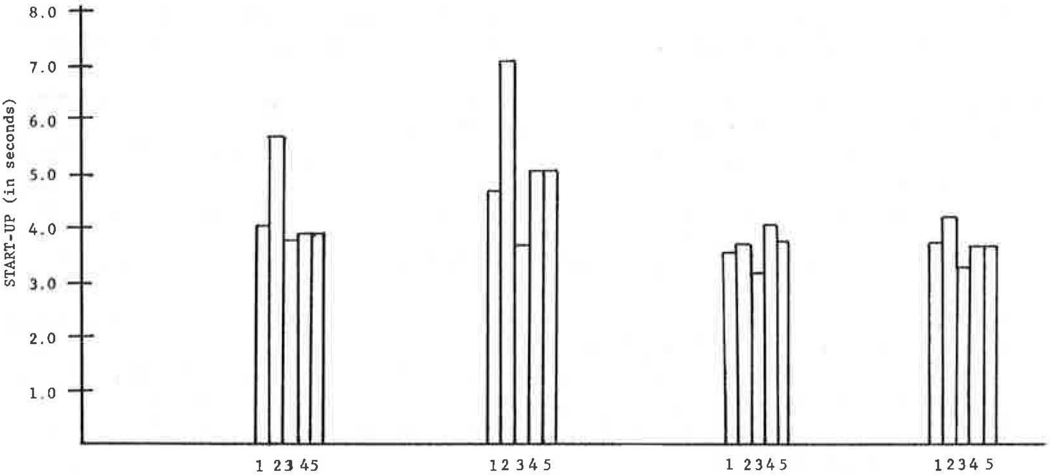
The results can be briefly summarized as follows:

1. Signal indication sequences 12 and 13 proved superior in conveying the message that the driver had a protected left turn, that the protected left turn was about to terminate, and that the driver did not have a protected left turn;
2. Sequences 12 and 13 encouraged left-turning motorists to yield the right-of-way more often than sequences 2 and 7;
3. The starting-up times for left-turn and right or through vehicle movements were lower for signal sequences 12 and 13 than for sequences 2 and 7;
4. Signal indication sequence 7 proved to be the most ineffective of the four sequences tested, for there was evidence of driver hesitation in both approach lanes during the third indication phase of this signal;
5. Fewer traffic flow violations resulted with sequences 12 and 13 than with sequences 2 and 7; and
6. Speeds of left-turning vehicles were affected by the signal indication sequence, and driver left-turning speeds were higher with signal sequences 12 and 13.

Table 2. Statistical analysis of starting-up times.

Vehicle Position	Lane	Source	df	SS	MS	F
1	Left	Signal sequences	3	33.264	11.088	7.921
		Residual	165	230.969	1.399	
		Total	168	264.233		
2	Left	Signal sequences	3	42.299	14.099	21.255
		Residual	20	13.267	0.633	
		Total	23	55.566		
1	Right or through	Signal sequences	3	19.637	6.546	6.717
		Residual	336	327.443	0.975	
		Total	339	347.080		
2	Right or through	Signal sequences	3	85.705	28.568	37.643
		Residual	226	171.516	0.759	
		Total	229	257.221		

Figure 5. Average starting-up times.



1-first vehicle in queue turning left from Patteson Drive; 2-second vehicle in queue turning left from Patteson Drive; 3-first vehicle in queue proceeding straight through intersection: 4-second vehicle in queue proceeding straight through intersection; 5-time required for vehicle to make complete left turn movement (time spent in conflict area).

Table 3. Statistical analysis of total time left-turning vehicle spent in intersection.

Source	df	SS	MS	F
Signal sequences	3	33.182	11.061	6.175[a]
Residual	212	380.868	1.797	
Total	215	414.050		

[a] $p < 0.05$.

Discussion of Field Study

The field study results show signal sequences 12 and 13 to be superior to sequences 2 and 7 in their ability to convey the intended message to the driver. The field test showed no significant difference between sequences 12 and 13. However, this was not totally unexpected inasmuch as the only difference between the two was the physical arrangement of the five faces. Fewer traffic violations were noted for sequences 12 and 13, and starting-up times were reduced over those of sequences 2 and 7. Sequence 7 proved to be the least effective in that it seemed to encourage driver hesitation during the amber interval.

Based on ease of installation and driver expectation, it is further recommended that signal sequence 13 be given preference over sequence 12.

ACKNOWLEDGMENTS

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the West Virginia Department of Highways or the U.S. Department of Transportation, Federal Highway Administration.

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ANALYTIC SURVEY OF SIGNING INVENTORY PROCEDURES IN VIRGINIA

Fred R. Hanscom, Virginia Highway Research Council

ABRIDGMENT

•THE NEED for a survey of signing inventory procedures within the Virginia Department of Highways was prompted by two issues: the variability of inventory uses among districts and the question of implementing a uniform inventory procedure throughout the districts. Of particular concern in reviewing the procedures were the format of the inventory forms, the applications of inventory information and the time and effort required to maintain the inventory, the reference uses of the inventory, and the needed revisions in district inventory procedures and formats.

FORMATS OF INVENTORIES

A diversity of inventory uses among districts necessitated a variety of formats for the record-keeping systems. In general, formats were one of two general classes: log sheets and index cards. The log sheet systems consisted of sheets on which information was contained for a number of signs. The index card system consisted of files of cards, each of which contained complete information for a single sign. Both classes generally involved filing by county, route number, and milepost.

Of the eight districts surveyed, five used the log sheet format and two used index cards. The eighth district, which covered a large geographic area, used a combination of both. A master log sheet kept in that district's office was used as a control form for sets of index cards representing signs in each residence within the district.

USES OF INVENTORY INFORMATION

Certain general uses of the inventory systems could be seen to be applicable throughout most of the districts. They are as follows:

1. Record of sign maintenance performed—General maintenance functions such as washing, clear coating, post painting, and patching with the date that work was performed could be recorded for each sign.
2. Record of sign replacement—Reasons for sign replacement fell into two categories, emergency and maintenance, which were recorded in 50 percent of the district inventories.
3. Aid in field inspections—The inventory was used to record dates conducted and signing observed for field inspections in all eight districts.
4. Verification of sign in place—In all districts but one, inventories had the capability of determining that a given sign was in place at the date of the field inspection. Signing records generally included sign message or standard Virginia signing code and location by route and milepost.
5. Accident investigation—Five of the eight district traffic engineers felt that sufficient information was available in their inventories to be useful as legal evidence related to accident causation.
6. Routine check of work performed—In some districts, much of the maintenance is performed in the residencies within the district. The inventory proved to be a convenient way to conduct spot checks to determine whether work was being performed.

Publication of this paper sponsored by Committee on Maintenance of Traffic Control Devices.

7. Check service life of materials—A secondary use of the inventory in some districts was to examine sign replacement frequency as a means of evaluating certain signing materials.

MAINTENANCE OF INVENTORY SYSTEMS

Inasmuch as inventories are maintained at the district level, the district traffic engineer generally has the responsibility for maintaining and revising the signing inventory procedure. In isolated instances, the residency offices keep maintenance logs, and, in one district, inventory cards are circulated between the district office and the appropriate residencies for purposes of work assignment and annotations.

The district traffic engineer's assessments of time and effort required to maintain the inventory procedures varied considerably. Some felt that maintenance of the inventory encroached negligibly on the work load of their technicians, and others felt that the effective maintenance of an inventory procedure would require the full-time services of a technician.

In conclusion, it should be noted that the survey of eight districts revealed eight distinct signing inventory systems. In each case, the district traffic engineer felt that his system was adequate and that no changes were needed.

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