HIGHWAY RESEARCH RECORD

Number 211

Aspects of Traffic Control Devices

4 Reports

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Foreword

Users of highways are advised by traffic control devices as to conditions or requirements that affect highway use at specific times and places in order that the proper action may be taken. Because of the interaction effects between the device and the respondent, much research has been conducted on the influence of devices upon users. The four papers in this RECORD are predominantly concerned with traffic control devices and indicate how complex problems with devices can be.

Highway and traffic engineers at all levels of government will find much to interest them in this RECORD. Researchers of a mathematical bent will find some of the papers to be oriented towards their field of endeavor.

The first paper by a Canadian researcher utilizing a queuing model presents a derivation of volume warrants and design charts for left-turn storage lanes at unsignalized grade intersections on four-lane and two-lane highways. The research was based on study of seven unsignalized intersections and fully tested by application to 80 intersections. This paper is an excellent example of the teaming of theory and data to produce a usable practical result.

A Yale University engineer has developed a computer program for better design of time-space diagrams for proper timing of traffic signals. The computer calculates the timing plan with the greatest efficiency from the several variables imposed in time-space diagrams. Of paramount importance is the availability of the program (written in FORTRAN language) and the fact it can be run on almost any digital computer.

A traffic consultant also deals with computers. Using "loop" detectors placed at intersections, the measured parameters of volume, speed, density and space and time headways are used with a computer to find queue length and delay at a signalized intersection.

Finally, a New York researcher presents a methodology for generating an objective, reliable and practical traffic operations measure. It uses four driver satisfaction factors: travel time, driver discomfort, driving hazards and vehicle running costs. The relationships with volume were especially significant. The paper presents a set of general prediction curves by which the travel time-volume relationship can be estimated from knowledge of the characteristics of specific streets.

Contents

Volume Warrants for Left-Turn Storage Lanes At Unsignalized Grade Intersections

M. D. HARMELINK, Project Research Engineer (Traffic), Department of Highways, Ontario

This paper describes the derivation of volume warrants and design charts for left-turn storage lanes at unsignalized grade intersections on four-lane and two-lane highways. The design charts are based on a theoretical analysis and on a series of field studies of traffic behavior at intersections.

The analysis is based on a queuing model in which arrival and service times are assumed to follow a negative exponential distribution. The arrival rates are determined by the volumes of left-turning, through or "advancing," and opposing traffic, and by the time interval required by the left-turning vehicle to clear the advancing lane. The service rates are determined by the volume of opposing traffic, and by the time interval required to make a left-turn maneuver.

Field studies of traffic behavior conducted at seven unsignalized Ontario intersections provided average values of the time interval required by a left-turning vehicle to make a left turn and to clear the advancing lane, the delay experienced by a left-turning vehicle because of opposing traffic, gap acceptance and rejection behavior, and actual arrival rates and headway distributions at various volume levels.

•THIS study was undertaken because of the lack of consistent volume warrants for left-turn storage lanes at unsignalized intersections. The usual method of analyzing such intersections individually on the basis of past experience, accident records, complaints from the traveling public, and engineering judgment has led to inconsistency from location to location.

It was felt that the volume warrants developed should be consistent in their evaluation of traffic parameters from location to location; they should provide reasonable recommendations for specific intersections; and they should be based on traffic and operational considerations, rather than on a benefit-cost analysis, because of the difficulty of translating the benefits received to a monetary value on a suitable rational basis.

The study contained three phases: a theoretical analysis, a series of field studies of traffic behavior, and analysis of a series of questionnaires completed for specific intersections by Department of Highways regional traffic engineers.

THEORETICAL ANALYSIS

Queuing theory may be used to analyze operational flow problems where the state of the system changes from time to time and which have elements that follow this basic behavior: A sequence of units arrives at some facility which services each unit and eventually discharges it (1). In our problem, a sequence of left-turning vehicles arrives at some intersection that permits each left-turning vehicle to proceed if and when there is a suitable gap in the opposing traffic stream, and then discharges the vehicle from the intersection. Morse (1) explains that instead of trying to predict in detail how the state of the system changes with time, we can calculate the probabilities that the system is in each of the possible states.

Paper sponsored by Committee on Traffic Control Devices.

In this analysis it was assumed that the arrivals of left-turning vehicles follow a Poisson distribution (random or negative exponential distribution) and that the service time distribution is also negative exponential, i.e., the probability of prolongation of service is independent of how long ago the service started. It may be shown (1) that the state of such a service system is dependent only upon the average arrival rate λ, and the average service rate μ, and the state probabilities are independent of time. The ratio $\rho = \lambda/\mu$ is called the "utilization factor"; steady-state solutions may be determined only if $\rho < 1$.

It may further be shown (1) that for the steady-state system with negative exponential arrival-time and service-time distributions, and where every arriving unit joins the queue,

$$P_n = (1 - \rho)\rho^n \tag{1}$$

and

$$Q_n = \rho^n \tag{2}$$

where P_n is the probability of n units in the system (both queue and service) and Q_n is the probability of n or more units in the system (queue plus service). The volume warrants were based on these two relationships.

The derivation of the warrant was based on the following conditions:

1. On four-lane highways, it is the presence of a left-turning vehicle extending into the through lanes that will affect safety and capacity; the probability of this occurrence should not exceed 0.005 for divided highways or 0.03 for undivided highways. (Divided highways are those with sufficient median width for the storage of at least one left-turning vehicle; undivided highways are those with less or no median width.) It is assumed that there is sufficient through or "advancing" traffic for such an occurrence to be undesirable.

On two-lane highways, it is the arrival of advancing, through vehicles behind a stopped left-turning vehicle that will affect safety and capacity (an arriving through vehicle is one that has been stopped or brought to creep speed by a left-turning vehicle in the advancing lane); the probability of this occurrence should not exceed 0.020 for design speed = 50 mph, operating speed (v) = 40 mph; 0.015 for design speed = 60 mph, operating speed (v) = 50 mph; and 0.010 for design speed = 70 mph, operating speed (v) = 60 mph.

These probability levels were determined from preliminary investigations, from the judgments of various highway department engineers and, particularly in the case of two-lane highways, from highway capacity considerations.

2. Both arrival-time and service-time distributions are negative exponential.

3. There is strict queue discipline: "first come, first served," and every arriving unit must join the queue.

4. No left-turning vehicle can begin its maneuver until the previous vehicle has completed its left turn.

5. On four-lane highways, the average time t_l required for making a left turn is 4.0 sec. On two-lane highways, t_l is 3.0 sec. These values were determined from field studies.

6. The required critical headway G_c in the opposing traffic stream for a left-turn maneuver is 6.0 sec on four-lane highways and 5.0 sec on two-lane highways. These values were determined from field studies.

7. On a two-lane highway, the average time t_e required for a left-turning vehicle to clear itself or "exit" from the advancing lane is 1.9 sec, as determined from field studies.

For four-lane highways, the average arrival rate is

$$\lambda = V_L \tag{3}$$

where V_L is the number of vehicles per hour making left turns.

For two-lane highways, the problem situation is that in which a left-turning vehicle is followed by a through vehicle. From the theorem of compound probability, the probability of this occurrence is L(1 - L), where L is the proportion of left turns in the total advancing traffic stream of through and left-turning vehicles. This is not the probability of an arrival, however, for the left-turning vehicle may have completed its turn before the following through vehicle arrives. Whether or not the following vehicle arrives before the turn is completed largely depends on three factors:

1. The average time t_w that a left-turning vehicle must wait for a suitable gap in the opposing traffic stream (t_w expressed in seconds). As developed by Adams (2):

$$t_w = \frac{3600}{V_o e^{-\frac{V_o G_c}{3600}}} - \frac{3600}{V_o} - G_c \tag{4}$$

2. The time interval t_e defined earlier; t_e = 1.9 sec.
3. The median time interval (headway) between vehicles in the advancing stream. The median headway was selected rather than the mean headway because of the high frequency of headways less than the mean. From theoretical considerations and field tests (3):

$$t_{median} = \frac{2}{3} t_A = \frac{2}{3} \cdot \frac{3600}{V_A} = \frac{2400}{V_A} \tag{5}$$

where V_A = advancing volume (through, left-turning, and right-turning vehicles, vph), and t_A = mean headway in V_A.

On two-lane highways, then, the mean arrival rate is the number of arrivals per hour of through vehicles behind left-turning vehicles:

$$\lambda = [L(1 - L) V_A] \frac{t_w + t_e}{\frac{2}{3} t_A} \tag{6}$$

where $L = V_L/V_A$ as defined earlier.

For both four-lane and two-lane highways, the average service rate μ is the number of left turns that can be made in one hour. This parameter is a function of:

1. The volume of traffic in the opposing lane (s) V_o, and hence the amount of time per hour in which left turns can be made (unblocked time). The amount of unblocked time per hour during which left turns are possible may be computed by deducting from the total time (a) all time in the opposing stream composed of headways less than G_c, and (b) a certain proportion of the time when the left-turning vehicle is less than G_c sec from an oncoming vehicle in those opposing stream headways greater than G_c (4). Because of the random traffic arrivals and the possibility of a vehicle arriving at a time anywhere in the blocked period of less than G_c sec, this proportion was taken to be 0.5, i.e., the average amount of blocked time in a usable gap is $G_c/2$. The amount of unblocked time may then be determined from graphs of observed headways for various volume conditions on both four-lane and two-lane highways. The graphs used in this analysis, which agreed well with observed distributions, are shown in Figures 7 to 9 (4).
2. The average time t_1 taken to make the left-turn maneuver.

The mean service rate

$$\mu = \frac{\text{Unblocked Time/Hr (sec)}}{t_1 \quad \text{(sec)}} \tag{7}$$

For four-lane divided highways, the warranting traffic volumes were determined from Eq. 1 and the probability level of 0.005. Thus,

$$P_0 + P_1 \nless 0.995$$

Now

$$P_0 = (1 - \rho)\ 1 = 1 - \rho$$
$$P_1 = (1 - \rho)\ \rho = \rho - \rho^2$$

Adding,

$$P_0 + P_1 = 1 - \rho^2 \nless 0.995$$
$$\rho^2 \nless 0.005$$
$$\rho \nless 0.0707$$

For a left-turn storage lane to be warranted, $\rho \geq 0.0707$. For any opposing traffic volume V_{0i} and its corresponding service rate μ_i,

$$\lambda_i = \rho\mu_i = 0.0707\mu_i$$

The warrant curve is shown in Figure 1 (all figures are in the Appendix). Volume conditions above and to the right of the curve warrant a left-turn storage lane.

Along the warrant curve itself, $Q_2 = \rho^2 = 0.005$ (from Eq. 2) and the probability of 3 or more units in the system, $Q_3 = \rho^3 = 0.000354$. If $\rho^3 > 0.000354$, a longer storage length, S = 75 ft, is required (assuming 25 ft per vehicle). Extending this principle, for any storage length S, with storage capacity of n vehicles, the probability of n + 1 or more vehicles in the system should not exceed 0.000354. Thus, the probability of exceeding the capacity of the storage lane by one or more vehicles will be the same for each length. The design charts shown in Figure 1 were constructed on this basis. For example: S = 75 ft, n = 3, $Q_4 = \rho^4 = 0.000354$, $\rho = 0.137$, $V_L = \lambda = 0.137\,\mu$.

For four-lane undivided highways:

$$P_0 = 1 - \rho \nless 0.970$$
$$\rho \nless 0.30$$

A left-turn storage lane is warranted when $\rho \geq 0.030$. The warrant curve $\lambda = 0.030\mu$ is shown in Figure 1. For $V_0 < 400$ vph, no left-turn lane should be provided unless $V_A > 400$ vph because of the advancing vehicles' freedom to maneuver at lower volumes. For this reason, the four-lane undivided highway curve is shown as a dashed line for $V_0 < 400$ vph. Once volume conditions reach a point above and to the right of the warrant curve for the divided case, the undivided and divided cases are in effect the same, and the required storage lengths shown apply to both.

For two-lane highways:

$$P_0 = 1 - \rho \nless 0.980 \ (v = 40 \text{ mph})$$
$$\nless 0.985 \ (v = 50 \text{ mph})$$
$$\nless 0.990 \ (v = 60 \text{ mph})$$

Therefore, $\rho \nless 0.020$, 0.015, and 0.010 for v = 40, 50 and 60 mph respectively. The warrant curves were determined for each speed condition from $\lambda = \rho\mu$; for a given V_0, the warranting V_A for a given left-turn proportion and operating speed was determined from:

$$\lambda = [L(1 - L)V_A] = \frac{t_w + t_e}{\frac{2}{3} \cdot t_A} = \rho\mu$$

TABLE 1

STORAGE LENGTH TO BE ADDED TO CHART VALUES OF LEFT-TURN LANE STORAGE LENGTHS (Length in Feet)

CHART VALUE OF STORAGE LENGTH REQUIRED	% T_L = % TRUCKS IN V_L					
	0%	10%	20%	30%	40%	50%
75'	0	25'	25'	25'	50'	50'
100'	0	25'	25'	50'	50'	50'
125'	0	25'	25'	50'	50'	75'
150'	0	25'	50'	50'	75'	75'
175'	0	25'	50'	75'	75'	100'
200'	0	25'	50'	75'	100'	100'
250'	0	25'	50'	75'	100'	125'
300'	0	50'	75'	100'	125'	150'
350'	0	50'	75'	125'	150'	175'
400'	0	50'	100'	125'	175'	200'
450'	0	50'	100'	150'	200'	225'
500'	0	50'	100'	150'	200'	250'

Therefore

$$[L(1 - L)V_A^2]\,(t_W + t_e) = 2400\rho\mu$$

$$V_A^2 = \frac{2400\rho}{L(1 - L)} \cdot \frac{\mu}{(t_W + t_e)}$$

The curves are shown in Figures 2 through 19. The storage length S of the lane was determined using the same principles and methods as for four-lane highways: for any storage length S with storage capacity of n vehicles, the probability of n + 1 or more units in the system should not exceed 0.000008, 0.000003375, or 0.000001 for v = 40, 50, or 60 mph respectively.

Assuming an average required storage length per truck of 50 ft (5), the additional storage length required because of trucks may be determined from Table 1.

Figure 20 shows a typical design of left-turn storage lanes on two-lane Ontario highways. The design distances also apply to four-lane highways, except that the advancing lanes need not be tapered out to provide a shield for left-turning vehicles on divided highways because of the median.

FIELD STUDIES

Field studies were conducted at seven Ontario intersections to determine the values of parameters used in the analysis (3). The parameters measured were:

1. The average time interval t_e required for a left-turning vehicle on a two-lane highway to "exit" from the advancing lane (the lane from which the left-turn is made). A sample of 150 measurements gave t_e = 1.9 sec.
2. The average time interval t_l required to make the left-turn maneuver. For two-lane highways, t_l was found to be 3.0 sec. For four-lane highways, t_l was taken to be 4.0 sec, assuming that 1.0 additional sec is required to cross the additional lane.

TABLE 2

COMPARISON BETWEEN THEORETICAL AND OBSERVED ARRIVAL RATES
TWO-LANE HIGHWAYS

Intersection	V_L (vph)	V_A (vph)	$L = \frac{V_L}{V_A}$	V_o (vph)	t_w (sec.)	t_e (sec.)	t_a (sec.)	Theoretical $\lambda = [L(1-L)V_A] \frac{t_w + t_e}{\frac{2}{3} t_a}$ (vph)	Observed λ (vph)
Hwy. No. 7 and 2nd Line E.	75 88	580 600	0.130 0.147	628 658	3.00 3.10	1.9 1.9	6.21 6.00	78 96	80 87
Hwy. No. 2 and Altona Road	60	484	0.124	218	0.80	1.9	7.44	29	13
Keele St. and York Univ. Ent.	112	451	0.248	551	2.53	1.9	7.98	70	74
Hwy. No. 2 and Liverpool Road	133	271	0.491	220	0.84	1.9	13.30	21	16
Hwy. No. 2 and Brock Road	108	261	0.414	171	0.60	1.9	13.80	17	14

3. The critical gap G_c, i.e., the size gap which has the property that the number of accepted gaps shorter than G_c, is the same as the number of maximum rejected gaps longer than G_c. From an analysis of accepted and rejected gaps, G_c was found to be 5.0 sec for two-lane highways and 6.0 sec for four-lane highways.

4. The average time interval t_w which a left-turning vehicle must wait for a suitable gap in the opposing traffic stream. In the analysis, t_w was computed by using Eq. 4. For given V_o conditions, the observed t_w values were compared with the theoretical. Sizable fluctuations occurred when t_w was averaged over short time periods such as 15 min, but the fluctuations were considerably smoothed out when t_w was averaged over a time period of 45 to 60 min (3). As shown in Figure 21, the results for two-lane roads show reasonable agreement with the theoretical curve. Although t_w was not used in the four-lane analysis, it is interesting to note that the results for four-lane roads show closer agreement with the theoretical two-lane curve than with the four-lane curve.

5. Volume counts.

6. λ, the number of through vehicles which arrived behind vehicles waiting to make left turns and were delayed by them (two-lane highways). The observed λ values were compared with the theoretical values computed from Eq. 6 (Table 2). The agreement between the theoretical and observed λ values is quite good, except for the intersection of Highway No. 2 and Altona Road where an unusually small number of left-turning vehicles was delayed.

The field studies were conducted by a crew of five, using only prepared forms, two synchronizable wristwatches with sweep second hands, and a stopwatch. Team A (an observer and recorder) noted and recorded the time of arrival at the intersection of each vehicle in the advancing lane, whether that vehicle was a left-turning vehicle, and the number of arriving through vehicles delayed by waiting left-turning vehicles. Team B noted and recorded the time of arrival at the observer station of each vehicle in the opposing lane. The fifth observer was stationed with Team A and, using the stopwatch, noted and recorded t_e and t_w for left-turning vehicles. Values of t_l were determined separately.

TABLE 3

JUDGMENTS OF REGIONAL TRAFFIC ENGINEERS ON SUITABILITY OF CHART APPLICATION

	No. of Cases
In Agreement with Chart Recommendation	67
Lane not Warranted by Chart (i. e., traffic volumes), but required because of poor visibility and/or accident record.	8
Lane not Warranted by Chart (i. e., traffic volumes), but required because of character of route	1
Not in Agreement with Chart Recommendation (in all cases, a lane was felt to be warranted although the chart did not indicate it).	4
Total	80

ANALYSIS OF QUESTIONNAIRES

As requested, highway department regional traffic engineers completed detailed questionnaires on chart application at 80 specific Ontario intersections covering a wide range of traffic conditions (3). The engineers were asked to supply the following information for each intersection: explicit intersection data; an evaluation of visibility, suitability of storage lane length (if in operation), sideroad traffic interference, and congestion conditions; the reason(s), if known, for construction of an existing left-turn lane; whether a storage lane was warranted by the charts; whether the rater considered the chart recommendation a reasonable one; and the reason for agreement or disagreement with the chart recommendation. Table 3 summarizes the questionnaire replies. These results were interpreted to mean that, in general, where traffic volume was the governing factor, the volume warrants and charts provided reasonable solutions.

It is recognized that intersections with poor visibility and/or a bad accident record may require the designer to exercise his judgment when volume conditions alone do not warrant a storage lane. It is also recognized that the analytical "models" could be considerably improved and refined. Nevertheless, it is believed that the charts, based on a theoretical analysis (although crude) as well as on field observation and the judgment of design and control engineers, provide a basis for design that is more consistent and reasonable than those previously used.

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Appendix

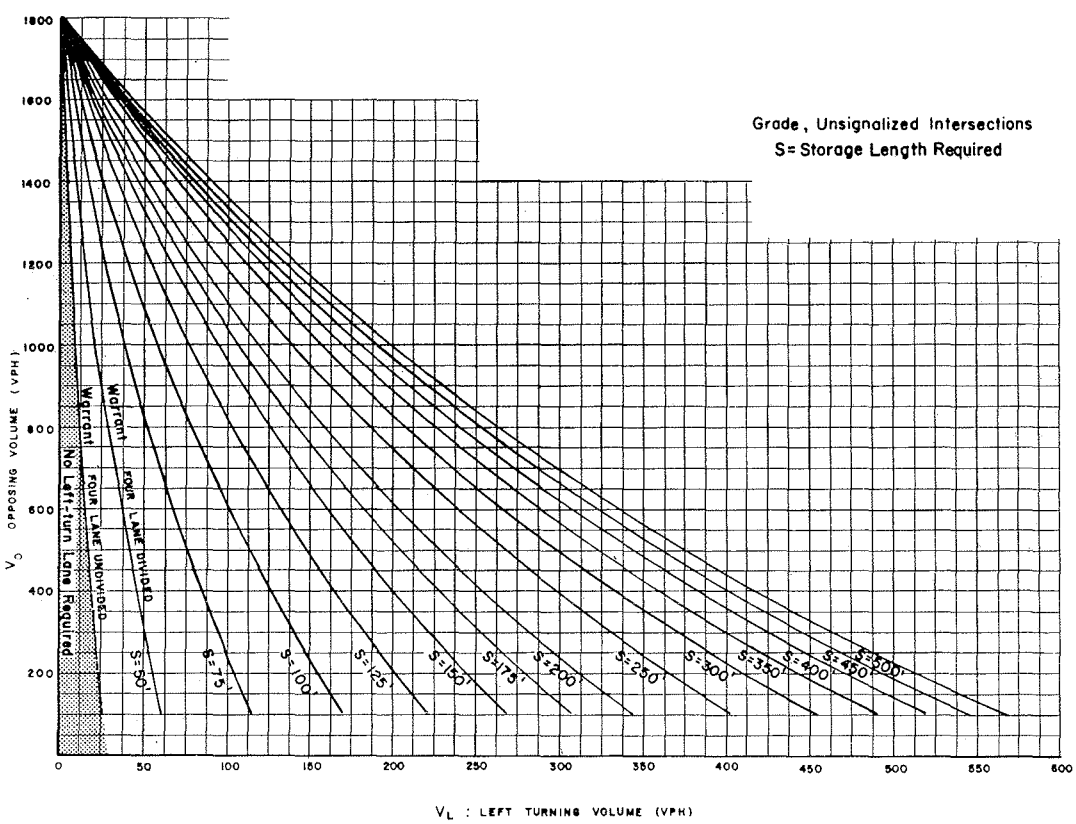

Figure 1. Warrant for left-turn storage lanes on four-lane highways.

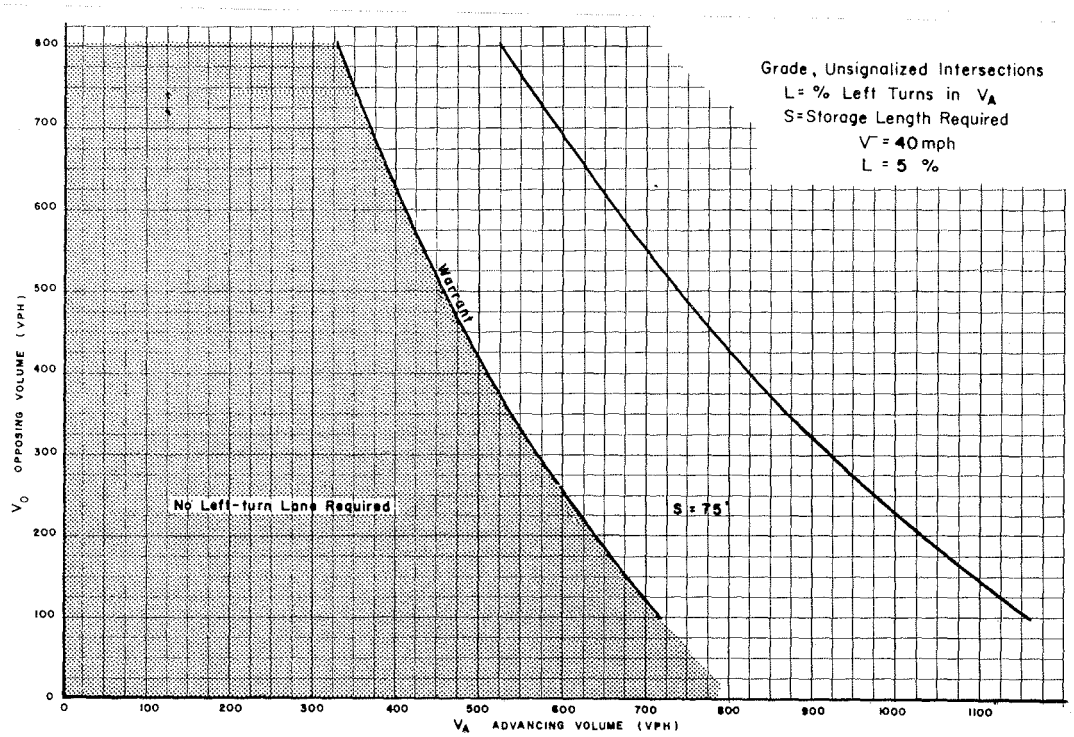

Figure 2. Warrant for left-turn storage lanes on two-lane highways.

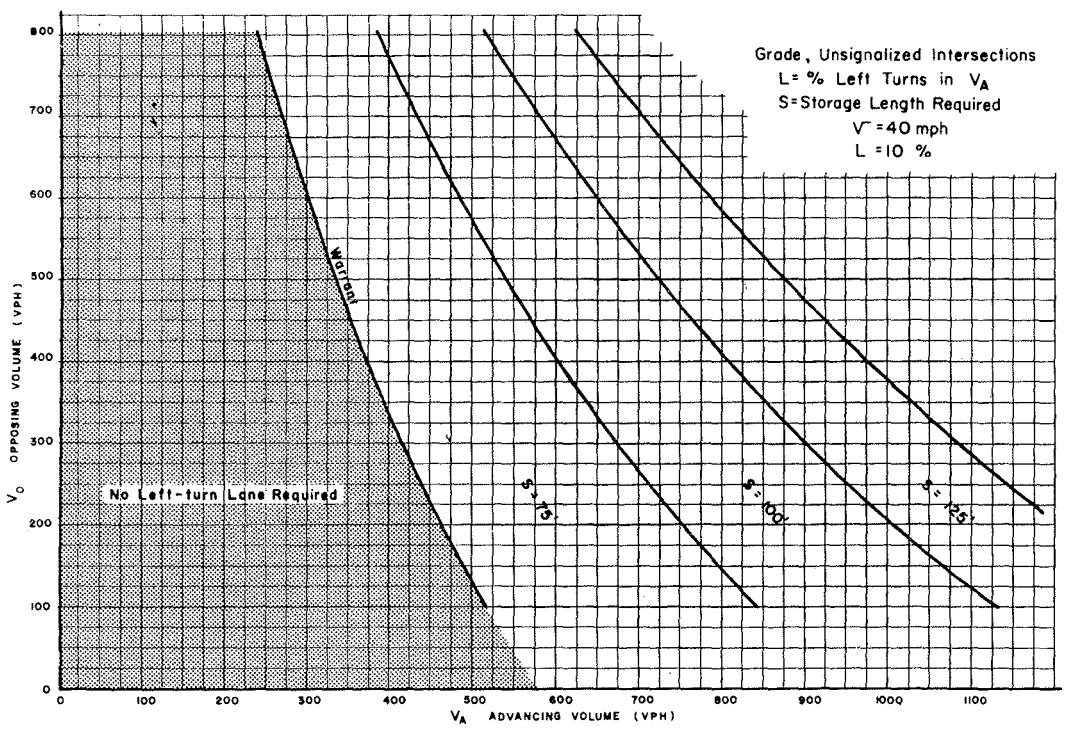

Figure 3. Warrant for left-turn storage lanes on two-lane highways.

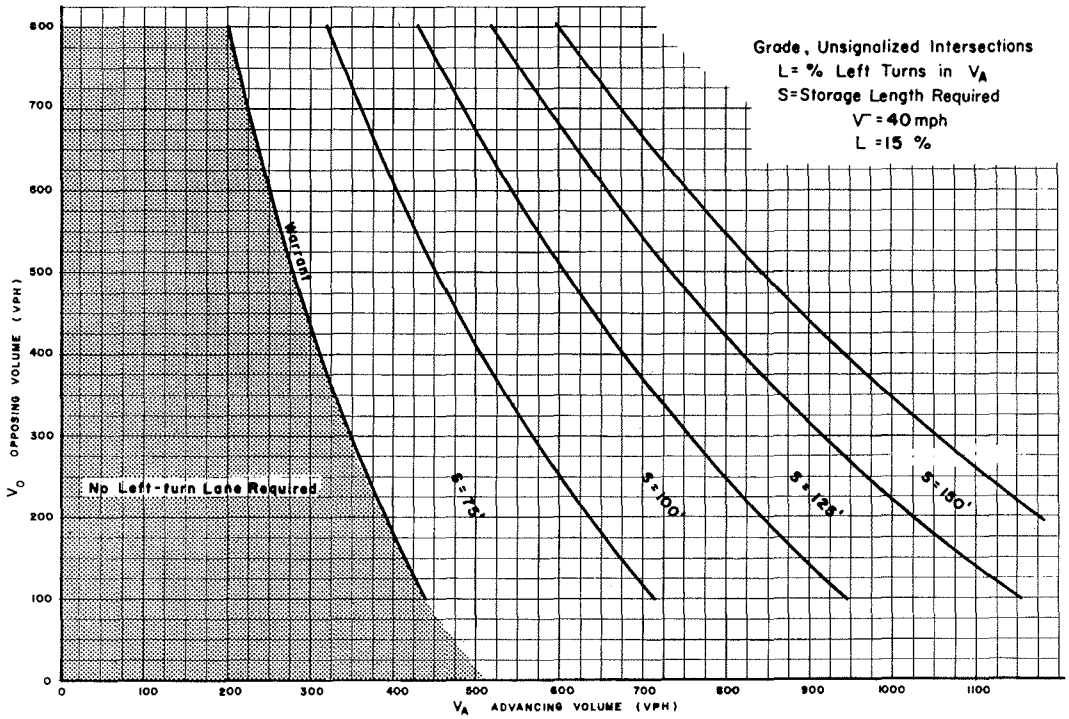

Figure 4. Warrant for left-turn storage lanes on two-lane highways.

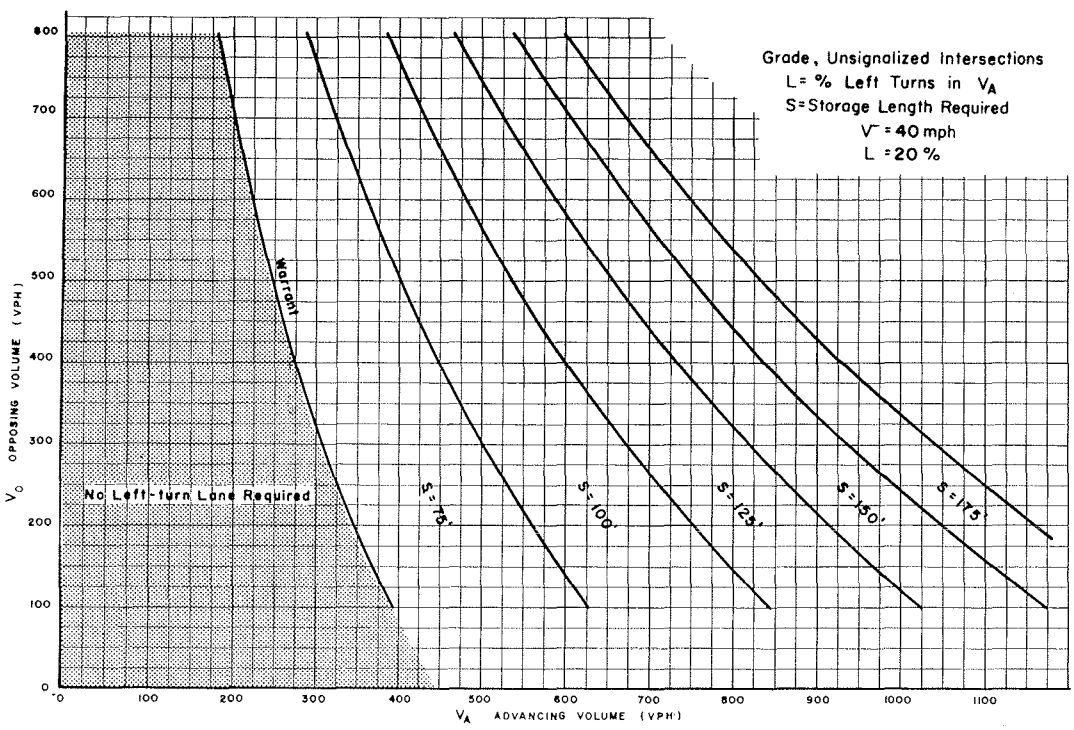

Figure 5. Warrant for left-turn storage lanes on two-lane highways.

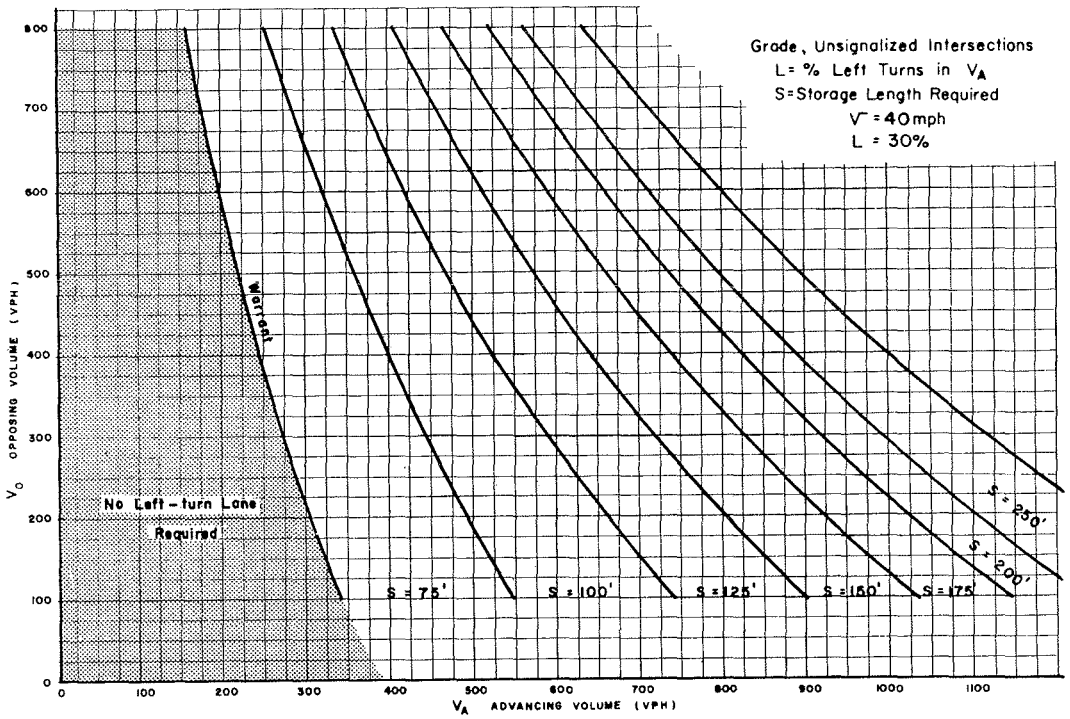

Figure 6. Warrant for left-turn storage lanes on two-lane highways.

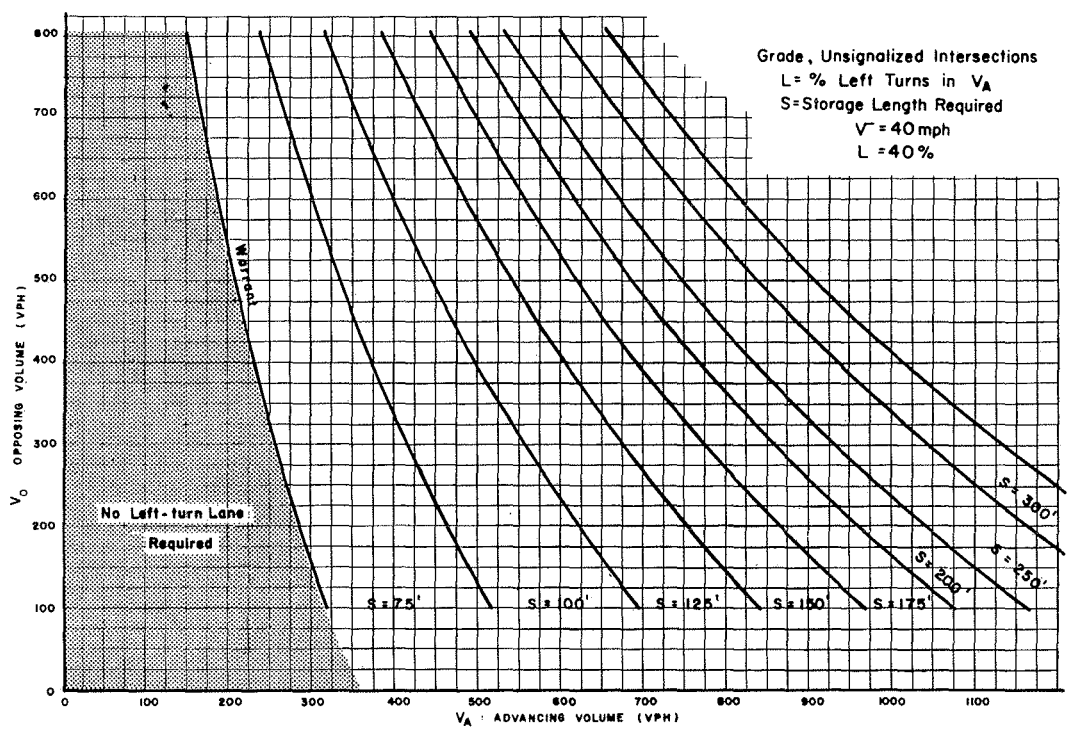

Figure 7. Warrant for left-turn storage lanes on two-lane highways.

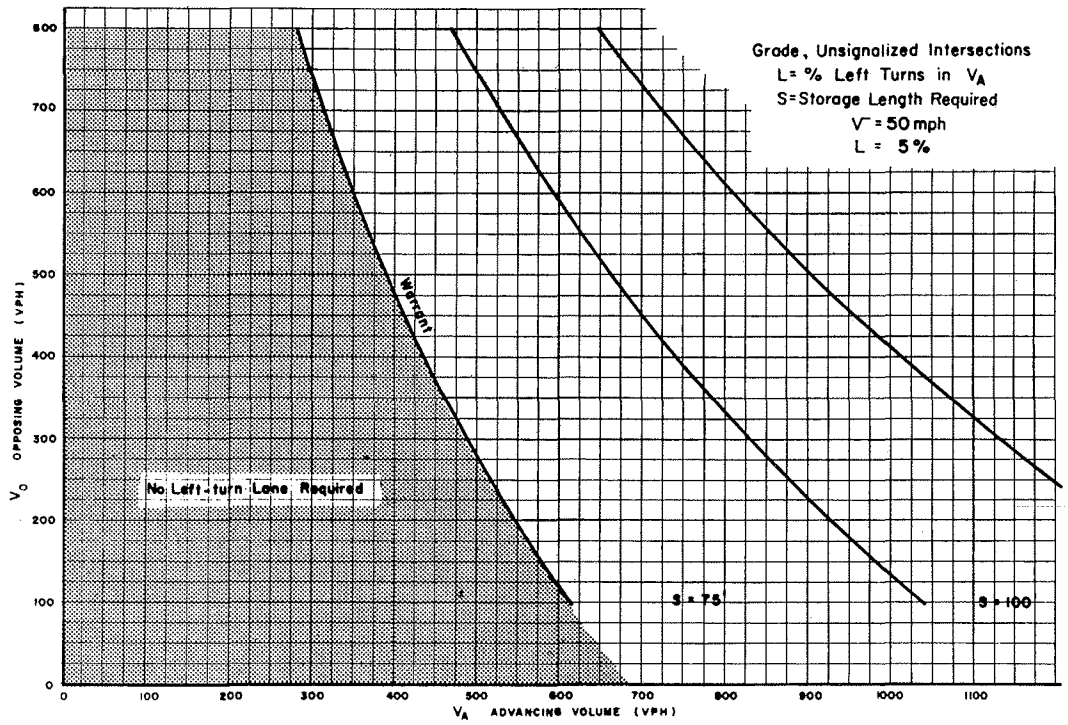

Figure 8. Warrant for left-turn storage lanes on two-lane highways.

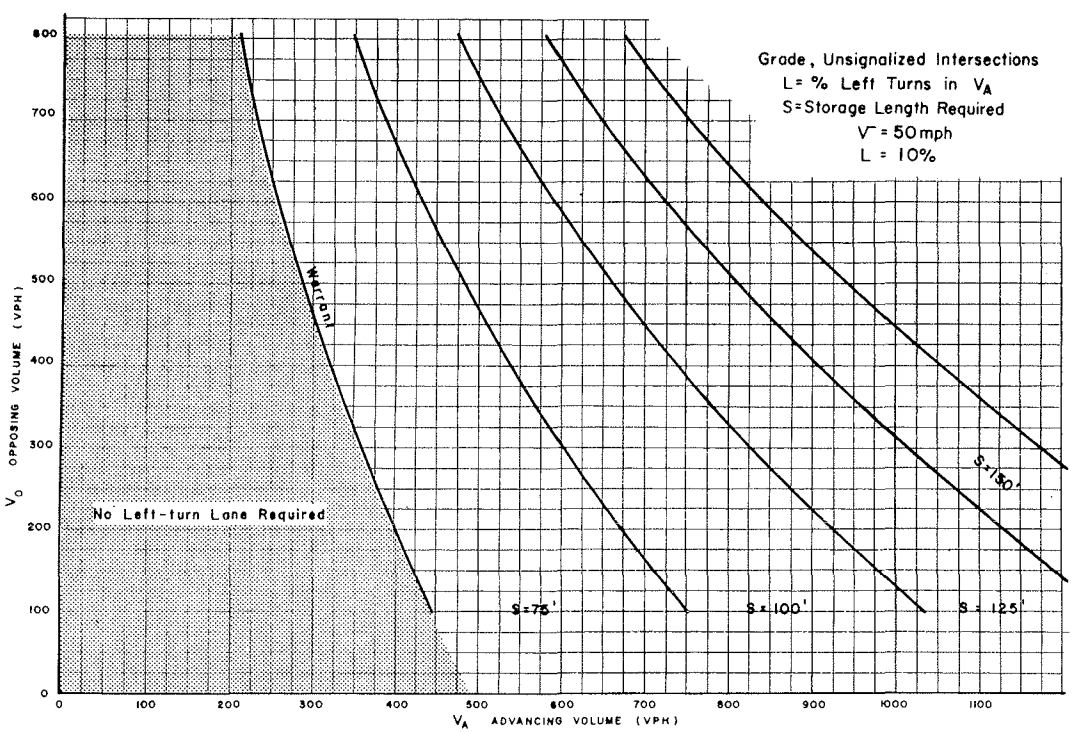

Figure 9. Warrant for left-turn storage lanes on two-lane highways.

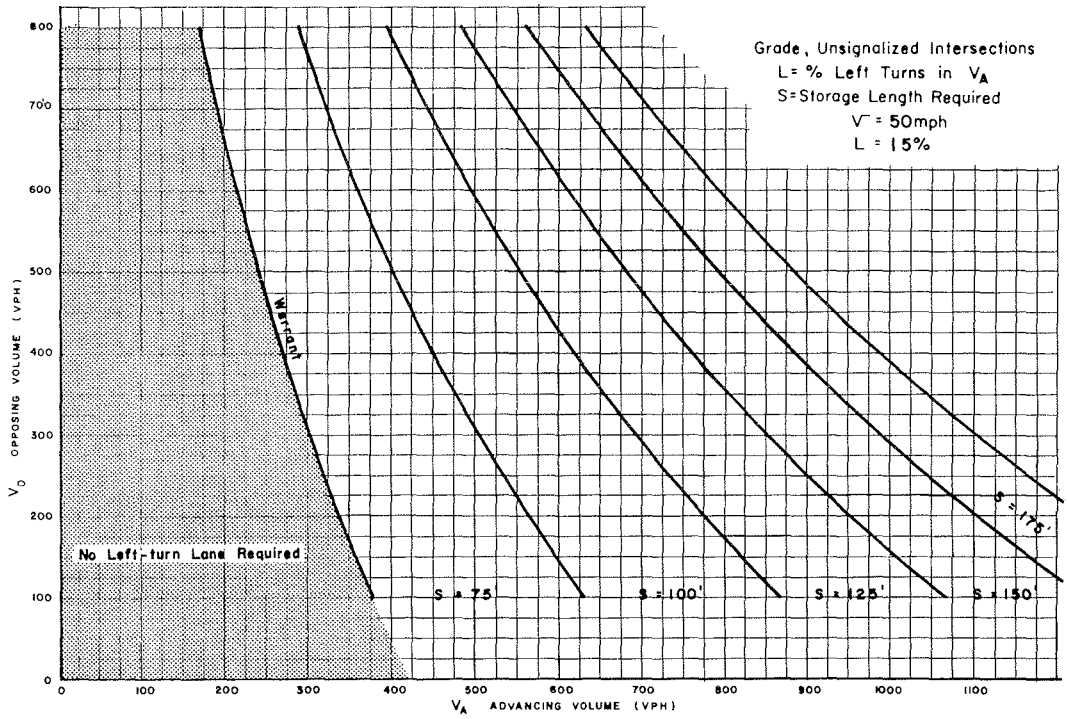

Figure 10. Warrant for left-turn storage lanes on two-lane highways.

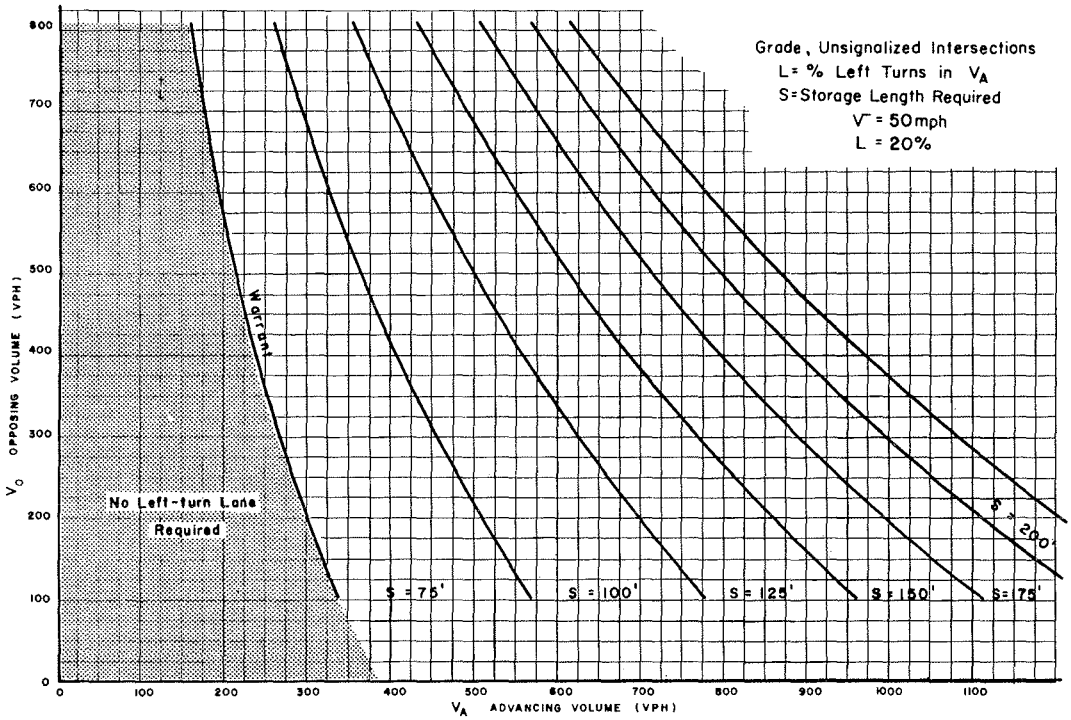

Figure 11. Warrant for left-turn storage lanes on two-lane highways.

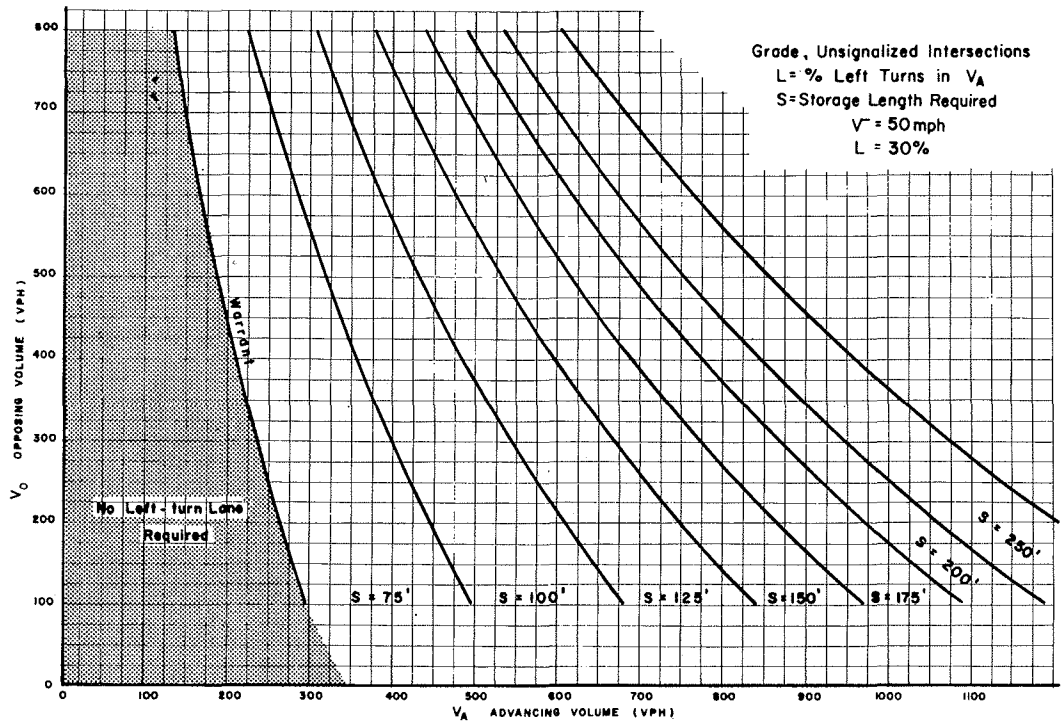

Figure 12. Warrant for left-turn storage lanes on two-lane highways.

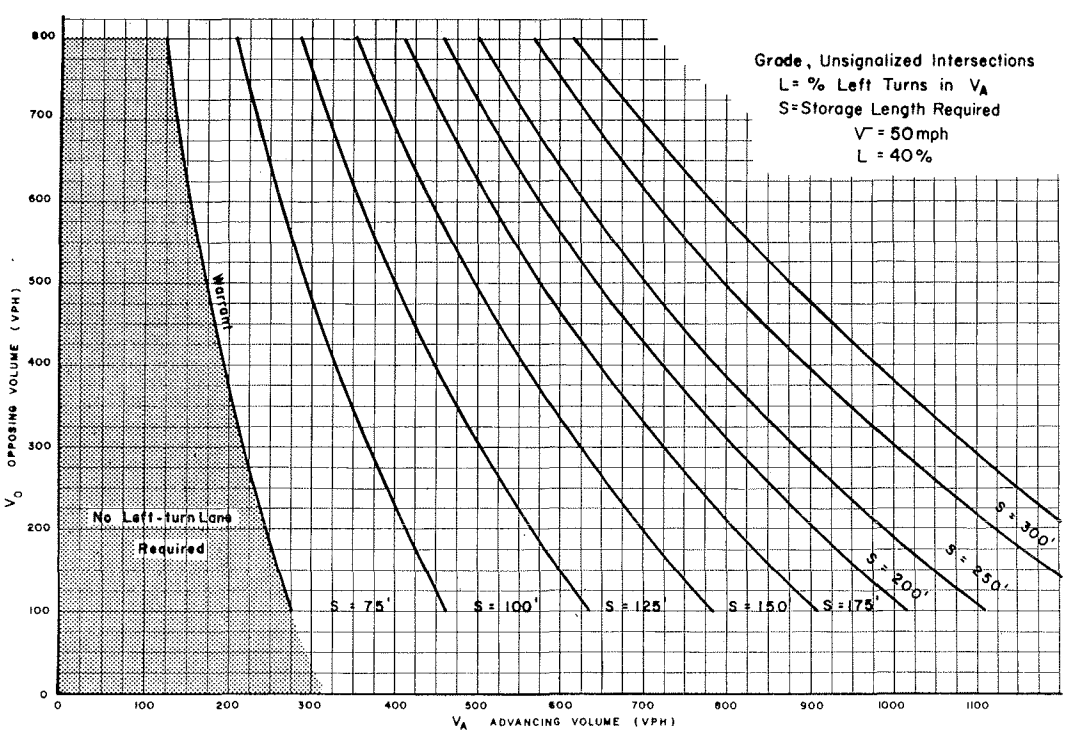

Figure 13. Warrant for left-turn storage lanes on two-lane highways.

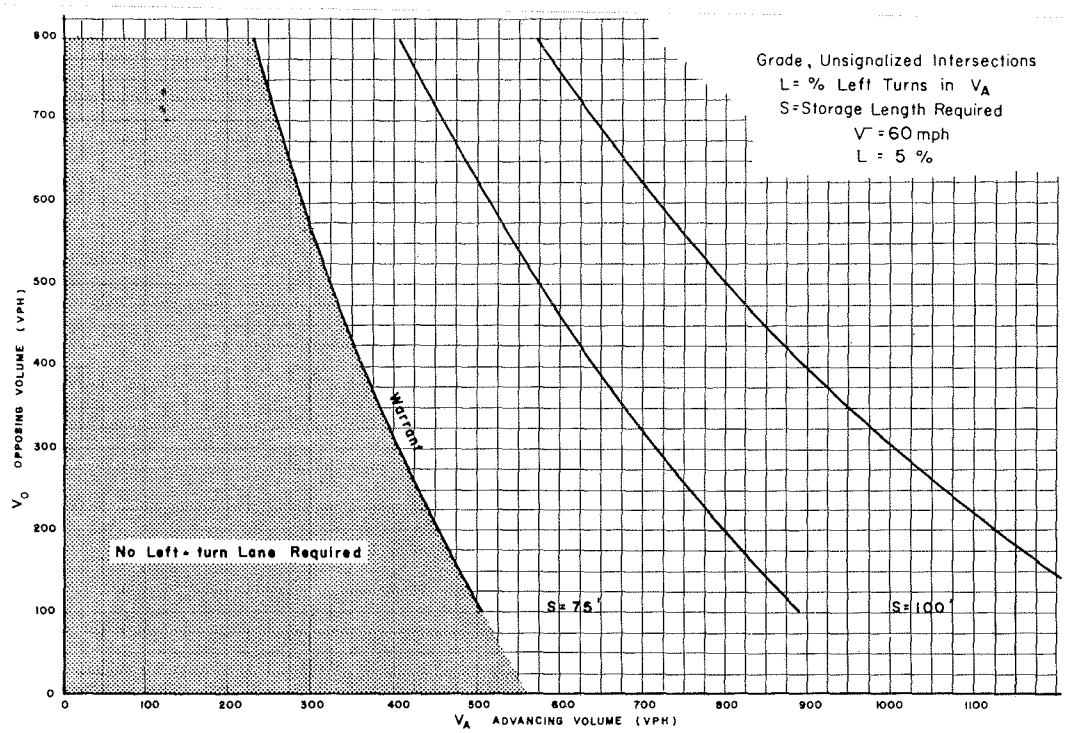

Figure 14. Warrant for left-turn storage lanes on two-lane highways.

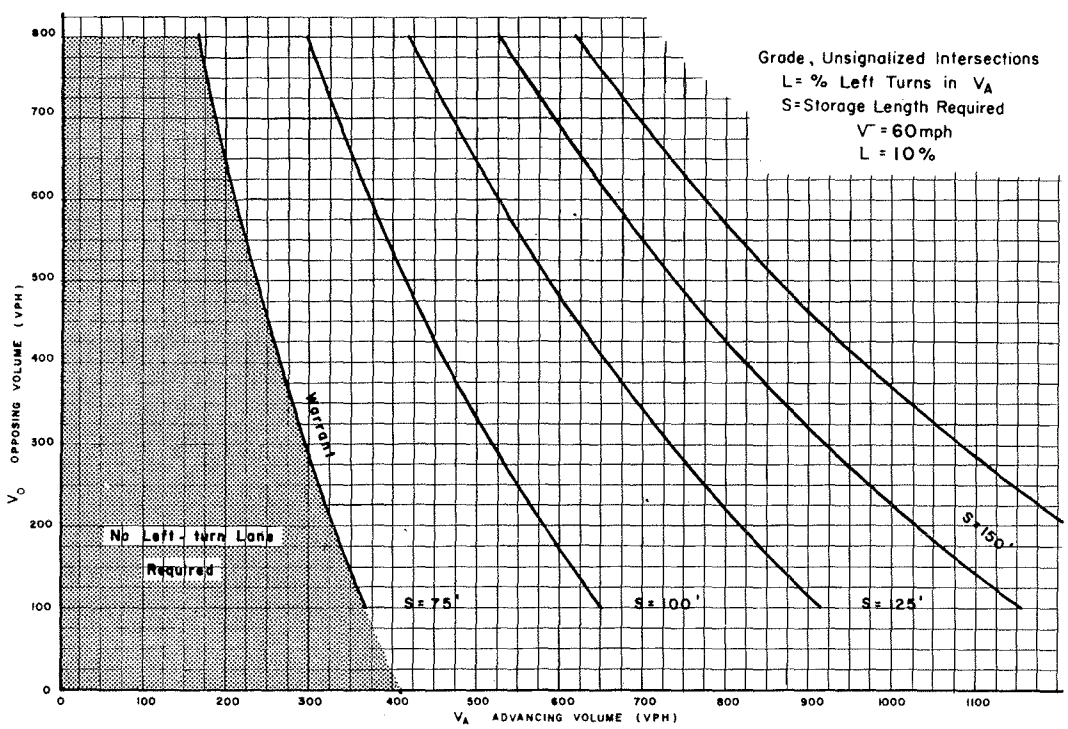

Figure 15. Warrant for left-turn storage lanes on two-lane highways.

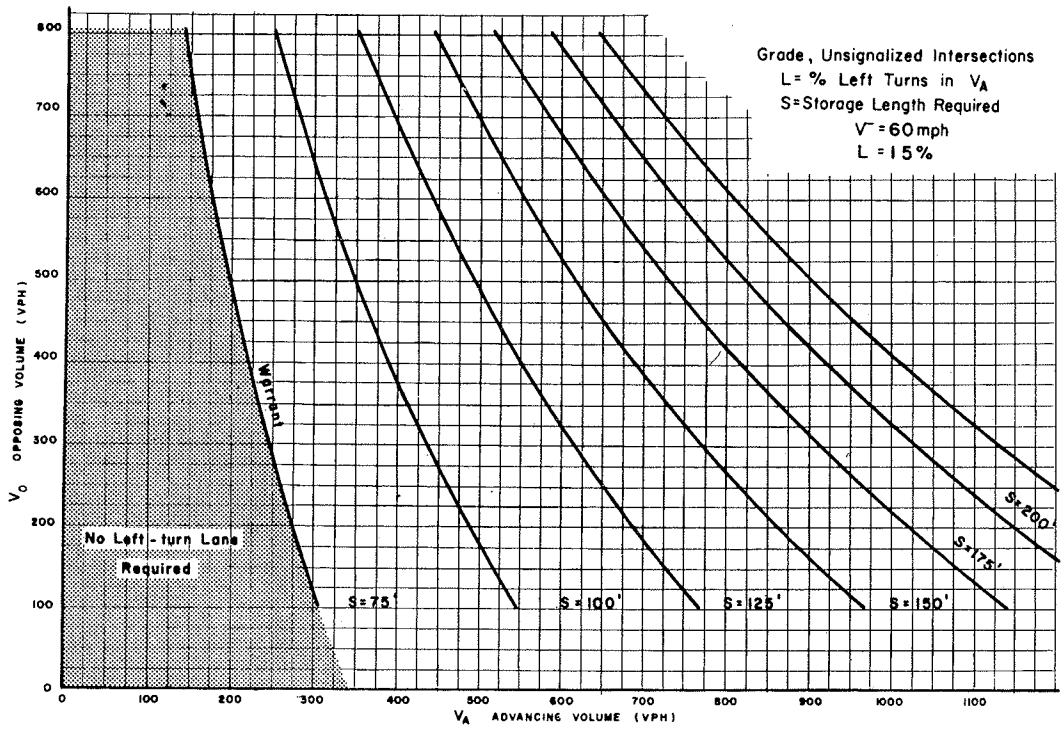

Figure 16. Warrant for left-turn storage lanes on two-lane highways.

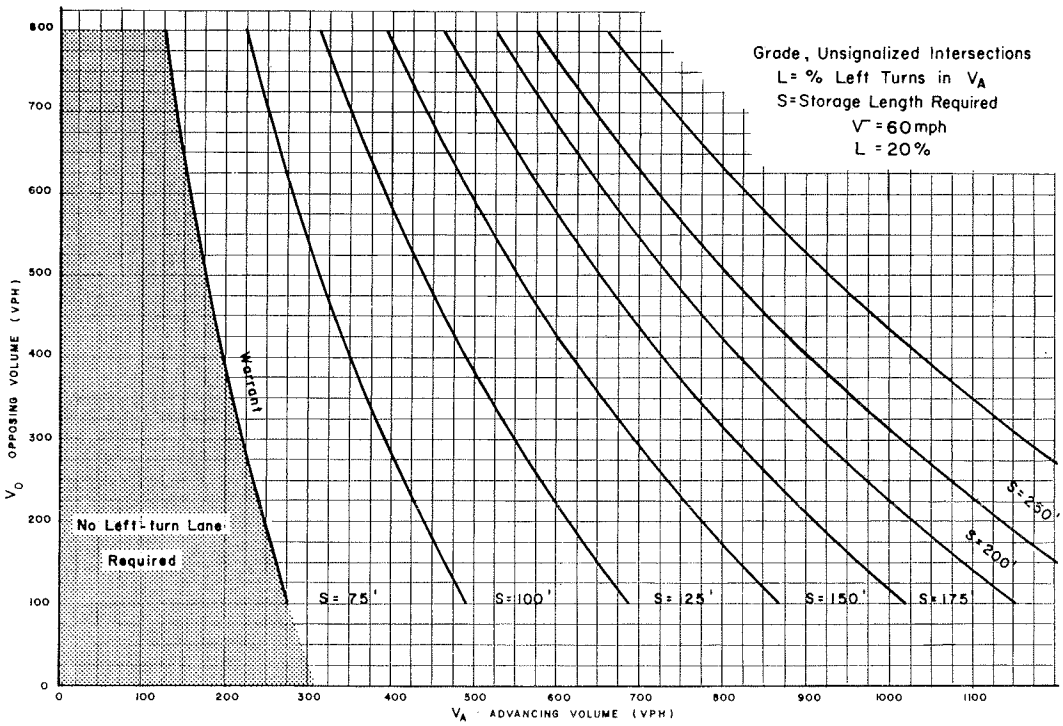

Figure 17. Warrant for left-turn storage lanes on two-lane highways.

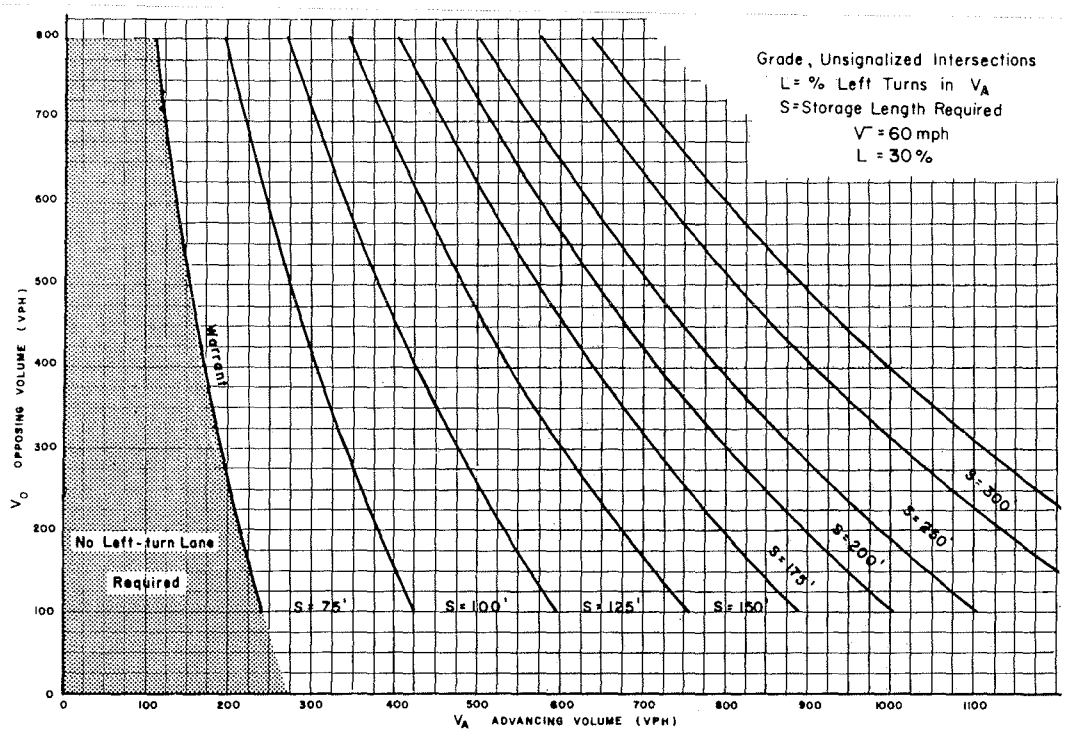

Figure 18. Warrant for left-turn storage lanes on two-lane highways.

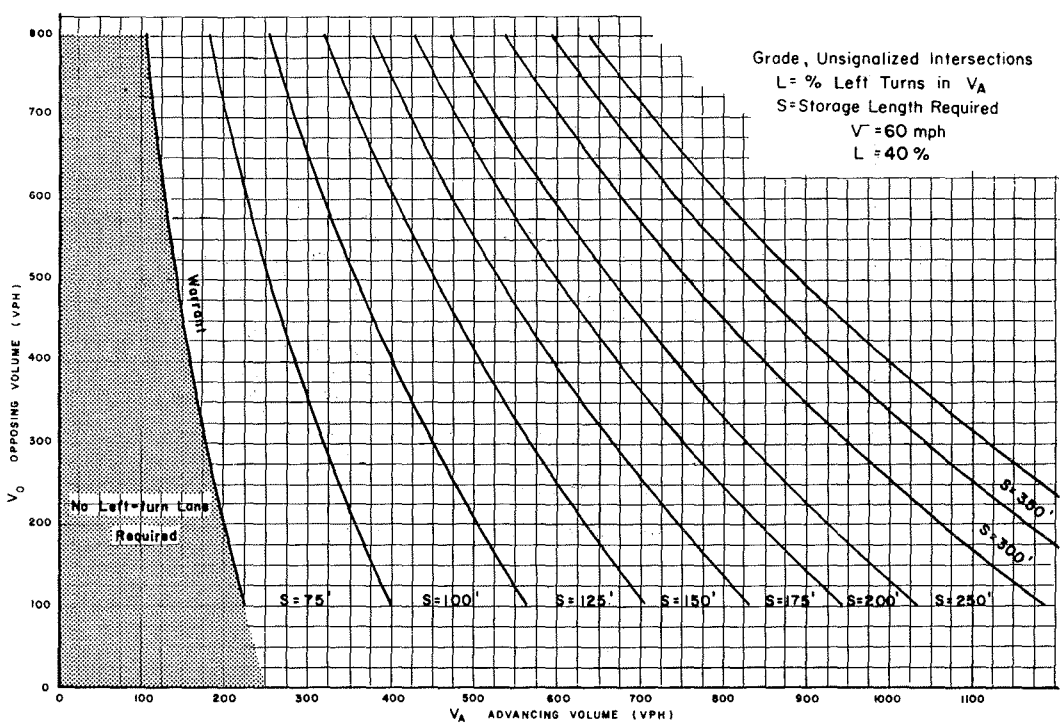

Figure 19. Warrant for left-turn storage lanes on two-lane highways.

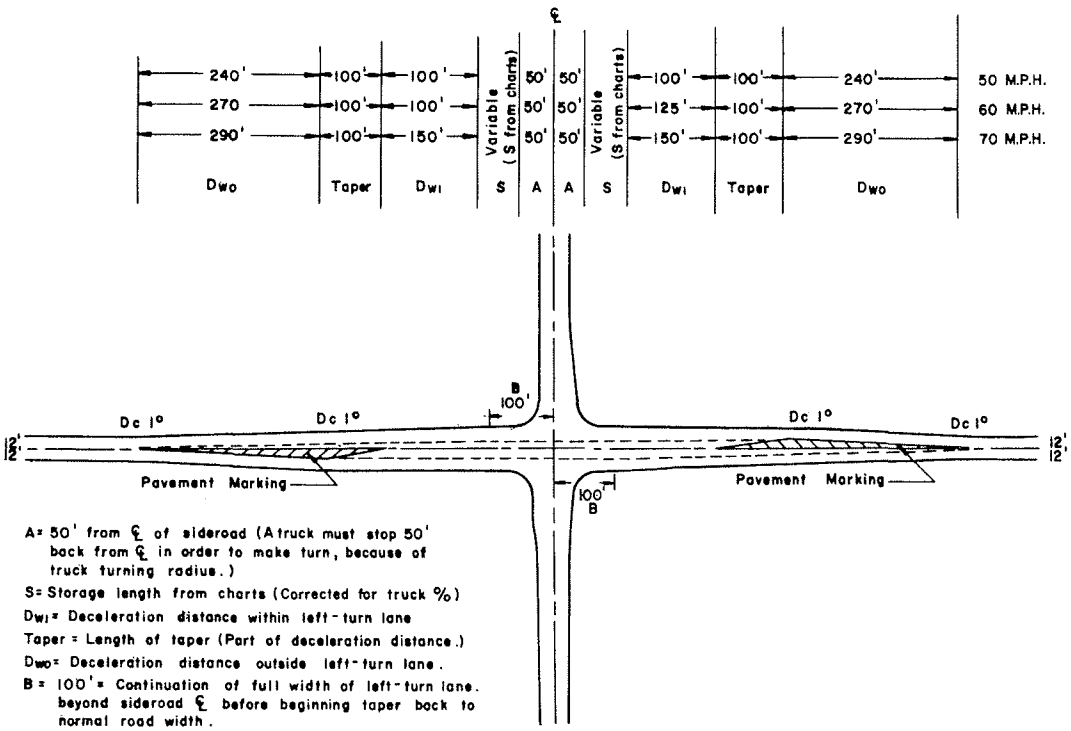

Figure 20. Cross-intersection with left-turn lanes of various design speeds (two-lane highway).

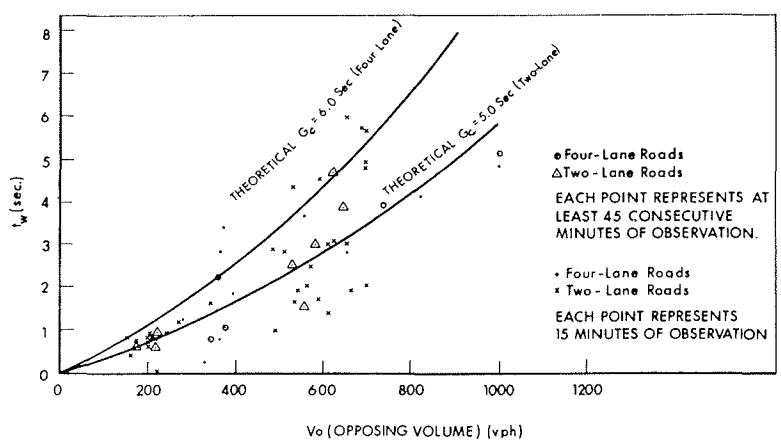

Figure 21. Relationship between t_w and V_o.

A Practical Computer Program for Designing Traffic-Signal-System Timing Plans

ROBERT L. BLEYL, Research Associate, Yale University, Bureau of Highway Traffic

This paper discusses the elements, techniques, and characteristics of a practical computer program developed for designing progressive traffic-signal-system timing plans. The elements discussed include directional travel distance variations, directional and sectional speed differences, band widths, offsets, cycle lengths, progressive speeds, and splits.

The computer program, written in FORTRAN IV programming language, converts all speed and distance units to travel time units. The timing plan resulting in the greatest efficiencies is then determined from a time-travel time diagram. The program favors the directional band widths in proportion to the desired relative band widths and prints a series of tables which indicate, from the ranges specified for the numerous variable elements, the optimum timing plan.

•IN recent years, various computer programs for designing progressive traffic-signal-system timing plans have been produced. A review of several available programs revealed a lack of certain desirable features which limit their value as practical programs. Accordingly, the development of a practical computer program for solving signal progression problems was undertaken. The program described in this paper is the outgrowth of that project.

A brief discussion of variables to be considered in determining an "optimum" timing plan is included first in this paper as an aid in understanding the problems associated with progressive traffic-flow plans. Since the techniques employed to handle the variables in the program differ from the conventional approach, they are discussed in detail. The computer program developed is described in the concluding section.

SIGNAL SYSTEM VARIABLES

It has generally been assumed that a traffic-signal system was constant throughout. Although this assumption facilitates the construction of a time-space diagram, it is unrealistic.

Virtually no signal system is homogeneous and consistent throughout. In addition to curves and grades, there are changes in roadway widths, adjacent land uses, and traffic conditions. These factors influence the movement of traffic and should be considered when attempting to time a system for progressive movement.

Signal Spacing

One axis of the traditional time-space diagram has always represented the centerline distance between signal installations. In progressive timing plans there are two signal-to-signal distances of more importance than the centerline distance between the signals. They are the stop line-to-stop line travel distances for each direction of travel. These two directional travel distances will differ when there are horizontal curves in the

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alignment, or when signalized cross streets are skewed, offset, or of differing widths. In some cases the differences in the directional travel distances are insignificant, but in others the differences are great enough to require offset adjustments in order to provide maximum efficiency. In any case, a practical program should provide for this contingency.

Travel Speeds

Many elements of the traffic-signal system influence the speeds at which vehicles travel. Included among these are the roadway widths and other cross-sectional elements, the use of the abutting property, and the desires, patterns, and volumes of traffic. Whenever these elements vary within a system, it seems logical to expect that the traffic speeds might also vary.

The old practice of timing a traffic-signal system for a single, constant speed throughout its entire length is far from reality. Traffic speed patterns may vary not only from one section of a system to another, but also by direction of travel at any point within the system. This natural variability in the speed patterns also should be considered in the design of progressive signal-system-timing plans.

PROGRESSIVE TIMING VARIABLES

There are five elements of the time-space diagram which may be variables: band width or efficiency, offsets, cycle length, progressive speeds, and splits or intervals.

Band Width

The primary objective in timing a progressive signal system is to determine the combination of signal-timing elements which will result in the largest band width for a given cycle length. The ratio of band width to cycle length is referred to as the "efficiency" of the system. In order for the "optimum" or best solution (the greatest efficiency) to be found, all four of the other signal-timing elements must be allowed to vary. The best solution is reached only when the optimum cycle length has been found, the appropriate splits used, the optimum progressive speeds determined, and the optimum offsets obtained. For practical application, the variability of all these elements must be held within acceptable ranges.

Offsets

The heart of the signal progression problem is the determination of the offsets which will yield the maximum efficiency for both directions of flow simultaneously under a given set of conditions. It can be shown that in order for the maximum band width in both directions to be reached simultaneously, the offset of the center of the green interval at every signal must be either 0 percent or 50 percent. This condition then specifies that there are only $2^{(N-1)}$ possible combinations of offsets, where N is the number of signals in the system.

There are several adequate algorithms, or techniques, for determining which of the $2^{(N-1)}$ possible solutions is the best solution without testing every possible combination (1, 2, 3). The algorithm used in the computer program described is essentially the same as that described by Brooks (1). From a base signal, which is the signal having the shortest green interval, the two progressive bands for this interval width are created. The interferences to these bands resulting from both the 0 percent and 50 percent offset conditions are determined for each signal. The total interference to the bands is then selected in such a way that it is a minimum; hence, the band width is a maximum.

Cycle Length

For all signals in a system to be timed for progressive movement, they must have the same basic cycle length or be harmonic to the basic cycle length (double, half, etc.). The harmonic case is rarely justified and generally causes more interference than

benefit to progressive traffic movement. Its use may be employed in special situations; but for this program it will not be further considered.

With all signals in a system having the same cycle length, the efficiency of the system will vary as that common cycle length is varied. At some point within the range of acceptable cycle lengths (usually between 40 and 120 sec), the efficiency will reach a peak. The cycle length at the peak efficiency may not necessarily be a multiple of 5 sec. For example, it may be determined that a cycle length of 58.9 sec is the optimum for a given set of conditions. However, inasmuch as the progressive speeds and the cycle length are inversely proportional, and since a cycle length which is a multiple of 5 sec is generally required from a practical standpoint, a slight adjustment in the progressive speeds will result in a usable cycle length at the peak efficiency.

Progressive Speeds

In time-space diagrams designed in the past, the progressive speed has frequently been considered as a by-product of the design. Any progressive speed that happened to result from a given timing plan was accepted as long as it was not extreme. Traffic flow was expected to adjust to that progressive speed, whether it was higher or lower than normal traffic speeds. However, studies by Desrosiers and Leighty (4) have shown that drivers will not adjust their speeds to coincide with some arbitrary progressive speed. Therefore, if the progressive timing is to be beneficial and serve its intended purpose, it is necessary for the system to have progressive speeds which coincide with the normal movement of traffic.

As previously indicated, many elements of the traffic-signal system influence the speeds at which vehicles travel. Field studies should be made for each section of the system to determine those progressive speeds that will be most beneficial to traffic flow. These speeds are often considered to be the directional average running speeds for the section. The desired progressive speeds must be specified in advance, and only slight variations from these speeds should be permitted in the design if the timing plan is to be effective.

Split

The split is the ratio of time devoted to each phase at a signalized location. This phase split is commonly determined on the basis of the roadway characteristics and traffic volumes for each approach. This determination, however, is subject to certain overriding conditions. A minimum amount of time generally must be provided for any phase. Where pedestrians are present, a minimum amount of time must be provided to allow the pedestrians to cross the street. Also, in determining the length of the green interval, which is the portion of the phase available to accommodate the progressive bands, a fixed amount of time for the clearance period must be deducted from the phase split allotment. Accordingly, there is a different green interval for each cycle length, and any procedure to find the optimum cycle must of necessity use the appropriate green interval for that cycle.

APPROACH

The techniques used in this computer program for determining traffic-signal-system timing plans differ from those used in the conventional time-space diagram. The conventional approach is based on a plot of time along one axis of the diagram (more commonly the X-axis) and of distance along the other. Speeds are represented by the slope of a line on the diagram; thus, a line parallel to the time axis represents a speed of zero. Figure 1 shows a conventional time-space diagram and identifies the various elements.

The approach used by the author in determining traffic-signal-system timing plans converts all speed and distance units to travel time units. The diagram is then constructed in terms of time along both axes; the distance axis being replaced by an average-travel-time axis. This modification of the conventional time-space diagram is made to account for the variable élements and to simplify the calculations.

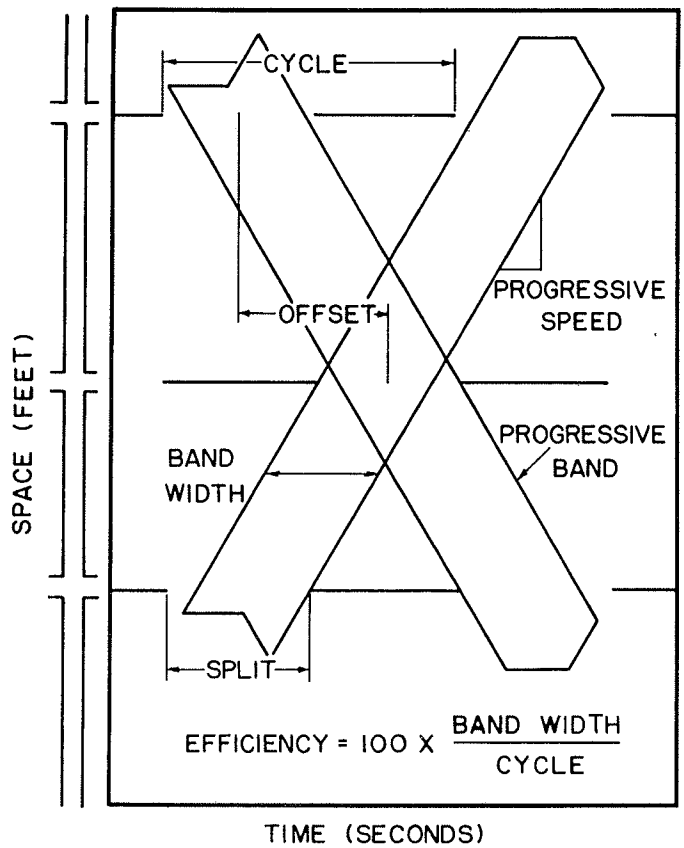

Figure 1. Conventional time-space diagram.

To construct the modified diagram, the travel distances between stop lines must be known for each direction of flow at successive signalized locations. The desired progressive speed in each direction for each section of the system also must be specified.

For example, a section of roadway having different directional travel distances and different desired progressive speeds is shown in Figure 2. The desired progressive speeds between signals at A and C are constant, but they are different between the other pairs of signals. Similarly, the directional travel distances between signals at D and E are constant, but they are different throughout the balance of the system. For simplicity in the example, the splits at all signals provide 50 percent green time to the system.

The time to travel between successive signals is calculated for each direction by dividing the travel distance for that direction by the desired progressive speed. The two directional travel times are then averaged to determine the average travel time for the section. This average travel time is used in determining the spacing of signals along the average-travel-time axis.

Inasmuch as travel times are employed in solving the problem instead of actual distances and desired progressive speeds, it may be easier to identify the system in terms of desired directional travel times than it is to calculate these from measured distances and desired speeds. The computer program permits this optional approach to be used.

In the example, the travel times in seconds for each section can be calculated by the formula:

$$\text{Travel time} = \frac{\text{feet}}{\text{mph} \times 1.47}$$

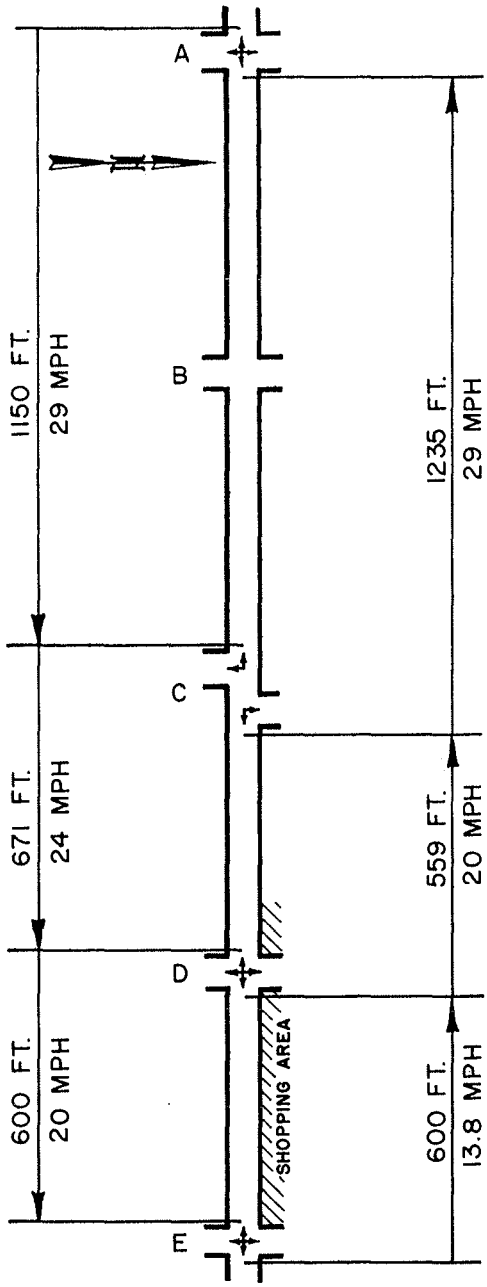

Figure 2. Roadway section for example.

TABLE 1

TRAVEL TIMES BETWEEN SIGNALS
(Seconds)

Row	Item	Section		
		AC	CD	DE
1	Westbound	29.0	19.0	29.6
2	Eastbound	27.0	19.0	20.4
3	Average	28.0	19.0	25.0
4	Difference	1.0	0.0	4.6
5	Offset shift (%)	2.0	0.0	9.2

The resulting travel times for each direction are shown in rows 1 and 2 of Table 1. The average travel time for each section is then determined (row 3 of Table 1).

The average travel-time spacing of all signals in the system is plotted along the space axis of the time-space diagram. The time-travel time diagram can then be solved manually by the usual trial-and-error, graphical, or mathematical techniques, or by a computer using appropriate algorithms to find the optimum or satisfactory linear progressive bands for both directions.

Besides the change of axis, the primary difference between the modified time-space diagram and the conventional time-space diagram is the indicator of speed. The speed of the progressive band in the conventional diagram is measured by the slope of the band. In the modified diagram, the speed is indicated by the difference in slope of the progressive band from a 45-deg angle. (If different scales are used for the two axes, speed is indicated by the difference in slope of the progressive band from the slope having a 1:1 ratio.) A progressive band having a slope of 45 deg will have a series of progressive speeds exactly equal to the desired progressive speeds; therefore, a solution with a slope in this region should be sought. At slopes flatter than 45 deg, the series of progressive speeds will be lower by the ratio of the travel time to cycle time (the slope of the progressive band).

One possible solution for the example problem (which, incidentally, is the optimum solution) is shown in Figure 3. For this solution, a 50-sec cycle has been selected which yields a band width of 20.2 sec and has an efficiency of 40.4 percent. The offset pattern for this solution is 0 percent-50 percent-0 percent-50 percent for signals A, C, D, and E, respectively.

The slope of the progressive band is slightly less than 45 deg and has a travel-time-to-cycle-time ratio of 0.94. Each of the desired progressive speeds is multiplied by this ratio to determine the actual progressive speeds at the 50-sec cycle (Table 2).

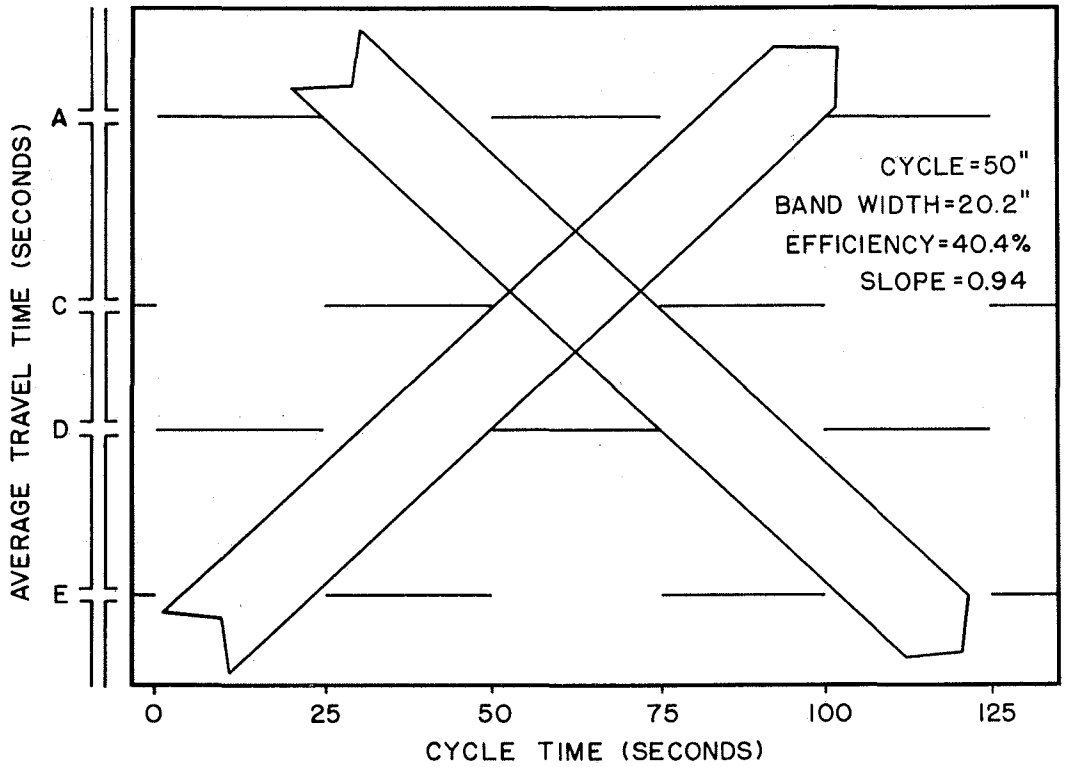

Figure 3. Solution of modified time-space diagram.

TABLE 2

DESIRED AND OBTAINED PROGRESSIVE SPEEDS (miles per hour)

Item	Section		
	AC	CD	DE
Westbound desired	29.0	20.0	13.8
Westbound obtained	27.3	18.8	13.0
Eastbound desired	29.0	24.0	20.0
Eastbound obtained	27.3	22.6	18.8

TABLE 3

OFFSET ADJUSTMENT FOR DIRECTIONAL DIFFERENCES

Row	Item	Signals			
		A	C	D	E
1	Basic offsets from diagram	0	50	0	50
2	Section AC adjustment = 2%	0	48	98	48
3	Section CD adjustment = 0%	0	48	98	48
4	Section DE adjustment = 9%	0	48	98	39

If travel times are measured instead of speeds and distances, they must be divided by this factor (0.94) to determine the travel times between signals for the progressive bands.

After the linear progressive bands in both directions have been found and the offsets for all signals determined, one final offset adjustment is necessary to account for the directional differences: the offset of all signals on one side of each section having different directional travel times must be shifted an amount equal to the difference between the travel time for the section and the directional average travel time for the section.

For the given example, this difference is included in row 4 of Table 1. By using the selected cycle length of 50 sec, the differences for each section can be converted to cycle percentages. These percentages, which are the required offset shift in seconds, are calculated by the formula:

$$\text{Offset shift} = 100 \times \frac{\text{difference}}{\text{cycle length}}$$

For the example, the results of this calculation are included in row 5 of Table 1.

The offset adjustment is accomplished by shifting the offsets of all signals on one side of the section an amount equal to the required shift contained in row 5 of Table 1. The shift is a decrease in offset for the direction having the shorter travel time.

For the example, the basic offset and all intermediate steps in the adjustment are given in Table 3. Row 1 contains the basic offset condition obtained from the time-space diagram. Each offset at this point should be either 0 percent or 50 percent for balanced progression. After all adjustments have been made, the offsets which yield equal band widths in both directions are contained in the last row of Table 3.

Further adjustments in the offsets may be made to favor one direction of flow over the opposing direction. If desired, the conventional time-space diagram can be constructed for each direction as shown in Figure 4.

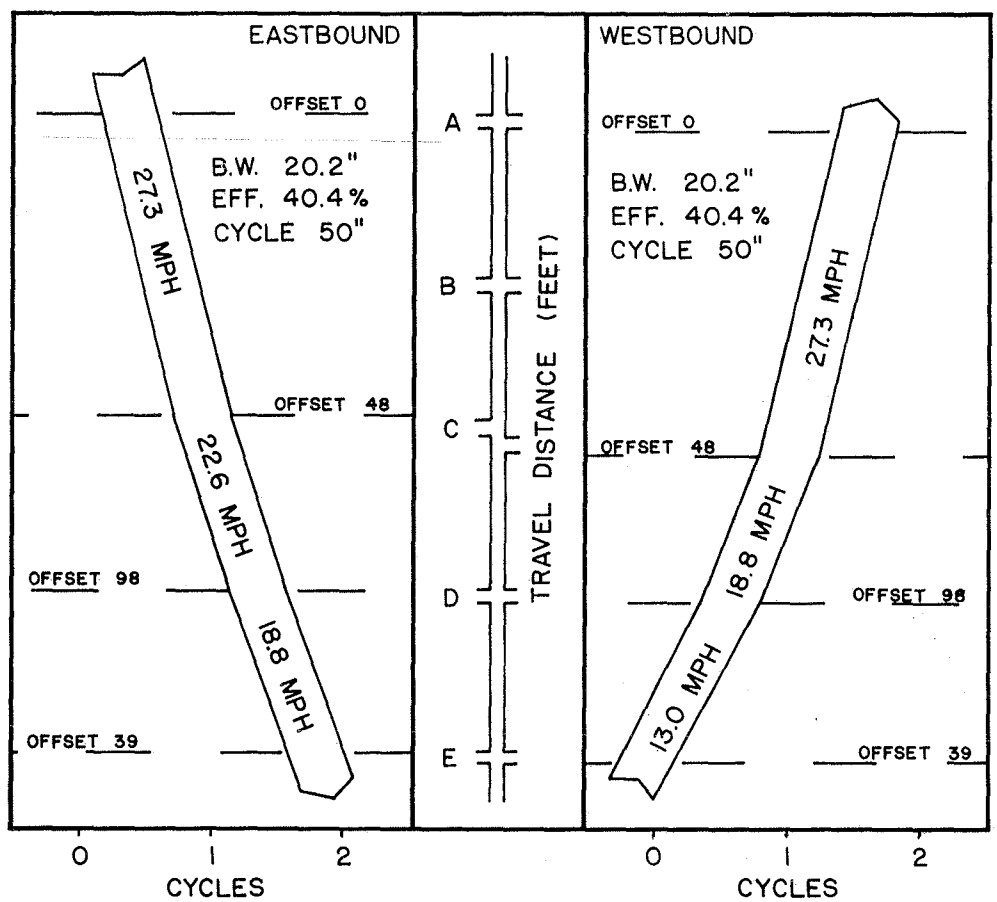

Figure 4. Conventional time-space diagram solutions.

COMPUTER PROGRAM

The computer program determines the timing plan which will produce the greatest efficiencies for a traffic signal system. The program is designed to handle all the variable elements, including signal spacing, travel speeds, cycle lengths, progressive speeds, splits, and offsets. The program was also designed so that the offsets can be adjusted to favor the band width in one direction over that of the opposing direction in proportion to the relative widths desired.

The program has been written in FORTRAN IV computer programming language. An attempt has been made to make the programming compatible with 360 FORTRAN specifications, level I, thus enabling the program to be run on almost any digital computer. The program has been extensively tested, but it has not been exposed to all possible combinations of conditions.

The running time for a problem varies with the cycle length range and the number of signals. For a typical 10-signal system and a 20-sec cycle length range, the execution time on an IBM 7040/7094 DCS is 3 sec. Input to the program is from data processing cards; output from the program is a series of printed tables.

Organization Identification Card

The name of the organization using the program is included on the printouts. This identification is coded on a single card which is placed as the first data card preceding all other data input cards. Up to 32 characters may be used for each of two lines of identification. The rightmost position of each line must be located in columns 40 and 80, respectively, of the organization identification card. An example of this card is shown with the illustration of the other input cards in Figure 5.

Input Cards

The input cards consist of 12 general control cards and a series of sets of 2, 3, or 4 signal cards. These various cards contain the basic information needed to define the system and its variability. Although the program is designed to accommodate variable conditions, it also handles constant conditions.

The general control cards are numbered from 1 through 12 and the signal cards are lettered A through E. Figure 5 shows a listing of input cards for a typical problem.

The first 40 columns of each input card are used to identify the information contained on the card. With two minor exceptions, the information contained in these 40 columns is ignored by the program. The identification information for each of the cards is in Figure 5. The last character of the identification is located in column 38 of all general control cards and signal cards.

General Control Cards—Card 1 contains the name of the system and is used to identify the problem when several runs are being made at one time and also to identify the output. Up to 40 characters may be used for the system name, which must begin in column 41.

Card 2 contains the subtitle of the run and is used primarily to identify the time of day and other conditions of the run. The subtitle, which may contain up to 40 characters, will also be included on all output. It must begin in column 41.

Card 3 identifies the number of signals in the system. The program requires at least 2 signals, and as many as 100 may be included. The units position of the number of signals must be located in column 45.

Card 4 indicates the minimum and maximum cycle lengths to be considered acceptable. If only one specific cycle length is acceptable, it should be coded as both the minimum and maximum. The units positions of the minimum and maximum cycle lengths are to be located in columns 45 and 55, respectively.

Card 5 contains the suggested maximum speed tolerance from the desired progressive speeds specified on other cards. If this item is coded 0, no tolerance will be allowed, and only cycle lengths which are multiples of 5 sec will be considered. In this case, the minimum and maximum cycle lengths coded on card 4 must also be multiples of 5 sec. If coded other than 0, the program will find, within the limits specified, the optimum cycle from all possible cycles. The units position is column 45.

```
1        10         20         30         40         50         60         70         80
I--------I---------I--CARD---I---------I---------I-COLUMNS-I---------I---------I

EXAMPLE OF ORGANIZATION IDENTIFICATION CARD
======= == ============ ============== ====
                         YALE UNIVERSITY                BUREAU OF HIGHWAY TRAFFIC

EXAMPLE OF INPUT CARDS FOR ONE RUN
======= == ===== ===== === === ===
 1                      NAME OF SYSTEM  HIGHLAND DRIVE (US-55) SIGNAL SYSTEM
 2                          SUB-TITLE  EVENING PRE-PEAK TRAFFIC PATTERN
 3          NUMBER OF SIGNALS IN SYSTEM      6
 4            MIN AND MAX CYCLES (SECS)     40         70
 5       SUGGESTED MAX SPEED TOL. (MPH)      3
 6    SYSTEM OFFSET TRANSPOSITION (PCT)      0
 7                     COLUMN HEADINGS   DIR. 1    DIR. 2   NAME/PED-X  SPLIT/CI
 8             DIRECTION IDENTIFICATION  OUTBOUND  INBOUND
 9            BAND WIDTH PROPORTIONMENT     55         45
A     LOCATION NAME + PHASE SPLIT (PCT)                      ROCK VIEW      52
B    PED X-ING + CLEARANCE TIMES (SECS)                          17           4
C                TRAVEL DISTANCE (FEET)    385
D       PROGRESSIVE SPEED DESIRED (MPH)     30
A     LOCATION NAME + PHASE SPLIT (PCT)                      BAKER          59
B    PED X-ING + CLEARANCE TIMES (SECS)                          15           4
C                TRAVEL DISTANCE (FEET)    767        848
D       PROGRESSIVE SPEED DESIRED (MPH)     30         31
A     LOCATION NAME + PHASE SPLIT (PCT)                      LANCASTER     47.5
B    PED X-ING + CLEARANCE TIMES (SECS)                        17.5          3.5
C                TRAVEL DISTANCE (FEET)  1514.8     1422.9
D       PROGRESSIVE SPEED DESIRED (MPH)   33.5       35.5
A     LOCATION NAME + PHASE SPLIT (PCT)                      EVERETT        54
B    PED X-ING + CLEARANCE TIMES (SECS)                          17           3
C                TRAVEL DISTANCE (FEET)    494        534
D       PROGRESSIVE SPEED DESIRED (MPH)     34         36
A     LOCATION NAME + PHASE SPLIT (PCT)                      YORK           60
B    PED X-ING + CLEARANCE TIMES (SECS)                          16           3
C                TRAVEL DISTANCE (FEET)   1049
D       PROGRESSIVE SPEED DESIRED (MPH)     37         39
A     LOCATION NAME + PHASE SPLIT (PCT)                      LAKE           64
B    PED X-ING + CLEARANCE TIMES (SECS)                          17           3
10   PROCESSING INSTRUCTION TO COMPUTER    RUN
11        PUNCHED OUTPUT REQUESTED ALSO    YES
12               ADDITIONAL RUNS FOLLOW     NO

EXAMPLES OF CARD E (OPTIONAL TO CARDS C AND D)
======== == ==== =  ======== == ===== = === =
E    TRAVEL TIME BETWEEN SIGNALS (SECS)     24
E    TRAVEL TIME BETWEEN SIGNALS (SECS)     18        19.5
```

Figure 5. Examples of input data cards.

Card 6 is used to transpose, by a constant amount, the offsets of all signals in the timing plan. This feature facilitates the establishment of the proper offset relationship for signals common to more than one signal system. The offset transposition on this card would be coded 0 in the initial runs and coded with the desired offset transposition in a subsequent run, if desired. The units position of the offset shift (in percent) must be in column 45.

Card 7 is included in the input deck for convenience sake only and does not change from run to run. It aids in reviewing a listing of the input cards, but contributes nothing to the program. It may not be omitted, however.

Card 8 identifies the two general directions in which traffic moves along the system. This identification would normally be the words INBOUND, OUTBOUND, or any of the

8 compass points followed by the suffix BOUND, such as N-BOUND. Up to eight characters may be used for identifying each direction. The directions must begin in columns 41 and 51, respectively.

Card 9 signifies the desired proportionment of the band widths by direction. This proportionment may be presented as a percentage (such as 50 50), or as a ratio (such as 2 1); if it is desired to have band widths proportional to directional traffic volumes, these volumes may be used to specify the proportionment. In favoring the band width in one direction over that of the opposite direction, the program will accomplish one of three things: (a) if the proportion is realistically attainable, it will be obtained; (b) if the band width in the preferred direction reaches its maximum possible width before the desired proportion is reached, no further adjustment will be made and the maximum band width condition will be indicated by asterisks on the printout; and (c) if the band width in the unfavored direction becomes small enough to no longer be meaningful (for this program, 5 sec or less), the maximum attainable band width in the favored direction will be selected. The units positions for the proportionment factors of the two directions indicated on Card 8 must be in columns 45 and 55, respectively.

Card 10 contains the processing instruction SCAN, to scan the cycle range in order to find the peaks in the efficiency curve (which peaks may be further investigated in subsequent runs), or RUN, to process the full program using the cycle length found in the scanning process to have the highest efficiency. With either of these instructions, the last letter of the message must be placed in column 45.

Card 11 is used to produce an intermediate deck of output cards containing the parameters of the optimum timing plan. This intermediate deck provides the results of the timing plan in a form which could be used by a supplemental program designed to plot, draw, or print the time-space diagram of the solution. The message YES or NO is punched with the last letter of the message in column 45. A YES message produces the intermediate output only if the option RUN on card 10 was selected.

Card 12 contains a message to inform the computer whether or not additional runs follow. If input cards for another run follow, a YES is punched with the last letter of the message in column 45. If NO is punched, the program will terminate after completing the current run.

The general control cards numbered 1 through 9 precede, in numerical order, the signal cards A through E in the input deck. General control cards 10 through 12 follow the signal cards.

Signal Cards—Card A contains the name of the signal location and the percent phase split to be devoted to the system being timed, regardless of minimum green time requirements for the cross street or for pedestrians to cross the system. The name may occupy up to 12 characters and must begin in column 61. The units position for the percent phase split is located in column 77.

Card B contains the minimum total time that must be provided during each cycle for the cross street or to allow pedestrians to cross the system. The units position is located in column 67. Also contained on this card is the required clearance interval (in seconds) for the traffic on the system. The units position (or the decimal point, if a decimal value is used) is located in column 77.

Card C contains the directional travel distances (in feet) from the signal described on the preceding Card A to the signal described on the following Card A. The units positions must be located in columns 45 and 55, respectively, for the directions specified on Card 8. In case the travel distances in both directions are identical, the second field may remain blank and the computer will make the entry.

Card D contains the desired directional progressive speeds (in miles per hour) from the signal described on the preceding Card A to the signal described on the following Card A. The units positions must be located in columns 45 and 55. In case the progressive speeds in both directions are identical, the second field may be left blank and the computer will make the other entry.

Card E contains the desired directional travel times (in seconds) from the signal described on the preceding Card A to the signal described on the following Card A. This card is complementary to Cards C and D. If Card E is included, Cards C and D must

be omitted. If Cards C and D are included, Card E must be omitted. The program distinguishes this card by the letter E in column 1. The units positions of the travel times must be located in columns 45 and 55 on Card E. In case the desired travel times are identical in both directions, the second field may be left blank and the computer will make the second entry.

A series of either three or four cards, ABE or ABCD, will be followed by another series of either three or four cards for each signal in the system. The last series will contain only two cards, AB. The entire set of signal cards designated by the alphabetic characters is located in the input deck between general control Cards 9 and 10. The order in which the signal card sets are placed in the deck must correspond with the direction designated on Card 8 under the heading DIR. 1.

If decimal distances, speeds, travel times, splits, cycle lengths, offsets, or intervals are used, these items may be coded on the various general control cards and signal cards by locating the decimal point in the specified units position and following it by one or two decimal places. The only exception is the number of signals on Card 3, which must be a whole number and may not have a decimal point punched.

Printed Output

The program is designed to prepare three tables on an on-line printer. If necessary, the program can be easily modified to punch the output or write it on tape for subsequent printing.

The first table (Fig. 6) is included in the output primarily as a reference convenience. It is a listing of the parameters and controls transmitted to the program from the input card deck which rapidly reveals mispunched or miscoded information. The

INPUT INFORMATION

1	NAME OF SYSTEM	HIGHLAND DRIVE (US-55) SIGNAL SYSTEM			
2	SUB-TITLE	EVENING PRE-PEAK TRAFFIC PATTERN			
3	NUMBER OF SIGNALS IN SYSTEM	6			
4	MIN AND MAX CYCLES (SECS)	40.00	70.00		
5	SUGGESTED MAX SPEED TOL. (MPH)	3.00			
6	SYSTEM OFFSET TRANSPOSITION (PCT)	0.00			
7	COLUMN HEADINGS	DIR. 1	DIR. 2	NAME/PED-X	SPLIT/CI
8	DIRECTION IDENTIFICATION	OUTBOUND	INBOUND		
9	BAND WIDTH PROPORTIONMENT	55.00	45.00		
A	LOCATION NAME + PHASE SPLIT (PCT)			ROCK VIEW	52.00
B	PED X-ING + CLEARANCE TIMES (SECS)			17.00	4.00
C	TRAVEL DISTANCE (FEET)	385.00	385.00		
D	PROGRESSIVE SPEED DESIRED (MPH)	30.00	30.00		
A	LOCATION NAME + PHASE SPLIT (PCT)			BAKER	59.00
B	PED X-ING + CLEARANCE TIMES (SECS)			15.00	4.00
C	TRAVEL DISTANCE (FEET)	767.00	848.00		
D	PROGRESSIVE SPEED DESIRED (MPH)	30.00	31.00		
A	LOCATION NAME + PHASE SPLIT (PCT)			LANCASTER	47.50
B	PED X-ING + CLEARANCE TIMES (SECS)			17.50	3.50
C	TRAVEL DISTANCE (FEET)	1514.80	1422.90		
D	PROGRESSIVE SPEED DESIRED (MPH)	33.50	35.50		
A	LOCATION NAME + PHASE SPLIT (PCT)			EVERETT	54.00
B	PED X-ING + CLEARANCE TIMES (SECS)			17.00	3.00
C	TRAVEL DISTANCE (FEET)	494.00	534.00		
D	PROGRESSIVE SPEED DESIRED (MPH)	34.00	36.00		
A	LOCATION NAME + PHASE SPLIT (PCT)			YORK	60.00
B	PED X-ING + CLEARANCE TIMES (SECS)			16.00	3.00
C	TRAVEL DISTANCE (FEET)	1049.00	1049.00		
D	PROGRESSIVE SPEED DESIRED (MPH)	37.00	39.00		
A	LOCATION NAME + PHASE SPLIT (PCT)			LAKE	64.00
B	PED X-ING + CLEARANCE TIMES (SECS)			17.00	3.00
10	PROCESSING INSTRUCTION TO COMPUTER	RUN			
11	PUNCHED OUTPUT REQUESTED ALSO	YES			
12	ADDITIONAL RUNS FOLLOW	NO			

Figure 6. First table printed by computer.

table includes the system identification and control information, minimum and maximum cycle lengths, suggested maximum speed tolerance, number of signals, band width proportionment factors, directional distances and speeds or travel times between each pair of signals, and the name, phase split, pedestrian crossing time and clearance period for each signal.

```
CYCLE SCAN FOR BEST FIT

HIGHLAND DRIVE (US-55) SIGNAL SYSTEM
EVENING PRE-PEAK TRAFFIC PATTERN

EFF = 14.606 AT  37. SECS                  15
EFF = 18.722 AT  38. SECS                      19
EFF = 16.979 AT  39. SECS                    17
EFF = 15.324 AT  40. SECS                  15
EFF = 12.892 AT  41. SECS               13
EFF = 11.641 AT  42. SECS              12
EFF = 14.928 AT  43. SECS                  15
EFF = 16.420 AT  44. SECS                   16
EFF = 15.544 AT  45. SECS                   16
EFF = 14.706 AT  46. SECS                  15
EFF = 15.458 AT  47. SECS                  15
EFF = 16.511 AT  48. SECS                    17
EFF = 17.439 AT  49. SECS                    17
EFF = 17.238 AT  50. SECS                    17
EFF = 19.959 AT  51. SECS                       20
EFF = 22.328 AT  52. SECS                         22
EFF = 23.864 AT  53. SECS                            24
EFF = 25.344 AT  54. SECS                             25
EFF = 26.769 AT  55. SECS                               27
EFF = 28.144 AT  56. SECS                                28
EFF = 28.323 AT  57. SECS                                28
EFF = 27.890 AT  58. SECS                                28
EFF = 27.473 AT  59. SECS                               27
EFF = 27.069 AT  60. SECS                               27
EFF = 26.679 AT  61. SECS                               27
EFF = 26.301 AT  62. SECS                              26
EFF = 25.739 AT  63. SECS                              26
EFF = 24.658 AT  64. SECS                             25
EFF = 22.986 AT  65. SECS                           23
EFF = 21.296 AT  66. SECS                         21
EFF = 19.658 AT  67. SECS                        20
EFF = 18.067 AT  68. SECS                      18
EFF = 16.523 AT  69. SECS                    17
EFF = 15.022 AT  70. SECS                  15
EFF = 14.148 AT  71. SECS                14
EFF = 14.771 AT  72. SECS                 15
EFF = 15.377 AT  73. SECS                 15
EFF = 15.447 AT  74. SECS                 15
EFF = 14.661 AT  75. SECS                 15

BEST FIND IS
   CYCLE OF  57.  EFFICIENCY =  28.323

ITERATION IMPROVEMENTS
              56.500              28.545
              56.500              28.545
              56.375              28.601
              56.375              28.601
              56.344              28.605
              56.359              28.608
              56.359              28.608
              56.359              28.608
              56.359              28.608
              56.359              28.608
```

Figure 7. Second table printed by computer.

The second table (Fig. 7) contains the results of an incremental cycle scan between the minimum and maximum cycle lengths to find the maximum efficiency obtainable at each increment. A plot of the maximum efficiency obtainable at each cycle length is included in this printout. The cycle length having the highest efficiency is identified, and improvements in the efficiency by an iterative process are also included. This tabulation is useful in making further investigations at other cycles where the efficiency reaches lesser peaks. These additional investigations are made by running the program again with the minimum and maximum cycles changed to encompass only the cycle span associated with the lesser peak in the efficiency.

If the instruction RUN is contained on Card 10, the program continues using the cycle having the highest efficiency and prints the third table (Fig. 8). This tabulation indicates the timing elements that yield the greatest efficiency under the specified conditions. Each signal is identified by its name. The offsets and the system's green and clearance intervals (in percent) are listed adjacent to the name. The offsets are given

TRAFFIC SIGNAL SYSTEM TIMING PLANS

HIGHLAND DRIVE (US-55) SIGNAL SYSTEM — YALE UNIVERSITY
EVENING PRE-PEAK TRAFFIC PATTERN — BUREAU OF HIGHWAY TRAFFIC

PLAN 56.4 (55- 45)	CYCLE LENGTH (SECONDS)	56.4	55.0	50.0	60.0
********** LISTING IS IN OUTBOUND DIRECTION **********	OUTBOUND DIRECTION				
	BAND WIDTH (SECONDS)	17.7	17.3	15.7	18.9
	EFFICIENCY (PER CENT)	31.5	31.5	31.5	31.5
	INBOUND DIRECTION				
	BAND WIDTH (SECONDS)	14.5	14.2	12.9	15.4
	EFFICIENCY (PER CENT)	25.7	25.7	25.7	25.7

SIGNAL LOCATION	OFFSETS-PER CENT BEG.	MID.	END.	INTERVAL G-PCT.-Y	SIGNAL SPCNG.	OPT. PROGRESSIVE	BEST	--OTHERS-- SPEEDS-MPH	
ROCK VIEW	27.5	50.0	72.5	44.9 7.1					
P.BAND	31.8		63.3	OUTBOUND	385.0	30.0	30.7	33.8	28.2
P.BAND	42.4		68.2	INBOUND	385.0	30.0	30.7	33.8	28.2
BAKER	26.9	52.9	78.8	51.9 7.1					
P.BAND	47.3		78.8	OUTBOUND	767.0	30.0	30.7	33.8	28.2
P.BAND	26.9		52.7	INBOUND	848.0	31.0	31.8	34.9	29.1
LANCASTER	78.3	98.9	19.6	41.3 6.2					
P.BAND	78.3		9.7	OUTBOUND	1514.8	33.5	34.3	37.8	31.5
P.BAND	93.8		19.6	INBOUND	1422.9	35.5	36.4	40.0	33.3
EVERETT	27.7	52.0	76.4	48.7 5.3					
P.BAND	33.0		64.4	OUTBOUND	494.0	34.0	34.8	38.3	31.9
P.BAND	45.3		71.1	INBOUND	534.0	36.0	36.9	40.6	33.8
YORK	27.3	54.7	82.0	54.7 5.3					
P.BAND	50.6		82.0	OUTBOUND	1049.0	37.0	37.9	41.7	34.8
P.BAND	27.4		53.1	INBOUND	1049.0	39.0	40.0	44.0	36.6
LAKE	73.4	2.7	32.1	58.7 5.3					
P.BAND	84.9		16.3	OUTBOUND					
P.BAND	94.8		20.6	INBOUND					

Figure 8. Third table printed by computer.

to the beginning, middle, and end of the green interval, since different offsets are usually required for pretimed systems than for actuated systems. The provision of all three offsets also facilitates the drafting of a time-space diagram, if desired.

The fourth column from the right (Fig. 8) (labeled OPT.) indicates the cycle length, band widths, efficiencies, and directional progressive speeds for the optimum solution. Since the optimum cycle cannot normally be installed in most equipment unless it is an exact multiple of 5 sec, the next column to the right (BEST) indicates the changes resulting from a change in the cycle length to the nearest multiple of 5 sec. The last two columns show the changes resulting from the cycle lengths 5 sec above and below the BEST cycle length. In creating these last three columns, no changes in the splits, intervals, or offsets have been made. They represent a direct expansion or contraction of the cycle length, as would be effective by solely a change in the cycle gear. More specific split, interval, band width, and offset information can be obtained by running the problem again using the progressive speeds and cycle length contained in any one of these last three columns. From a practical viewpoint, however, another run would not materially change the timing values in Figure 8.

In cases where the spacing between signals is given in the input as desired travel times instead of travel distances and desired progressive speeds, the progressive speeds (Fig. 8) are in terms of seconds instead of miles per hour. The signal spacing (in the column to the left of the progressive speeds) is listed in terms of seconds instead of feet.

The offsets to the beginning and ending edges of the progressive bands as they pass the signal listed above them are given in the left part of Figure 8 on the same lines as the progressive speeds. This information is helpful in making further adjustments for leading and lagging green intervals, split phases, and other modifications, and in manually drafting the time-space diagram. If the progressive band utilizes the full green interval at any signal, an asterisk will precede the progressive band offsets on this printout.

Other information includes the plan number, which is the same as the optimum cycle, together with the requested band width proportions located in parentheses. The proportions are expressed in percentages. The system title, subtitle, and the name of the organization using the program are also included.

If the program processing option on input Card 10, SCAN, is used, as may be desirable in the first run for a system using large minimum and maximum cycle lengths, only the first two tables (Figs. 6 and 7) will be prepared by the computer.

Punched Output

If the message YES is punched in Card 11 and the option RUN was selected on Card 10, the program will punch a deck of data processing cards containing all the parameters necessary for a supplemental computer program to plot, draw, or print a time-space diagram of the optimum solution. This intermediate deck consists of two identification cards and one additional card for each signalized location in the system.

The first identification card contains the title and subtitle, as contained on Cards 1 and 2 of the input deck. The second card contains the directional identification contained on input Card 8, the optimum cycle length, the two directional band widths, and the two directional efficiencies, in that order.

Each of the remaining cards contains the following information for each signal: the name of the signal location; the offsets to the beginning, middle, and end of the green interval; the phase split, the offsets to the beginning and ending edges of the progressive band in direction 1; the average distance to the next signal; the progressive speed to the next signal; the offsets to the beginning and ending edges of the progressive band in direction 2; the progressive speed from the next signal; and the code 1 or 2 to signify that distance and velocity are in units of feet and miles per hour or in units of seconds and seconds, respectively. The last card is identified by zero distance and velocities.

ACKNOWLEDGMENT

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Use of a Computer and Vehicle Loop Detectors To Measure Queues and Delays At Signalized Intersections

A. CHRISTENSEN, Computing Devices of Canada, Ottawa

A computer program has been developed to obtain from the pulses coming from vehicle loop detectors placed near an intersection the traffic parameters of volume, speed, space headway, density, and time headway. By finding the relations between time headway and queue length and time headway and delay, it is shown how the computer program can be used to find queue length and delay at signalized intersections. For the intersection studied, vehicle arrivals at the approaches were random; i. e., there was no platoon structure remaining after passage from the previous intersections.

Graphs are shown of plots of delay vs volume, and of queue length vs volume. A means for correcting the counts lost is also developed.

•A NECESSARY part of the installation of the computer-controlled traffic signal system in Toronto is to be able to evaluate results. Therefore, many analysis programs have been devised by the Traffic Research Corporation in order to study and improve the traffic control system. Among the most important parameters to be measured are queue length and delay at signalized intersections. This report describes one program which was devised for this purpose. The program also provides other traffic parameters, such as volume, density, speed, time headway and space headway.

PROGRAM DESCRIPTION

The program is called the VDCA (volume-density curves type A). It calculates the following traffic parameters from traffic data recorded on magnetic tape:

1. Volume—vehicles per hour per lane (veh/hr/lane);
2. Speed—mph, arithmetic average over a time period at the detector;
3. Density—vehicles per mile per lane (veh/mi/lane), over a time period at the detector;
4. Space headway—ft, over a time period at the detector; and
5. Time headway—seconds per vehicle, over a time period at the detector.

The computer flow chart for finding these parameters is shown in Appendix B. A typical printout is shown in Figure 1.

The same detectors that are used for control purposes are also used for evaluation purposes. The positioning and use of the detectors for control purposes has already been reported (3). For reasons of economy, two loops in two adjacent lanes are connected to one piece of detector electronics. It is therefore possible for two vehicles to cross the detector loops at once, and this results in a certain loss of counts. However, a formula has been derived which allows the computer to correct the counts automatically as the program is run. A derivation of this formula is given in Appendix A. Tests have shown the corrected counts generally accurate to within 5 percent.

Paper sponsored by Committee on Traffic Control Devices.

VEHICLE DENSITY CALCULATIONS

Control Cards

Date	Start	End
Nov. 19	17.0	17.15

Control Parameters

A1	A2	A3	A4	A5	A6	A7
.200	1.990	1.000	11.060	.10820	.66000	.009330

Detector	Lanes	Mean Speed	Counts/ Lane/Hr	Density	Mean Space Headway	Mean Time Headway
143	2	22.70	491.	23.91	220.79	6.97
144	2	20.08	642.	61.56	85.76	3.47
145	2	18.49	623.	58.19	90.73	3.95
146	2	22.20	396.	18.96	278.48	8.78

Figure 1. Typical VDCA output.

Method of Calculating Space Headway and Density

Figure 2 shows the basic method used to calculate space headway. Pulses generated by vehicles are taken a pair at a time, and the average speed of the two vehicles is determined. Space headway Sn is then found by multiplying the average speed by the time headway Hn. Density, D, in veh/mi is found by dividing the number of feet in a mile by space headway: D = 5280/S veh/mi/lane, where S is the arithmetic average of all Sn.

Meaning of Density as Calculated

The term density is generally used only to apply to homogeneous conditions, because speed, time headway and space headway are nearly uniform in space and time. These conditions do not apply near a signalized intersection. The density found under nonhomogeneous conditions depends to some extent on the method of calculation used. The calculation performed in the VDCA program is felt to be reasonable; in fact, it produces very useful results.

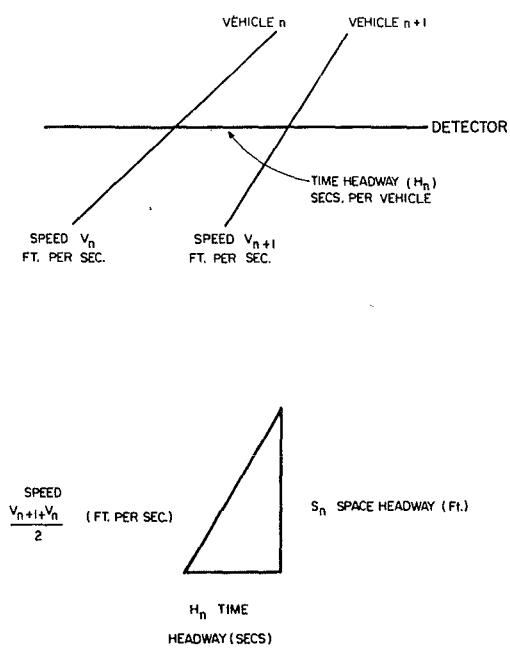

Figure 2. Method of calculating space headway.

Volume-Density Curves

The first use that was made of the VDCA program was to plot volume-density curves (Fig. 3). The first part of the curve, near the origin, always is a straight portion whose slope is equal to the free travel speed on the road link upstream of the intersection. When queues begin to extend past the detector, indicating that approach capacity is being reached, the volume-density curve becomes parallel to the axis of density—the volume reached at this plateau is a measure of approach capacity.

Since density as calculated by the program continues to increase, as queues become longer, density can also be taken as a measure of congestion even though volume remains the same.

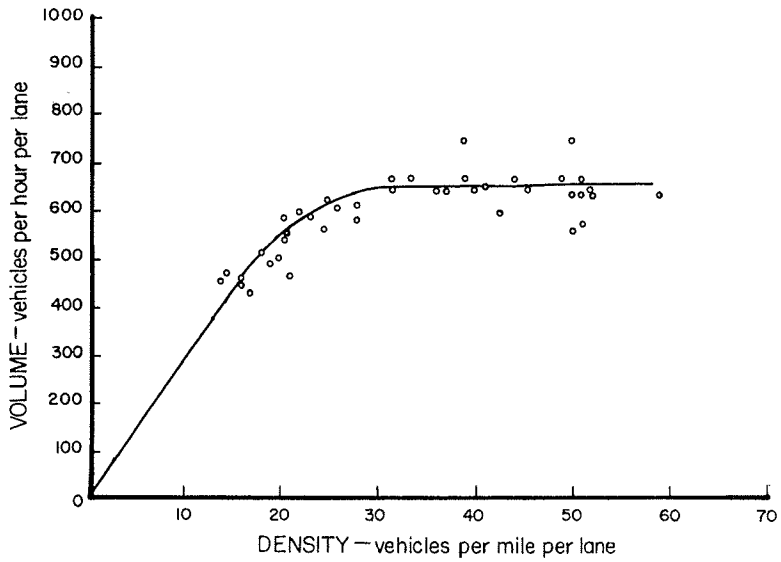

Figure 3. Volume vs density, east leg, fixed time—a.m. and p.m. (points are 15 min averages).

Calculation of Queue Length and Delay

F. V. Webster of the Road Research Laboratory has produced formulas from which queue length and delay can be found, if cycle length, split, and volume are known (1). Arrivals are random in time, meaning that there is no platoon structure in the arrivals within the cycle. His formulas have been written into a computer program by a group at MIT (2). We have prepared a modified form of the MIT program which we call QUEUE. It is theoretically possible to calculate from VDCA output the queue length and delay, provided inputs to the intersection are random in arrival. However, it has been found that in dealing with real data from detectors, difficulties present themselves when the intersection is at capacity or near it. Figure 3 shows that the throughput of an approach at capacity has considerable scatter. Thus, queue lengths and delays, as

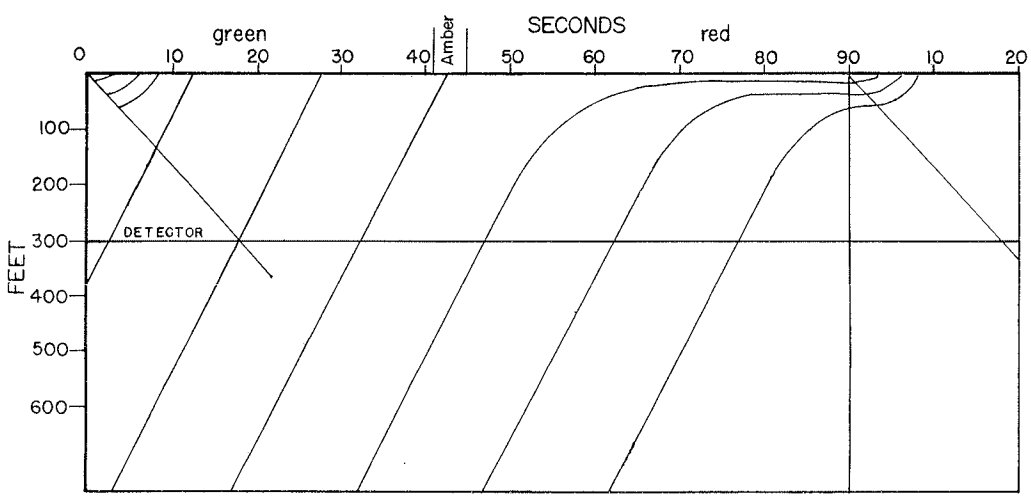

Figure 4. Trajectories in light traffic.

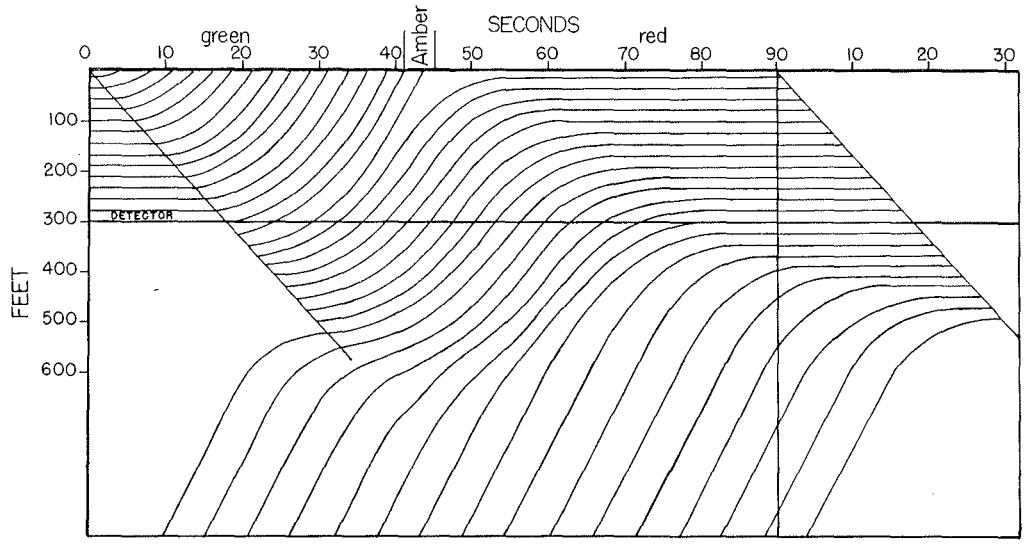

Figure 5. Trajectories in moderate congestion.

calculated, can be greatly in error near full capacity conditions when real data are used. What is needed is some quantity, measurable at the detector, which continues to increase near capacity conditions as queue lengths and delays increase, volume remaining nearly constant. This quantity could then be related to queue length and delay, resulting in more accurate results near capacity. Density, as calculated by the VDCA method, meets these requirements. However, application of this method would require a repetition of Webster's work in order to find the relationships between density, queue length, and delay. An alternative method has been devised by using average time headway at the detector. This method is described as follows.

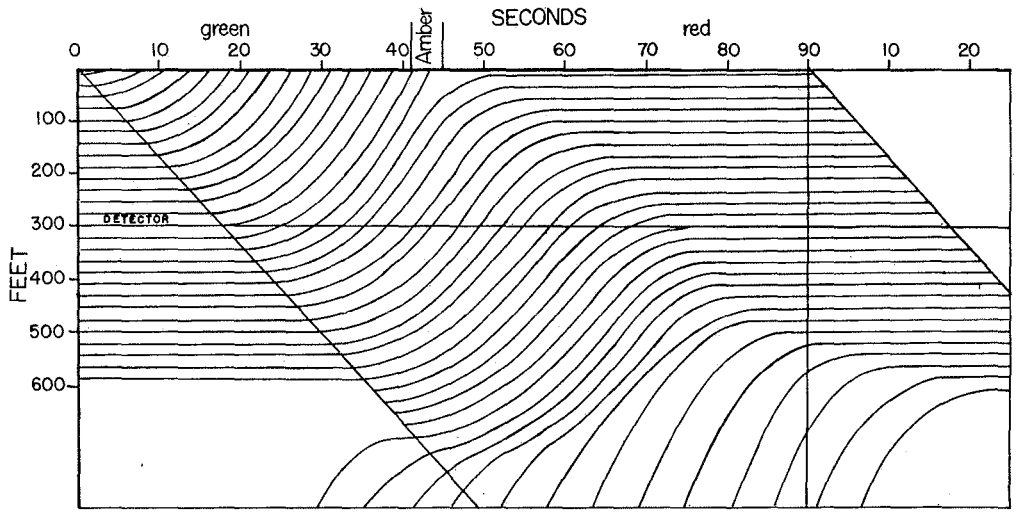

Figure 6. Trajectories in heavy congestion.

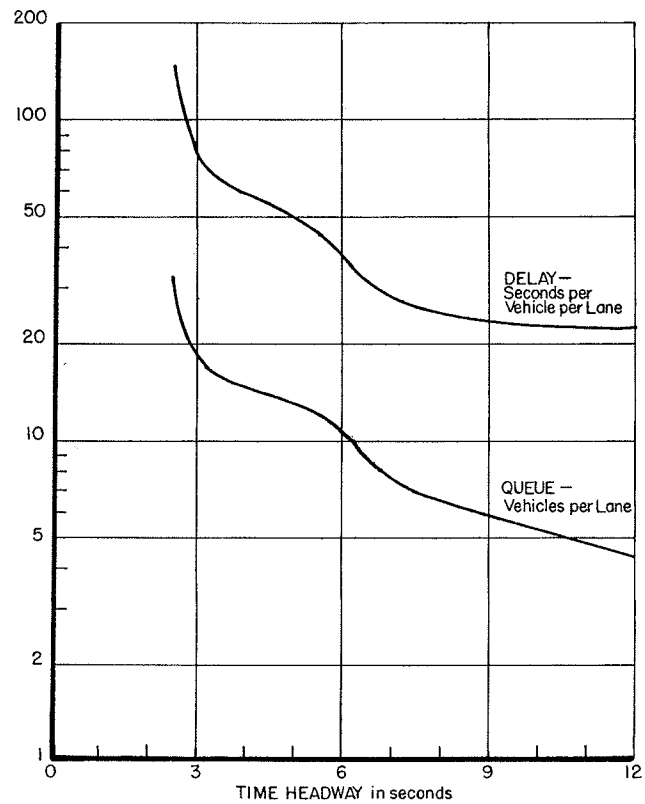

Figure 7. Calibration curve, east leg.

Figure 4 shows that when the intersection is not near capacity the average time headway at the detector is the same as that upstream of the detector, and one finds time headway simply by dividing 3600 sec by volume. As the intersection reaches capacity (Figs. 5 and 6), average time headway over the whole cycle is again found by dividing 3600 sec by volume. However, Figures 5 and 6 show that as congestion increases, vehicles move over the detector in a decreasingly smaller portion of the total cycle time. If, therefore, the average time headways can be found for only those portions of the cycle during which there is substantial flow over the detector, a quantity will have been found which, like density, continues to change rapidly while volume remains nearly constant. Two problems remain:

1. To find a means of calculating, for any split, cycle length and volume, the modified average time headway to use in producing calibration curves of time headway vs queue length, and time headway vs delay.
2. To find a means of making the VDCA program produce, not the average time headway over the cycle, but the modified time headway desired.

Appendix C explains in detail how problem 1 has been overcome. Essentially, an approximate method whereby trajectories of the vehicles can be found for different conditions has been devised. It would have been better to use a computer model of intersection behavior to obtain the necessary data. At least one such model has been constructed (5) which could be used for the purpose, but use of such a model was not considered warranted for the first trials of the method. In any case, calibration curves for the conditions prevailing at one intersection have been constructed. Figure 7 shows such a curve for the east leg of the intersection studied. As an example of how to use the calibration curves, consider the case when time headway is 3 sec (Fig. 7). Reading

TABLE 1

WEBSTER RELEASE RATES

Leg	Split	Max. Volume From Volume Density Curves (veh/hr/lane)	Webster Max. Volume (veh/hr/lane)	Webster Release Rate (sec/veh)
N	0.58	840	885	2.2
S	0.49	660	695	2.3
E	0.42	650	685	2.0
W	0.34	530	557	2.0

NOTE: The reason the splits are all different is that the north and east legs have advanced greens.

from the queue length curve, queue length (veh/lane) at the end of red is 19. Reading from the delay curve, average delay in seconds per vehicle is 75.

Problem 2 has been solved by limiting to a certain maximum level the pulse length that the VDCA program will accept. In this way, time headways are calculated only during those times during the cycle when vehicles are moving at a substantial speed over the detector, i.e., those times when only substantial flow is occurring. It should be explained that the detector has infinite presence available if desired, but for normal use we set cutoff at 30 sec by adjustments in the hardware. For the VDCA program, no changes in the hardware are made, but the program is such that it does not use pulses greater than a certain length. This critical length has been taken as 2 sec but it is an adjustable parameter.

Results for an Intersection

For release from a queue at an approach, Webster assumes a fixed time (lost time) for each phase of a cycle during which no vehicle moves; let this time be L sec. After L sec, vehicles are released at a constant rate during green + amber time; let this constant rate be r sec per vehicle. With the help of the VDCA program, r can be found; for example, the curve in Figure 3 rises to a plateau where volume is constant. The resulting volume is the capacity of the approach. The volume in this case is 650 veh/hr. For equilibrium conditions, it is reasonable to assume that this limiting volume is 95 percent of saturation flow. Saturation flow for this particular approach is thus 650/0.95 = 685 veh/hr. At this particular approach the cycle length was 90 sec, green time was 34 sec, and amber time was 4 sec. The number of vehicles released per cycle under saturation flow is thus:

$$\frac{\text{Green} + \text{Amber} - L}{r} = \frac{34.5}{r}$$

The hourly saturation flow rate is

$$\frac{34.5}{r} \times \frac{3600}{90}$$

Equating this to 685 and solving for r, it is seen that r = 2.0 sec.

The intersection studied was Bayview Avenue and Eglinton Avenue in Toronto. Performing these calculations for each approach produced the results given in Table 1.

Calibration curves were made for each approach, and the VDCA program was run for 15-min intervals from 7:30 a.m. to 9:00 a.m. and 4:30 p.m. to 6:00 p.m., for a week (Monday to Friday). The resulting delay and queue length measurements are shown in Figures 8 to 15. Not all points were plotted, since many fell on top of one another. On each plot the theoretical Webster curve is superimposed. It should be noted that since, during the times of day studied, total intersection throughput averaged over 4000 veh/hr, the results shown in the figures represent the passage of over 60,000 vehicles.

The accuracy of the results is dependent upon inputs to the approaches being random. In other words, no platoon structure remains after the vehicles have traversed the distance from the previous signalized intersection. Randomness was tested by means of the LINKS program, the features of which have already been reported by Wormleighton (4). In every case, except the east leg in the a.m., inputs were random to a high degree, and even on the east leg a.m., there was only a moderate departure from randomness.

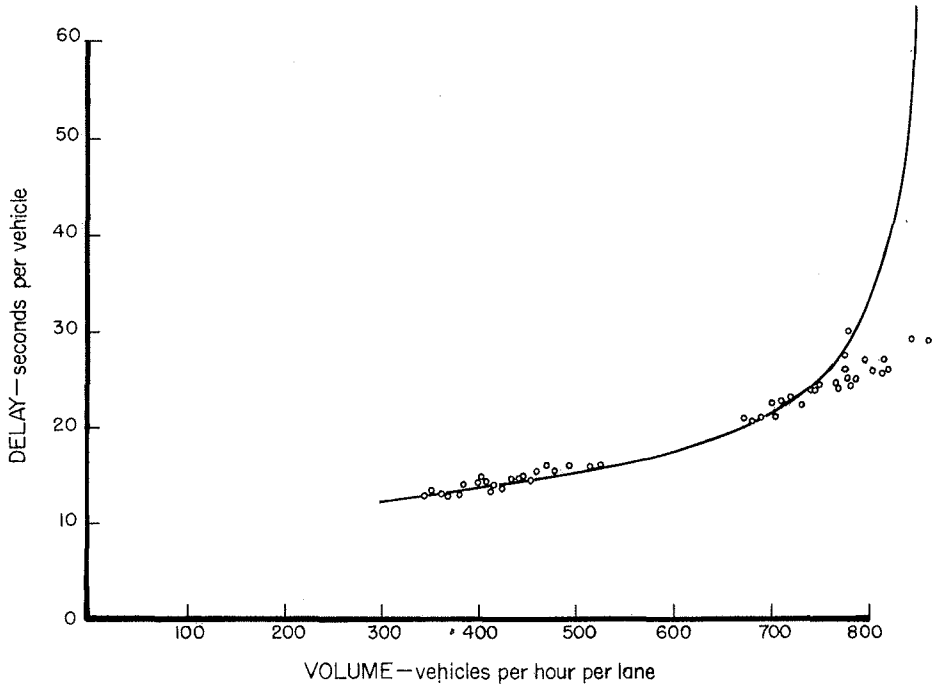

Figure 8. Delay vs volume, north leg (90 sec cycle, 0.58 split).

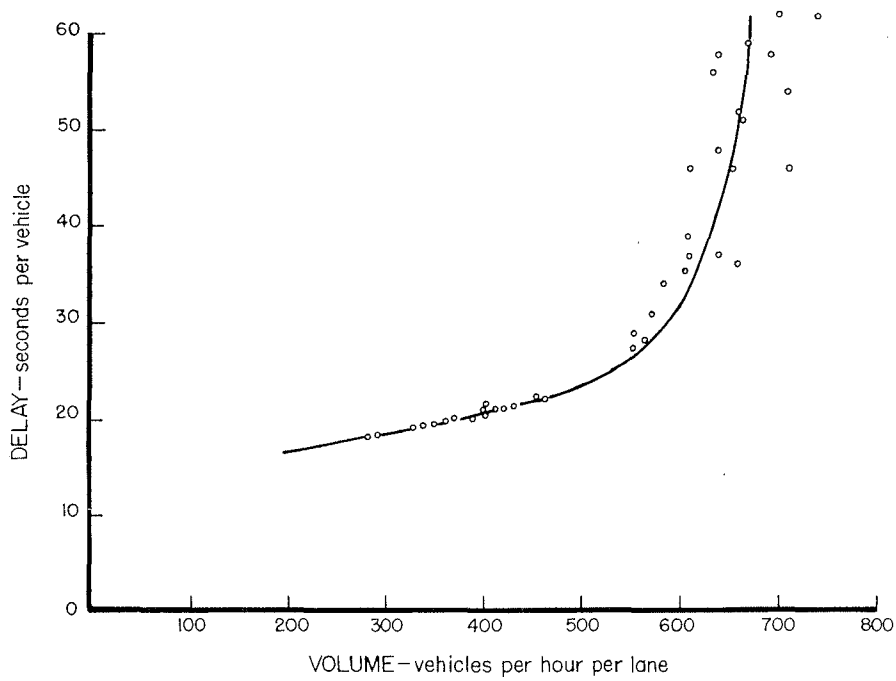

Figure 9. Delay vs volume, south leg (90 sec cycle, 0.49 split).

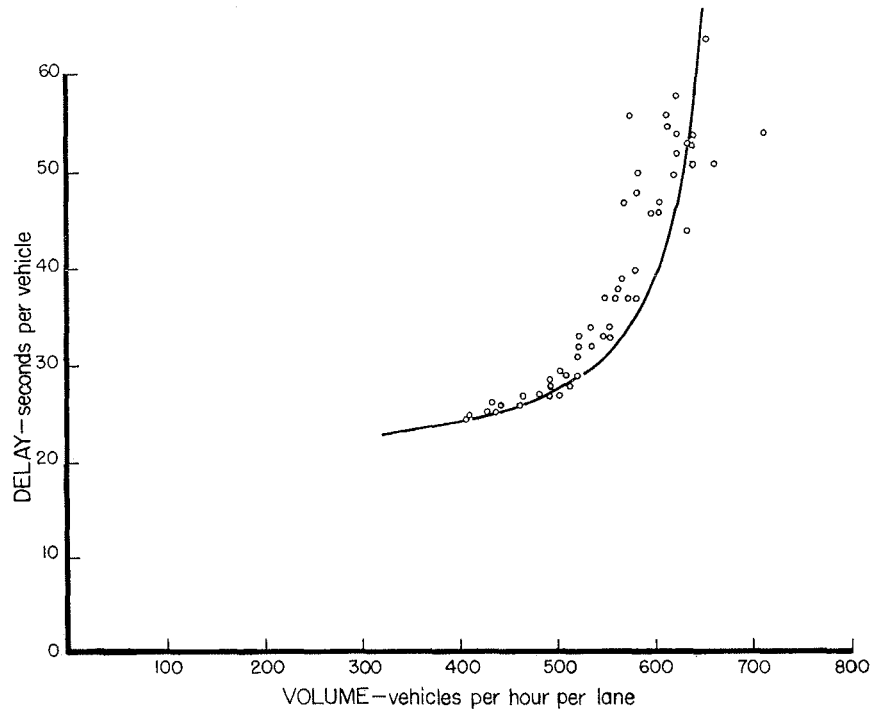

Figure 10. Delay vs volume, east leg (90 sec cycle, 0.42 split).

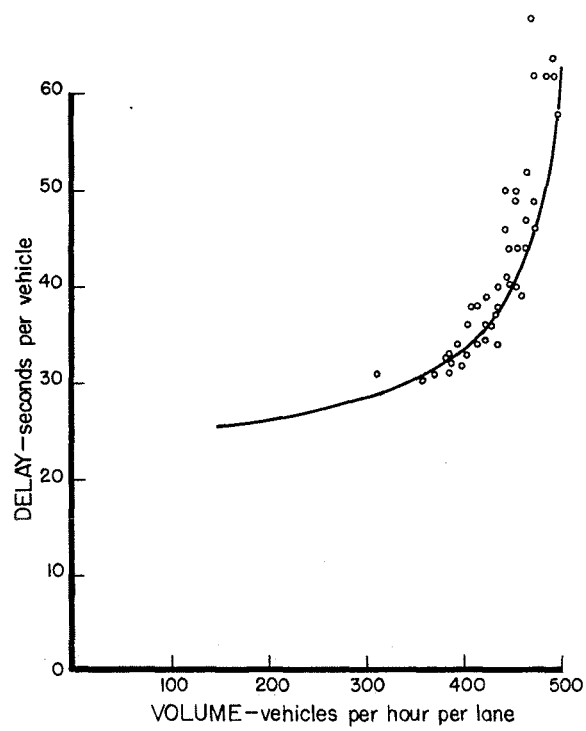

Figure 11. Delay vs volume, west leg (90 sec cycle, 0.34 split).

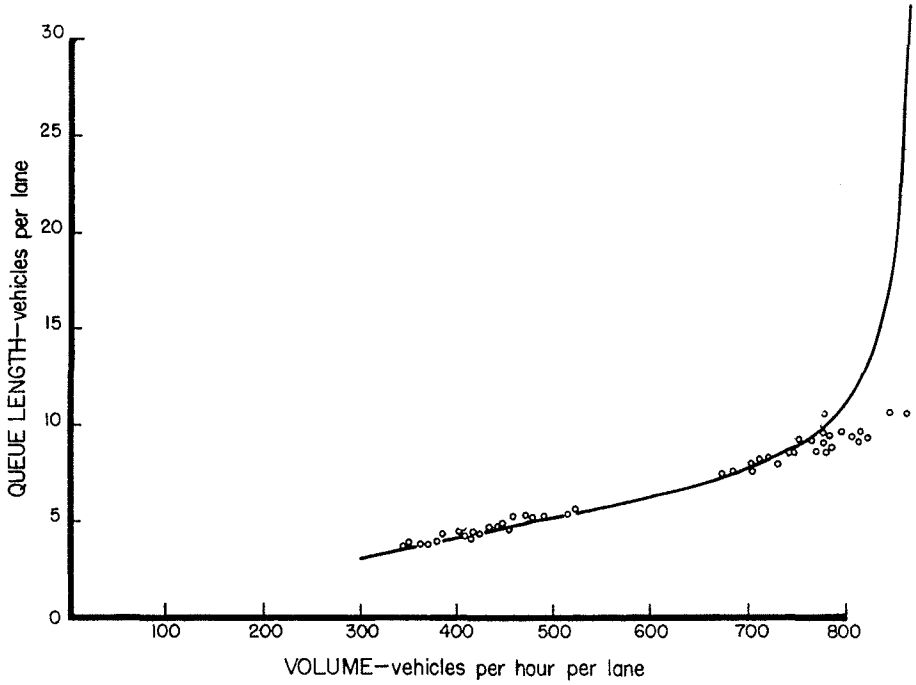

Figure 12. Queue length vs volume, north leg (90 sec cycle, 0.58 split).

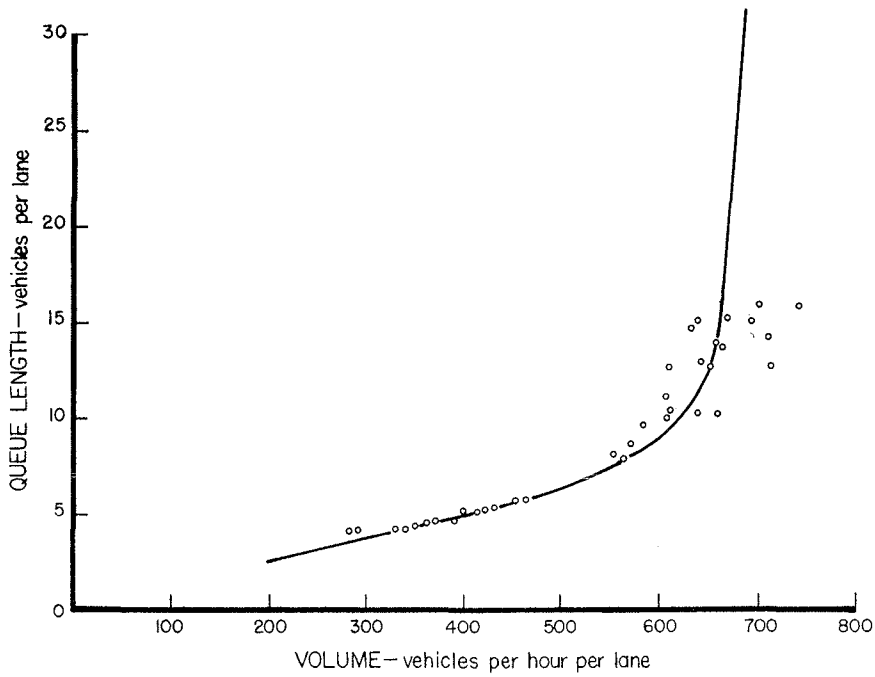

Figure 13. Queue length vs volume, south leg (90 sec cycle, 0.49 split).

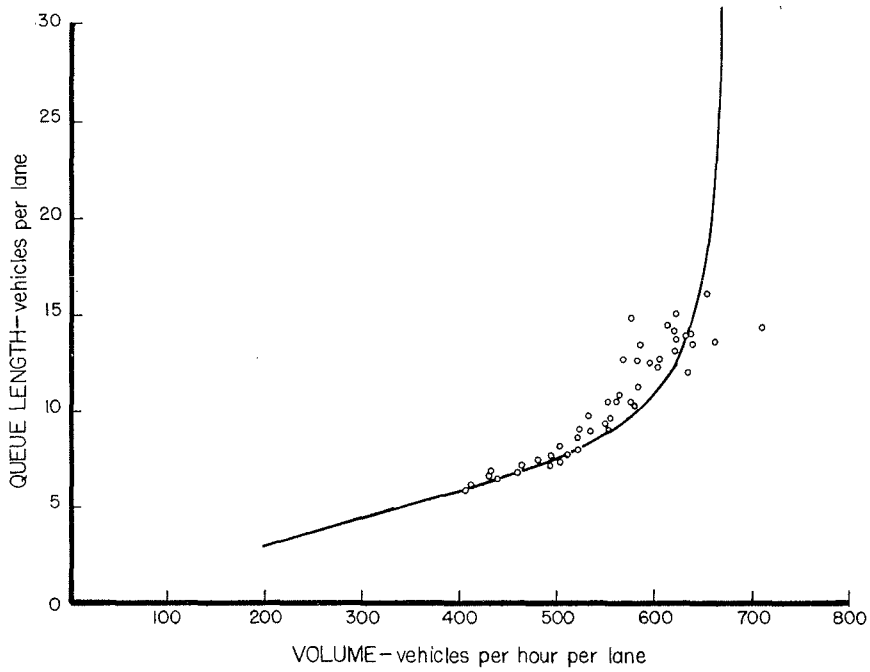

Figure 14. Queue length vs volume, east leg (90 sec cycle, 0.42 split).

Test of the Method

Trials have been held to test the accuracy of the method. Observers counted vehicles in the queue at the end of red for many cycles. The queue lengths were averaged over 15-min periods and compared with queue lengths as predicted by the VDCA method. The results of one such trial are given in Table 2. However, the results shown in Figures 12 to 15 were not checked against observation.

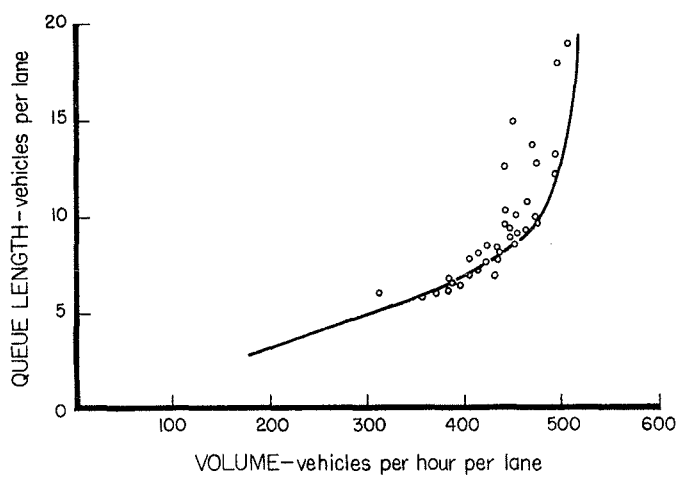

Figure 15. Queue length vs volume, west leg (90 sec cycle, 0.34 split).

TABLE 2

OBSERVED VS CALCULATED QUEUE LENGTHS

Time	Average Observed Queue at End of Red	Calculated Queue at End of Red
1609-1624	9.3	9.2
1624-1639	8.0	9.5
1639-1654	15.2	14.0
1654-1709	21.2	20.0

Use Under Other Conditions

By constructing enough trajectory diagrams, it is possible to use the VDCA method when arrivals have a definite platoon structure; this procedure has been used at Traffic Research Corporation.

It is also theoretically possible to extend the method to testing non-fixed-time intersections, but the work required would be prohibitive. Also, accuracy would be doubtful, since the method depends to a large extent on averaging over many minutes. For general use in evaluating intersection performance, we have still another program called DELAY, details of which we hope to publish soon. The DELAY program is an off-line simulation which uses as input the actual detections that have been received and recorded on magnetic tape. For the intersection studied, results from the VDCA method agree substantially with results from the DELAY program method.

CONCLUSION

A method has been found whereby the output from vehicle loop detectors situated near an intersection can be used to find traffic parameters, such as volume, density, space headway, and time headway. It has also been shown how the time headway, as calculated, can be used to produce queue lengths and delays at signalized intersections.

ACKNOWLEDGMENTS

Thanks are due to Sam Cass, Commissioner of Traffic, Metropolitan Toronto, for partial support of this research, and for donation of the computer time. Thanks are also due to Glyn Jones of Traffic Research Corporation who did the necessary programming.

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Appendix A

CORRECTION FOR COUNTS LOST DUE TO OPERATION OF ONE DETECTOR OVER TWO LANES

The design of the detector used is such that when a certain rate of increase of inductance occurs, the detector is turned on. Thus, a vehicle entering activates the detector. The opposite occurs when a vehicle leaves.

When two vehicles are passing at once, one in the first lane and one in the second lane, the following occurs. Suppose that vehicle A, in lane 1, is slightly ahead of vehicle

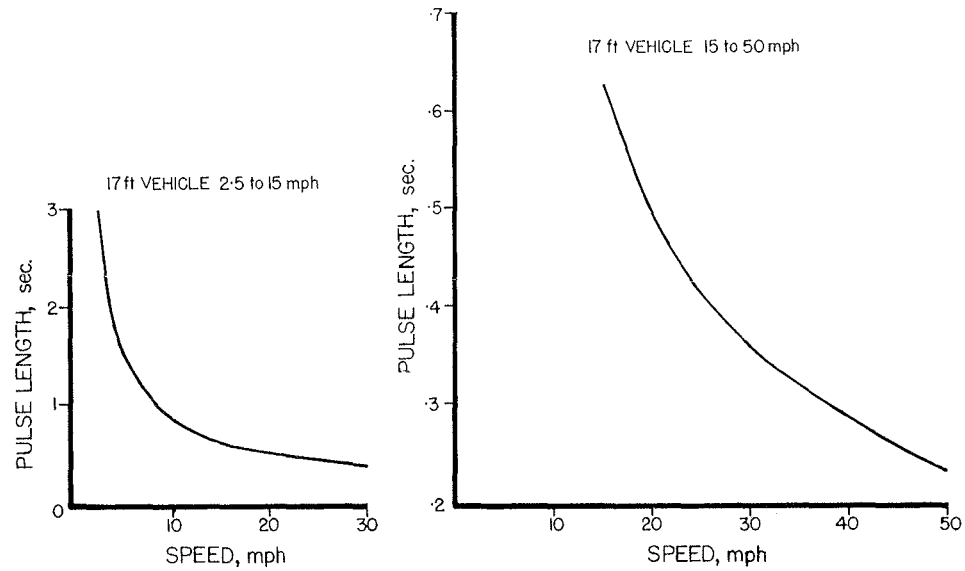

Figure A-1. Detector calibration.

B, in lane 2. Then, when the front of vehicle A passes over the loop, the detector is turned on. When the front of vehicle B crosses the loop, nothing happens, since the detector is already on. When the back of vehicle A leaves the loop, the detector is turned off. When the back of vehicle B leaves the loop, nothing further happens. It is thus evident that, if vehicles are of the same length, passage of two vehicles at once has no effect on the generated pulse length, but one count is lost.

Derivation of Correction Formula

The following shows the derivation of a formula which corrects the loss of counts due to double-lane operation. These assumptions are made: (a) in two-lane operation, traffic divides equally; (b) speeds are the same in both lanes; (c) all vehicles are 17 ft long; (d) vehicles are uniformly spaced; and (e) the pulse length vs speed calibration curve for all detectors is as shown in Figure A-1. Symbols used are as follows:

- C' Measured vehicle counts per hour, both lanes together, as received by the detector, assuming all vehicles pass over the loop, and assuming no vehicle makes more than one count;
- C Corrected vehicle counts per hour, both lanes together, after correction for loss of counts due to one detector covering two lanes;
- k Correction factor for loss of counts due to one detector covering two lanes $\left(k = \frac{C'}{C}\right)$;
- t' Measured average pulse length, sec;
- D' Measured vehicle density, veh/mi/lane, before correction factor k is applied;
- D Corrected vehicle density, veh/mi/lane $\left(k = \frac{D'}{D}\right)$;
- V Measured vehicle speed, mph;
- v Measured vehicle speed, ft/sec;
- L Measured vehicle length, ft, as seen by the detector ($L = t'v$);
- S' Measured vehicle spacing, ft, per lane; and
- S Corrected vehicle spacing, ft ($S = kS'$), per lane.

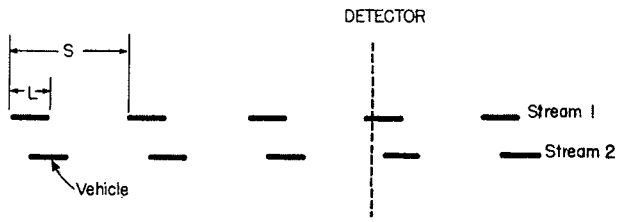

Figure A-2. Diagram of two lanes of vehicles.

Figure A-2 shows vehicle positions—if two parallel streams of traffic are crossing the detector—for one particular offset of stream one with respect to stream two. Since it is assumed that there is no correlation between the offset of streams one and two, on the average all possible offsets will be obtained. Therefore, for a fraction 2L/S of the time, two vehicles will be crossing the detector at the same time, and only one count will be obtained for two vehicles. During the remaining time (S - 2L)/S, there will be one count for each vehicle. For the time period needed to travel S, the average number of counts is

$$C' = \left(\frac{S - 2L}{S}\right) \times 2 + \frac{2L}{S} \times 1 = \frac{2S - 2L}{S} = 2\left(1 - \frac{L}{S}\right)$$

Since corrected counts C is 2, the correction factor k is

$$\frac{2\left(1 - \frac{L}{S}\right)}{2} = 1 - \frac{L}{S}$$

Since S = kS'

$$k = 1 - \frac{L}{kS'}$$

Solving for k,

$$k = \frac{1}{2} + \frac{1}{2}\sqrt{1 - \frac{4L}{S'}}$$

ignoring negative square root. It has been found in practice that more accurate results are obtained by replacing the 4 by 5.72. Therefore:

$$k = \frac{1}{2} + \frac{1}{2}\sqrt{1 - \frac{5.72}{S'}}$$

Correction in Terms of Density

Since density is 5280/S, and since L may be taken to be 17 ft, the following is an alternative formula for k:

$$k = 1 - \frac{17D}{5280} = 1 - \frac{D}{311}$$

Thus, counts lost are directly proportional to true density.

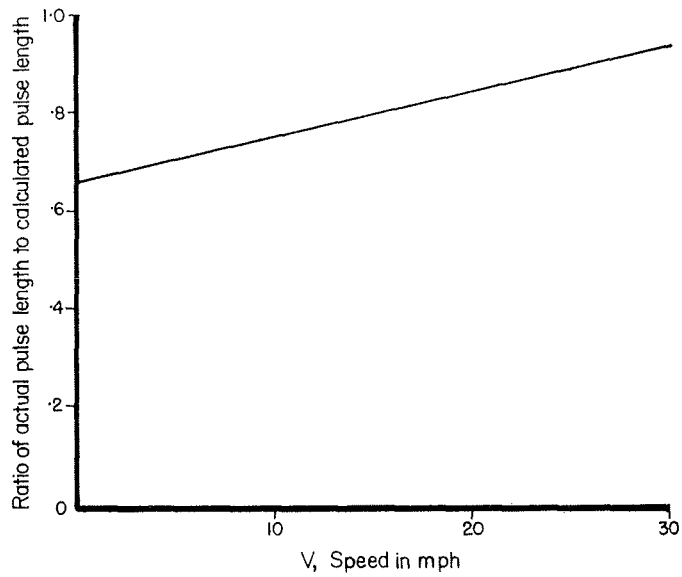

Figure A-3. Alternative detector calibration.

Formulas for Speed

Figure A-3 shows one form of the detector calibration curve. It is really a comparison of the detector used with a perfect detector. A perfect detector would simply give a straight line at unity. From Figure A-3:

$$L = 17\ (0.66 + 0.00933V)$$

Since $t'v = L$ and $v = {}^{44}/_{30}\ V$, it is possible to solve for v and V' and get

$$v = \frac{11.2}{t' - 0.1083}\ \text{fps}$$

$$V = \frac{7.65}{t' - 0.1083}\ \text{mph}$$

Appendix B

VDCA PROGRAM DESCRIPTION

Figure B-1 shows the flow chart for the VDCA program. In step one, choice of a_1 determines the shortest acceptable pulse, a_2 the longest, and a_3 sets the minimum acceptable spacing between pulses. In step two, the pulse lengths are converted to speeds for each two successive pulses, and an average speed for the two is obtained. In step three, the time between the pulses is found. In step four, the space headway is found by multiplying speed and time headway. From here the program branches to single lane or double lane.

Single Lane

Step five finds the arithmetic mean of all headways from step four. Step six finds the arithmetic mean speed of all speeds from step two. Step seven calculates the

FIGURE B-1. VDCA PROGRAM FLOW CHART

Pulse p_n occurs at time T_n seconds, and is t_n seconds long.

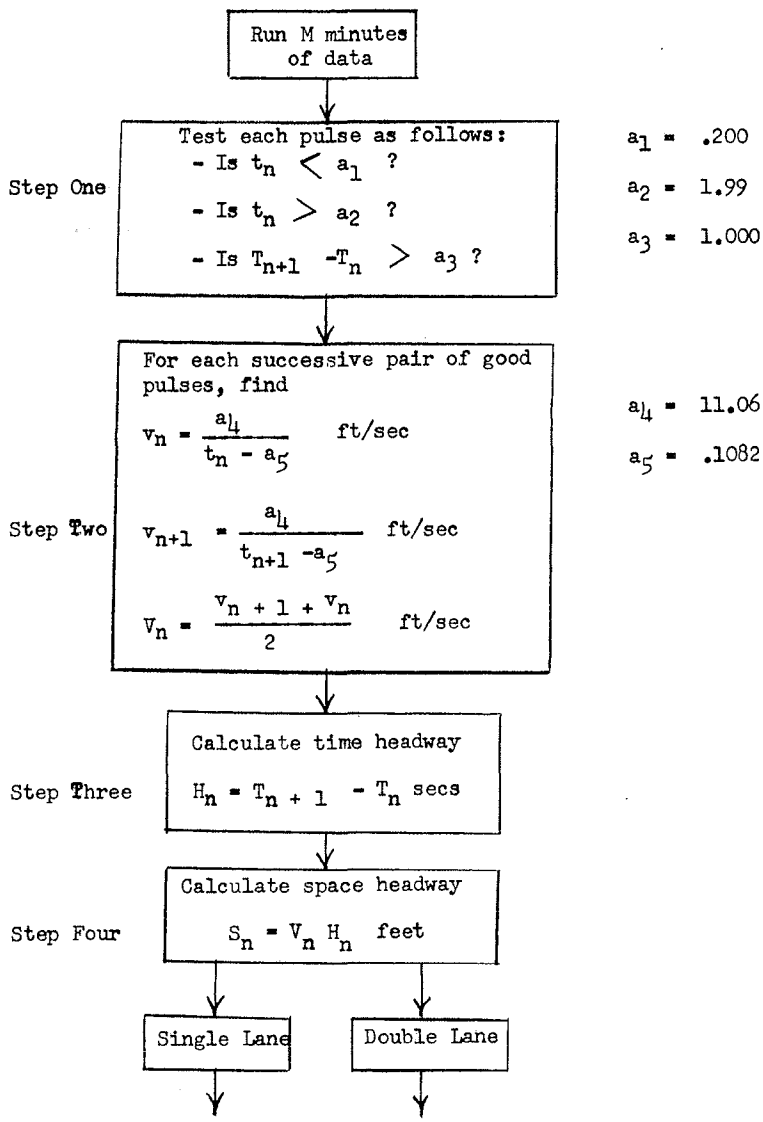

FIGURE B-1 (continued).

Single Lane

Step Five

Calculate arithmetic mean space headway for the time M

$$S' = \frac{\sum S_n}{N} \text{ feet}$$

where N is number of good pairs of pulses.

Step Six

Calculate arithmetic mean speed for time M

$$V = .682 \frac{\sum V_n}{N} \text{ mph}$$

Step Seven

Calculate density

$$D = \frac{5280}{S'} \text{ vehicles/lane/mile}$$

Step Eight

Find $\sum c$, total number of pulses, good or bad, in time M.

Step Nine

Find counts per hour per lane

$$K = \frac{60 \sum c}{M} \text{ vehicles/hour}$$

Step Ten

Find mean time headway

$$H = \frac{\sum H_n}{N}$$

Print:

Detector number, lanes, uncorrected counts per hour (= K for single lane),

V, correction factor (= 1 for single lane), D,S',H.

FIGURE B-1 (continued).

Double Lane

Step Eleven

Calculate arithmetic mean space headway, per lane, for time M.

$$S' = \frac{2 \sum S_n}{N}$$

where N is number of good pairs of pulses in M.

Step Twelve

Calculate arithmetic mean speed for time M.

$$V = .682 \frac{\sum V_n}{N} \text{ mph}$$

Step Thirteen

Calculate measured length of car

$$L = 17 (a_6 + a_7 V)$$

$a_6 = .66$

$a_7 = .00933$

Step Fourteen

Calculate $\frac{5.72L}{S'}$

Step Fifteen

Is $\frac{5.72L}{S'} \leq 1$?

If > 1, set = 1

Step Sixteen

Find correction factor

$$k = 1/2 + 1/2 \sqrt{1 - \frac{5.72L}{S'}}$$

Step Seventeen

Find $\sum c$, total number of pulses, good or bad, in time M

Step Eighteen

Find measured counts per hour

$$C' = \frac{60 \sum c}{M}$$

FIGURE B-1 (continued).

Step Nineteen	Find corrected counts per hour, per lane $K = \frac{C'}{2k}$
Step Twenty	Find corrected space headway $S = kS'$ feet
Step Twenty-one	Calculate density $D = \frac{5280}{S}$ vehicles/mile
Step Twenty-two	Calculate mean time headway $H = \frac{2 \sum H_n k}{N}$ seconds

Print:

Detector number, lanes, C', V, k, K, D, S, H

density. Step eight finds the total number of pulses, good or bad. Step nine converts the total number of pulses found in step eight to vol/hr/lane. Step ten finds the arithmetic mean of all headways found in step three.

Double Lane

Step eleven calculates measured space headway per lane. Step twelve calculates arithmetic mean speed in mph for use in step thirteen. Step thirteen uses the calibration curve to find measured vehicle length. Steps fourteen, fifteen and sixteen calculate the correction factor. Step seventeen finds the total number of pulses, good or bad. Step eighteen converts measured counts to hourly volume, per approach. Step nineteen finds corrected volume, veh/hr/lane. Step twenty finds corrected space headway. Step twenty-one finds density. Step twenty-two finds corrected mean time headway.

Speed of the Program

The program is written in FORTRAN IV. The limit to the speed is set by the time it takes to find the data on the tape. In the worst case, when four detectors (one intersection) are being done, ratio of real time to computer time is 6 to 1. The number of detectors which can be processed at one time is 100. This takes little more time than doing four detectors.

Appendix C

PREPARATION OF CALIBRATION CURVES

As explained in the body of the report, time headways are calculated only for the vehicles which are in motion over the detector. Any vehicle which stops on the detector is not counted. Table C-1 shows figures for the conditions prevailing on the east leg of Bayview-Eglinton. All but the starred figures have been calculated using Webster's formulas. It is found that up to a volume of 600 veh/hr, no vehicles stop on the detector. Therefore, time headways are simply 3600 sec divided by the volume. However, above 600 veh/hr, a certain number of vehicles form a queue past the detector, and it is necessary to find the time during which vehicles are in motion. This is done by drawing trajectory diagrams.

TABLE C-1

VALUES FOR 0.42 SPLIT, 90 SEC CYCLE, RELEASE RATE 2.0 SEC PER VEHICLE

Volume (veh/hr/lane)	Delay (sec/veh/lane)	Queue (veh/lane)	Time Headway (sec/veh/lane)
250	21.2	3.6	14.4
300	22.1	4.4	12.0
350	23.1	5.1	10.3
400	24.3	5.8	9.0
450	25.7	6.5	8.0
500	27.8	7.5	7.2
550	31.1	8.7	6.5
600	38.5	10.8	6.0
650	67.0	16.8	3.2[a]
672	147.8	32.5	2.5[a]

[a]Calculated from trajectory drawings.

The trajectories for a volume of 650 veh/hr are shown in Figure C-1. A volume of 650 is about 16 veh/cycle. The Webster formula states that the queue at the end of red is 16.8 vehicles, say 17. It is therefore necessary to produce trajectories such that 16 vehicles enter per cycle, and the queue at the end of red is 17 vehicles. The first step is to produce a line representing the motion wave as vehicles start from the queue. It has been found in our studies that for Toronto this has a speed of 100 ft every 6 sec. Thus, draw from the origin (start of cycle) lines of this slope. Next, mark out along this line the position of each vehicle in the queue. We chose a spacing of 22 ft, representing a jam density[1] of 240 veh/mi/lane.

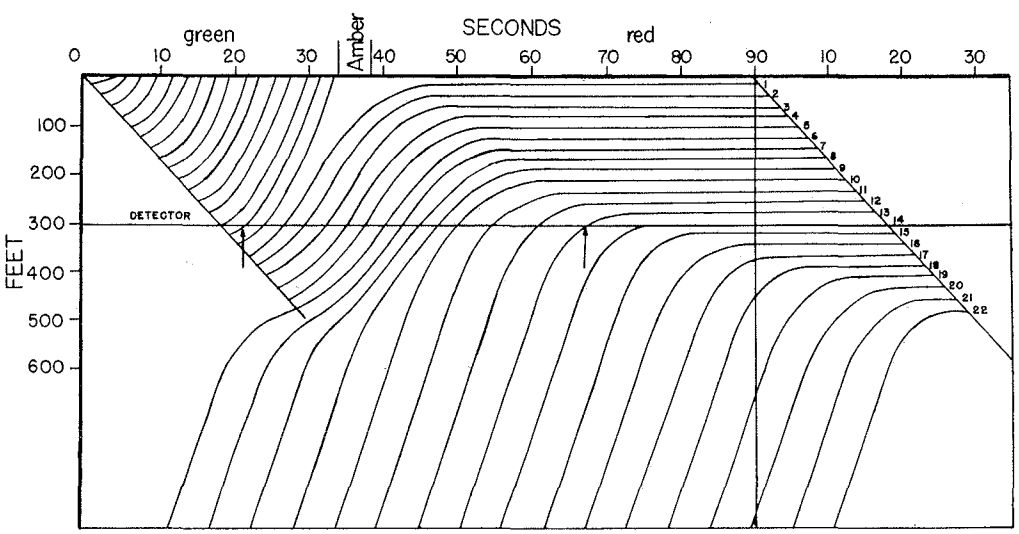

Figure C-1. Drawing of trajectories.

[1]Jam density is that density in traffic flow which occurs when all vehicles have been forced to stop.

The release points along the time axis occur every 2 sec, starting at 3.5 sec. Once these points are marked, trajectories can be drawn in for those vehicles released every cycle. For the remainder of the vehicles, a deceleration rate of about 4 ft/sec/sec has been chosen. The time of arrival of each incoming vehicle can be found by using the fact that the 17th vehicle is the last in the queue at the end of red, and the fact that deceleration is 4 ft/sec/sec. Once the 17th vehicle is found, the remaining vehicles are spaced out to cover the cycle. A speed of 30 mph as been chosen for the speed of incoming vehicles. Uniform spacing is allowed as an approximation to random arrival, since the VDCA method averages over many cycles. The remaining parts of the trajectories can now be drawn in. They need only be approximate, since their form does not alter critically the calculation of headway.

It will be seen from Figure C-1 that the 14th vehicle is stopped over the detector. Therefore, the time during which vehicles are in motion over the detector is that time between the arrows. In this case it represents 45 sec. Since there are 14 vehicle intervals, the average time headway is $^{45}/_{14}$ or 3.2 sec.

For the remaining volume in Table C-1, it may be assumed that the queue is so long that release over the detector is practically the same as that at the stop bar. Assume therefore that the time over which vehicles are in motion is about 35 sec. There are again 14 vehicle intervals, and headway is therefore $^{35}/_{14} = 2.5$ sec. The values given in Table C-1 are those shown in Figure 7 of the main body of the report.

The Effects of Street Geometrics and Signalization On Travel Time and Their Relationships to Traffic Operations Evaluation

J. F. TORRES, Cornell Aeronautical Laboratory, Buffalo

The effects of street geometrics and signalization are discussed in terms of travel time, which is defined as the time of travel through a street section averaged over all drivers and all specified time periods within a prescribed class. This is the key factor in the evaluation of traffic operations. Travel time is shown to be significantly and reliably related to volume, given specific street section characteristics.

An extensive sample of urban arterial street field survey data collected through a collaborative effort of state and local agencies, is employed to study the dependence of the travel time-volume relationship to the geometrics and traffic control factors. The principal variance-producing factors in the study sample are identified. A major result of the study is the determination of a set of general prediction curves by means of which the travel time-volume relationship can be estimated from knowledge of the characteristics of given specific streets. The application of the travel time results to the evaluation of traffic operations is indicated.

•THE need for a means for objectively, reliably, and practically evaluating traffic operations on streets and highways has been generally recognized by transportation and traffic engineering circles. The availability of soundly based evaluation procedures would permit traffic engineers and administrators to make rational judgments on the performance of streets and highways. They would be able to weigh the benefits of, say, widening a street, changing the signalization, or controlling parking. They would be able to estimate the effects of variations in the volume of traffic. The operational performance of streets and highways could be predicted with respect to future traffic demands. All such estimates and predictions should be made on the basis of simple measurements made by field personnel in order to be operationally acceptable.

We have performed a research study (1) that is directed toward this problem. We have developed a traffic operations evaluation procedure, which we feel is objective, reliable, and, what is considered to be extremely important, practicable. Our attention has been focused on the class of arterial streets, since they have a greater number of factors affecting performance and since they carry the great bulk of traffic.

The determination of the operational performance of any operational situation requires the utilization of an appropriate measure. Such a measure, in this particular case, should reflect the true operational performance of streets and highways. And, to be efficient, such a measure should be capable of providing consistent, unbiased, and sensitive estimates of the street operational performance. Driver satisfaction has formed the keynote for the study, and hence the measure is postulated to contain the most significant individual contributions to the streets' performance which bear on driver satisfaction. Further, the measure must not only reflect the differences among

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streets, characterized by such factors as geometrics and signalization, but also must be directly sensitive to the traffic experienced. Certainly, the performance of a street will depend on the volume of traffic.

METHODOLOGY

The Measure of Traffic Performance

The measure of traffic performance postulated in this study is structured in terms of the significant driver dissatisfaction factors, and for a given length of street takes the form:

$$M = \alpha_1 x_1 + \alpha_2 x_2 + \alpha_3 x_3 + \alpha_4 x_4 \tag{1}$$

where

x_1 = travel time,
x_2 = driver discomfort,
x_3 = driving hazards, and
x_4 = direct vehicle running costs.

Each of the driver dissatisfaction factors is required to have an operational indicator, i.e., a measurable traffic variable that will allow the convenient estimation of the factors. Traffic volume has been selected as the common measurable traffic variable to employ for the four factors.

It was initially hypothesized that the expected value (conditional on traffic volume) for each driver dissatisfaction factor for an individual vehicle would be given by:

$$y_i = a_i + b_i v \qquad i = 1, 2, 3, 4 \tag{2}$$

where v represents directional (say, 15-min) volume, and a_i and b_i are constants corresponding to the physical characteristics of a given street section type. Experimental observations subsequently showed, through a statistical analysis, that Eq. 2 adequately represents the relationship between the individual driver dissatisfaction factors and traffic volume for at least one of the factors (travel time) for the range of volumes that span roughly the time period between 7 a.m. and 7 p.m. The observation periods thus excluded the very low-traffic hours.

Since it is of interest to estimate the net loss of benefits for the population of drivers using a given street, it is necessary to weight the y_i by the corresponding volumes. The average of these weighted values, with respect to the volume distribution is then given by:

$$x_i = E\{v y_i(v)\} \qquad i = 1, 2, 3, 4 \tag{3}$$

The x's (cf. Eq. 1) thus represent the statistical average, over the relevant distribution of traffic volumes, of the driver dissatisfaction factors corresponding to the population of drivers that use the street in the time period over which the volume was measured (say, 15 min). The α's in Eq. 1 are the unit costs or values associated with each of the driver dissatisfaction factors.

The performance measure, M, thus provides a direct operational indication of actual driver satisfaction (or dissatisfaction). It is emphasized that M employs explicitly the important factors that traffic engineers normally use in gaging street performance, plus others that research has determined to have significant value. The four component factors in M were researched in this study to different degrees. However, the most comprehensive investigation was performed on travel time. The results presented subsequently will be confined to this factor.

Equation 2 can be illustrated (Fig. 1) for the travel time factor. This relationship is basic to the study. Eq. 2 demonstrates the effect of volume on the driver dissatisfaction factors (e.g., travel time) on a facility with given street characteristics (e.g., given geometrics and signalization).

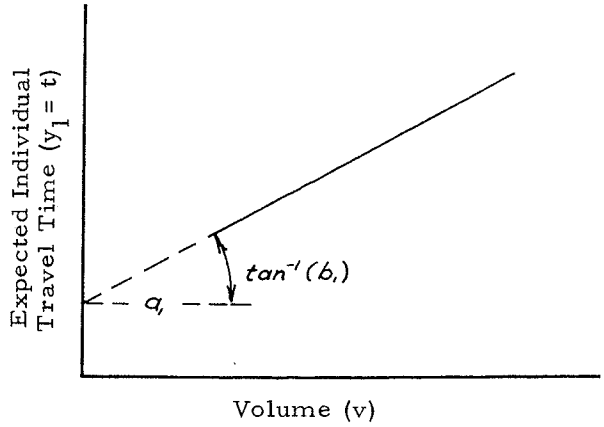

Figure 1. Typical factor relationship.

The Basis for Practical Procedures

The form of the forestated equations (e.g., Eq. 2) suggests that the effect of geometrics and traffic control will enter through the coefficients a_i and b_i. The effect of volume, of course, will enter through the variable v. Ideally, in an evaluation situation, given the significant street characteristics for a specific street section, the corresponding coefficients a_i and b_i would be selected from a predetermined set. And, subsequently, given the traffic volume distribution for the street section, the performance measure M would then be obtained.

The problem of developing an adequate evaluation basis and procedure then resolves to that of determining a way of simply and reliably relating the coefficients a_i and b_i to the significant street characteristics for sufficiently broad classes of streets. The set of such relationships would form the basis for evaluation. It is crucial, however, for the reliable determination of street operational performance that the relationships y_i (or equivalently the a_i and b_i) be consistently estimated for given street sections, once the geometrics and traffic control characteristics for the given street are identified. Thus, two different street sections with the same street characteristics would be expected to have the same slope b_i and the same intercept a_i. On the other hand, two street sections with widely different characteristics would be expected to have significantly different slopes and intercepts.

Accounting for the Variation

Many factors affect the variation of operational performance (characterized, for example, by travel time). This has led many to believe that the problem of obtaining reliable general estimating relationships is an impossible task. However, it is reasonable to expect that some factors will have a much greater effect on the variations than others. An initial problem then was the identification of these more influential factors. The approach for determining a set of general prediction relationships for the coefficients required the implementation of the following steps:

1. The identification of the significant variance-producing geometrics and traffic control factors.
2. The classification and structuring of streets with respect to the most significant factors.
3. The validation of the hypothesis that representative correspondence relationships can be obtained for street classes.
4. The determination of the general prediction relationships (coefficients vs street characteristics) for each of the driver satisfaction factors.

A statistical investigation is implicit in these four steps. Adequately comprehensive and adequately measured data corresponding to the individual driver satisfaction factors had to be employed. It is recalled that the evaluation measure (Eq. 1) is in terms of four driver satisfaction factors of which the most important is believed to be travel time.

THE TRAVEL TIME FACTOR

Travel time received the major effort from the overall program and has provided some of the more interesting results, some of which are believed to be directly applicable to the suboptimal evaluation of streets which fall within the scope of the study sample. The discussion of the travel time factor will also serve to illustrate the manner in which the other driver satisfaction factors could be treated.

The detailed investigation of the proposed methodology required comprehensive sets of travel time, volume, and street inventory data. A search conducted among various agencies early in the study yielded only two existing sets of data which were felt to be adequate for the study. One of these was from the Chicago Area Transportation Study. The other was from the Pennsylvania Department of Highways. The analysis of these preliminary sets of data established the feasibility of identifying the significant variance-producing factors, thereby obtaining reliable travel time and volume relationships. However, the results obtained were based on samples of data which were limited in size as well as limited geographically. But after the favorable results obtained from these initial studies, the way appeared to be clear for a more extensive investigation using more comprehensive data purposely gathered for this study.

The Data Set

A quite extensive set of travel time, volume, and street inventory data was subsequently collected in a planned survey through a substantial collaborative effort of several local and state traffic engineering and planning agencies. The collaborating agencies who participated in this research study are given in the Appendix. The data were collected following the preparation of careful experimental plans after extensive and intensive discussion with staff personnel of the various localities. The careful preparation served to guarantee a more efficient payoff with respect to the effort expended. Further, close coordination was maintained with the agencies throughout the conduct of the field surveys.

The overall set of data is composed of seven basic subsets which correspond to the following plans: Albany area street plan, Baltimore street plan, Buffalo street plan, Dallas street plan, Detroit street plan, Pittsburgh street plan, and San Diego street plan. Urban arterial street sections were selected for each plan which carried relatively heavy volumes and which were relatively homogeneous throughout the length of the test section. The total sample is composed of 158 street sections. Each section has an average of 40 to 50 paired observations of travel time and volume per direction. Travel time was measured by an adaptation of the floating-car method. Volumes were measured over 15-min intervals for each street section.

There are four general types of signal system types which describe the street sections in the data sample. These are: (a) pre-timed coordinated, (b) progressive, (c) traffic-actuated, and (d) pre-timed non-interrelated. The technical nature of these signal systems is distinct enough so that it was expected that different response relationships would hold for each of these types.

Methodology of Analysis

The response (or analysis variable) used for the study was the paired set of observations (slope, intercept) for the travel time-volume relationship for each street section. As indicated earlier, these two coefficients were hypothesized to be a function of the street factors, such as geometrics and signalization. Several specific factors had been initially indicated to significantly affect the variation of travel in urban areas. Among the most prominent of these were: (a) traffic volume (v), (b) pavement width

(w), (c) signal density (s), and (d) speed zoning (z). Travel time (per mile) is then related to all these factors in terms of the equation:

$$t = a + bv \tag{4}$$

where

$a = a(w, s, z, \ldots.)$
$b = b(w, s, z, \ldots.)$

that is, a linear relationship between t and v, where the regression coefficients are functions of the factors. For each street section (in each direction of flow) the 50 odd points (t, v) were then fitted with the best fitting line.

Significantly, it was found that the intercept could be well estimated by the speed zoning or speed of progression factors in most cases. In the remainder of the cases, such as streets with non-interconnected signals, it could be estimated by the lowest valued travel time observations. This finding enabled a sharpening of the analysis, since it implied that the slope coefficient is the variable primarily affected by the remaining street factors. The subsequent analyses are then based on the relationship between the slope b and the street factors. The variance of the observations around the regression lines is also investigated.

ANALYSES AND RESULTS

The mathematical analysis of the travel time data was performed in three steps after the identification of an initial set of potential variance-producing factors: (a) the analysis of variance of the factors, (b) test of homogeneity of the street sections classified according to the factors, and (c) determination of the general relationships that hold for broad street classes.

Analysis of Variance—Albany Area Data

An analysis of variance was performed on two factorial experimental designs that employed the Albany area data. The objective was to determine which factors were significant as well as the degree of significance. The analysis of one of these designs (which is representative of the other), a ¼ replicate of a six-factor experimental design, is given in Table 1 for:

Factors	Criterion Points
A. Number of lanes	3
B. Moving lane width (ft)	11
C. Signals per mile	3.5
D. Percent green time	50
E. Intersections per mile	10
F. Parking	one side

Each factor is taken at two levels. The criterion points indicating the levels of the factors are shown in parentheses after each factor. Values of the individual factors greater than the criterion point constitute one level, and values less than the criterion point constitute the second level. The factor interactions indicated by "e" in the analysis of variance table have been employed as the estimate of the error.

The fractional replicate given in Table 1 was designed in such a way that the indicated interactions could be estimated with little likelihood of confounding by other factor effects. The analysis of variance of this six-factor experiment indicates that all of the main effects are significant at least at the 10 percent level, with the exception of factor E, intersection density. In fact, signal density C is significant at the 0.1 percent level, which may be instrumental in the significance of the interactions AC and ACF.

TABLE 1

ANALYSIS OF VARIANCE FOR QUARTER REPLICATE DESIGN FOR SIX FACTORS, ALBANY AREA DATA

Section No.	Effect Estimated	Factor Values						Ratio of Mean Squares (F)
		A	B	C	D	E	F	
14	T	3	6.7	21.05	41	10.53	2	-
37	A	4	10.3	6.45	46	6.45	1	7.42[b]
20	B	2	11.0	3.51	53	15.79	2	5.17[a]
31	AB + CE	4	11.3	3.13	50	14.06	1	e
75	C	2	10.0	0.88	50	6.61	0	48.69[d]
33	AC	4	11.3	0	0	15.63	1	19.83[c]
69	AE	2	12.0	0	0	11.54	0	16.85[c]
41	E	4	11.3	3.45	42	6.90	1	NS
42	D	2	13.3	3.70	51	16.67	1	12.77[b]
25	AD + BF	4	10.0	4.08	59	7.14	2	e
22	BD + AF	2	11.0	13.33	56	10.00	2	4.92[a]
46	F	4	11.0	3.45	59	10.34	0	8.54[b]
76	CD	2	10.0	0.56	60	6.18	0	e
27	ACD	4	9.0	1.05	60	4.71	0	e
62	ACF + BCD	2	12.0	1.32	60	10.60	2	19.71[c]
77	DE + CF	4	13.5	1.80	58	7.21	0	e

Defining Contrasts: {I, ABCE, ABDF, CDEF} (2)

Slopes normalized to a per-lane basis for compatible comparisons.

[a]Significant at 10 percent level.
[b]Significant at 5 percent level.
[c]Significant at 1 percent level.
[d]Significant at 0.1 percent level.

The results of the analyses of variance of these two factorial designs suggest that signal density has a very significant effect on the variation. To a lesser degree, road width (or, equivalently, number of lanes and lane width), percent green time, and parking also have an effect. The effect of intersection density seems dubious. The weak effect indicated for parking may be partly due to the fact that it was not possible to identify from the data which street side had parking for those that allowed parking on only one side.

Factorial Design Analysis—Detroit Data

Of further interest is the analysis of the significance of the factor effects in the full-factorial experimental design for three factors, which employs the Detroit set of data. This set of data, it is recalled, is derived from a set of sections of a high type of arterial with a progressively phased signal system (Detroit street plan). The three factors considered for this study are street section, signal split, and number of lanes.

The analysis of variance of this factorial design is summarized in Table 2. The analysis was performed on both the slopes and on the standard errors. Furthermore, each of these was analyzed for both the favored direction of flow (signal-wise) and for the counter-favored direction. Thus, four analyses are shown in the table. The analysis shows that there is no significant difference in slopes between the two sections (factor A) in the favored direction. Going from a 50 to 30 split to a 45 to 35 split produced an increase in the slope which is significant at the 15 percent level. An increase in the slope, significant at the 5 percent level, was produced by going from 4 lanes to 3 lanes. In the counter-favored direction, all the main effects for the slopes become significant.

The analysis of variance on the standard error shows that signal split is significant at the 1 percent level along the favored direction. No other factor is significant for both the favored and counter-favored directions. Number of lanes and signal split are shown to produce a significant effect, whereas the difference between sections is not significant.

TABLE 2

ANALYSIS OF VARIANCE OF FULL FACTORIAL EXPERIMENTAL DESIGN FOR THREE FACTORS, DETROIT DATA[a]

Section No.	Effect Est.	Slopes (min/veh-mi)				Standard Error (min/mi)			
		$b \times 10^5$	Avg. Effect	Mean Square	F	$\sigma \times 10^3$	Avg. Effect	Mean Square	F
(a) Favored Direction									
8	T	107	-	-	-	349	-	-	-
6	A	167	20.5	841	0.57	394	11.3	253	0.05
4	B	104	-29.0	1,682	1.14	297	-103.8	21,528	5.2[c]
2	AB	125	-17.0	578	e	297	-31.8	2,016	e
7	C	85	-51.0	5,202	2.85[b]	347	-33.3	2,211	0.54
5	AC	100	-20.0	800	e	388	-11.3	253	e
3	BC	64	-6.5	85	e	255	-29.3	1,711	e
1	ABC	50	2.5	13	e	214	-9.3	171	e
(b) Counter Favored Direction									
8	T	211	-	-	-	363	-	-	-
6	A	335	118	27,613	3.66[b]	329	-8.3	136	0.05
4	B	184	-105	21,841	2.90[b]	304	-14.3	406	0.16
2	AB	270	-7	98	e	330	31.8	2,016	e
7	C	358	89	15,842	2.10[b]	312	-29.8	1,770	0.71
5	AC	483	13	313	e	291	-4.3	36	e
3	BC	190	-59	6,845	e	279	14.8	435	e
1	ABC	325	12	288	e	325	1.8	6	e

[a]The two directions have been pooled together (they have been indicated to be homogeneous).
[b]Significant at 5 percent level.
[c]Significant at 1 percent level.

Multiple Regression Analyses of Factors

Four multiple regression analyses were performed on the Albany area set of data. Two of these analyses were performed on the two sets of streets sections which constitute the two aforementioned factorial designs. The other two multiple regression analyses were made for: (a) the set of 2-moving-lane streets and (b) the set of 4-moving-lane streets, after first removing from consideration all those streets where the responses were significantly different (at the 1 percent level) between directions. In each case, the slope responses were regressed on the indicated independent variable.

Table 3 summarizes the results of these analyses. Signal density can be seen to be the most significant factor. Road width is also indicated to have a strong effect. Intersection density and parking are indicated to have a possible effect. These analyses reinforce the significance of the signal density factor and the road-width factor. The parking factor is once again indicated to have a possible effect.

Analyses of covariance were performed on the four sets of data after first arbitrarily dividing each set into two subsets for comparison purposes. The hypothesis that the two multiple regression equations for each pair are essentially the same could not be rejected, at about the 10 percent level of significance, which suggests that the employed factors may be explaining most of the variation. However, this may be partially because the observational sets have a large amount of variation. As a consequence, care would have to be taken in applying the multiple regression equations for prediction purposes.

Tests for Homogeneity

The analyses of variance discussed in the foregoing pages have singled out the factors that are most likely to introduce significant variation. Signal density appears to have an overwhelming effect on the variation. Pavement width and parking are also significant, but to a lesser degree. Strong contrasts are obtained between directions on the progressively signalized street sample. On the basis of the significant factors, which includes type of signalization, the street sections from the various samples were aggregated into groups which were considered to be relatively similar. These

TABLE 3

MULTIPLE REGRESSION ANALYSES OF GEOMETRIC AND CONTROL FACTORS, ALBANY AREA DATA

Variables	Coefficient	Standard Error	t-value	Mean Value
(a) First Factorial				
Half-width	-529	257	-2.06[a]	20.7
Signal density	1279	390	3.28[a]	4.6
Percent green	24	98	0.24	61.2
Intersection density	917	352	2.61[a]	9.0
(b) Second Factorial				
Number of lanes	-2821	947	-2.98[b]	3.1
Lane width	813	763	1.07	10.9
Signal density	1478	258	5.73[b]	4.3
Percent green	-6	65	0.09	59.0
Intersection density	231	317	0.73	10.0
Parking	519	1436	0.36	0.9
(c) 4-Lane Set				
Half-width	248	422	0.59	26.0
Signal density	-113	534	-0.21	3.8
Percent green	-89	79	-1.13	60.0
Intersection density	-47	391	-0.12	7.7
Parking	3642	2106	1.73	0.9
(d) 2-Lane Set				
Half-width	-476	417	-1.14	17.0
Signal density	1528	498	3.07[b]	4.3
Percent green	-131	102	-1.29	52.5
Intersection density	601	317	1.90[a]	10.7
Parking	2618	2536	1.03	1.3

[a]Significant at 5 percent level.
[b]Significant at 1 percent level.

groups, which have similar geometrics and traffic control characteristics, were conjectured to have similar responses. At the same time, some control groups were aggregated which were expected to have dissimilar responses.

Analyses of covariance (ANOCOV) were then performed on this large set of groups to test whether the responses for each group were similar (or dissimilar as the case may be). Generally, the results strongly confirm the hypothesis that uniform responses are obtained, once the street sections for each group are selected with similar factor characteristics. Furthermore, street sections aggregated with dissimilar characteristics give nonuniform responses. These findings strongly suggest that no significant factors have been overlooked, and all other factors are likely to have a minor effect.

The results of these analyses of covariance thus tend to strongly confirm that the major variance-introducing factors have been identified. The way now appeared to be prepared for the final and key step, which was to determine the actual effect of the factor levels on the responses. Of course, this would lead to general prediction relationships. The development of these relationships is presented in the sections that follow. But, first, the effect of signal density on the standard error is discussed.

SIGNAL EFFECT ON THE STANDARD ERROR

Signal density has been strongly indicated to have a significant effect on the slope responses. Hence, from the quantitative findings, as well as from theoretical considerations, a significant relationship would be expected between the standard error and the factor of signals per mile. This conjecture is confirmed by a plot of these two variables performed for the Albany area data (Fig. 2). Each point on the figure represents one street section. In particular, it is of interest to examine the relationship between the standard error and signals per mile for the arterial streets within the Albany area data set. Therefore, the mean line is determined for the points corresponding to arterial streets.

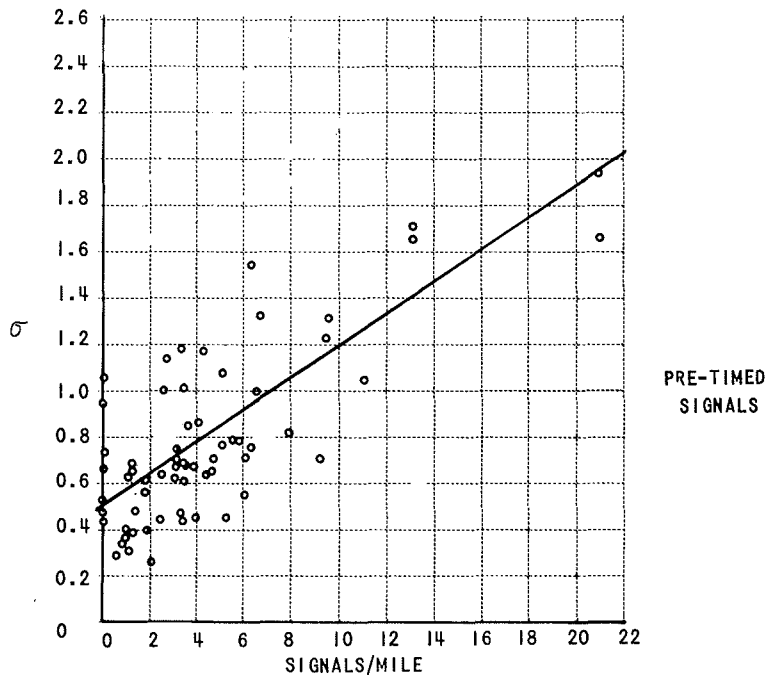

Figure 2. Relationship between standard error and signal density, Albany area data.

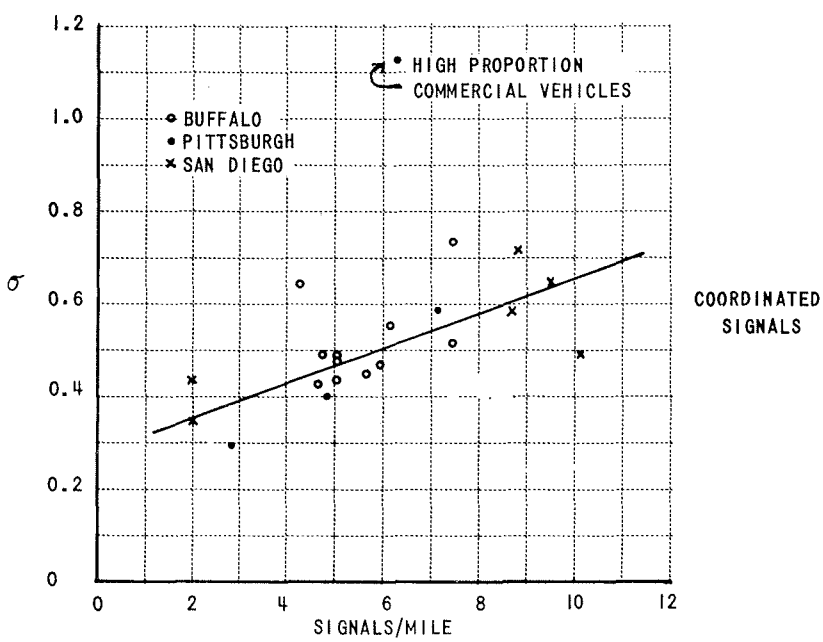

Figure 3. Relationship between standard error and signal density, pre-timed signals.

A comparable set of streets was selected from the data corresponding to the other localities; i.e., the selected streets were required to have pre-timed signal systems. The relationship corresponding to this set is shown in Figure 3. Here, again, the effect of signal density on the standard error is demonstrated. More significant is the fact that the points corresponding to the various localities are consistently distributed, with low variance, around the mean line (with the exception of one Pittsburgh section which had a large standard error attributable to a high proportion of commercial vehicles). This suggests stability in the measurement procedures.

Relationship Between Slope and Signal Density for Pre-timed Coordinated Signals

Streets with noncoordinated signal systems exhibit considerable variation in their responses. The traffic control factors peculiar to these systems, in combination with

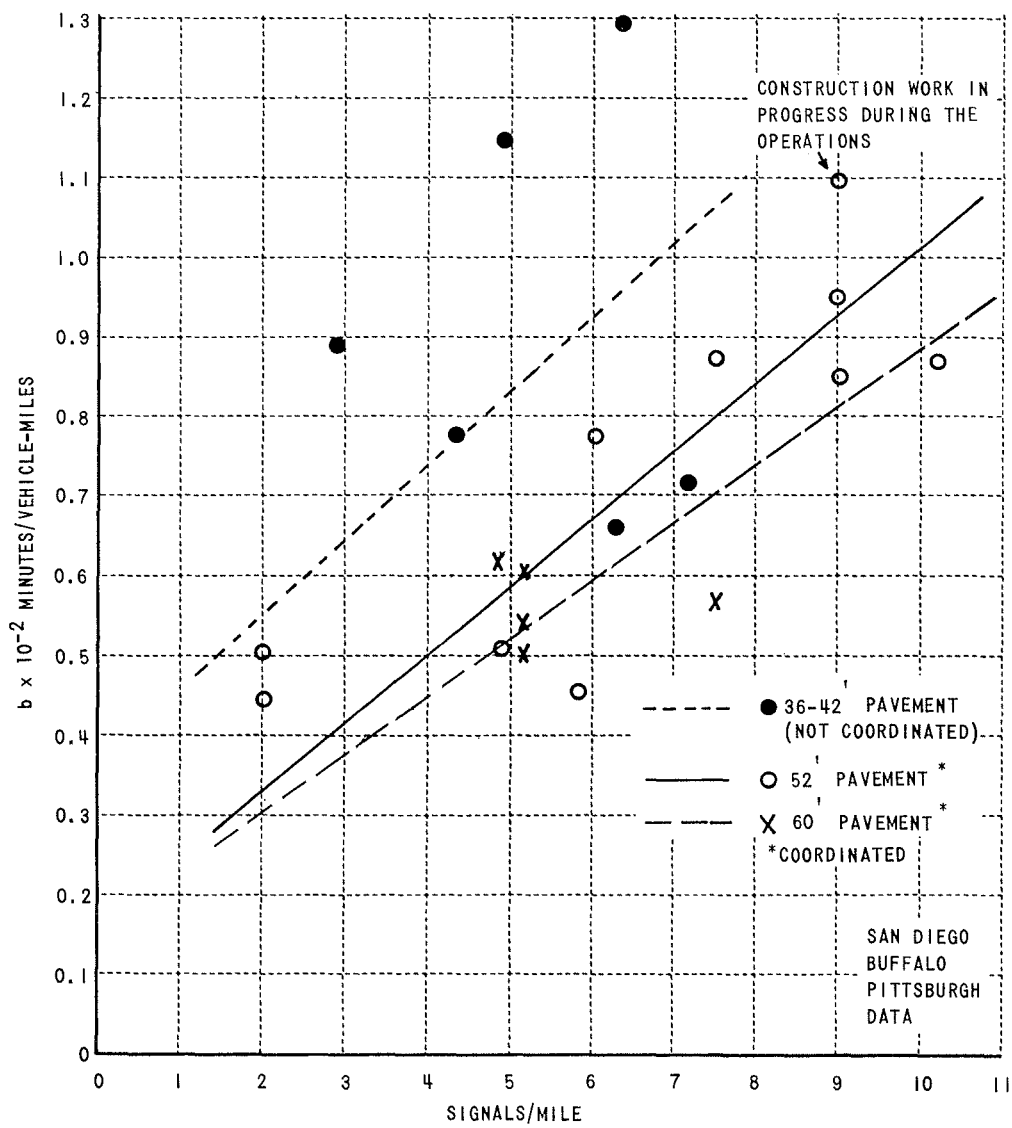

Figure 4. Relationship of slope to signal density.

the geometrics, tend to produce a significant effect on the responses. This variation compounds the problem of obtaining reliable estimates of the response through the use of street characteristics. Once the signals are coordinated, stable and predictable responses are then more likely to be obtained. Since signal density was found to be a very significant factor in the preliminary analyses, it was felt that an intensive exploitation of this factor would explain much of the variation and go a long way towards obtaining general prediction relationships.

The relationship between the slope coefficients and signal density has been examined for the set of streets with pre-timed signals from the Buffalo, Pittsburgh, and San Diego data. The data points (each point represents one street section) corresponding to these sets are shown in Figure 4. There are three groups of streets, corresponding to three pavement widths. The mean lines have been drawn through the corresponding sets of points for the 42-ft and the 52-ft pavement street sections. The 52-ft pavement set of points, the largest group, has a very significant relationship. It appears that most of the variation is explained by the signal density factor. The distribution of points for the 60-ft pavement set is indicated to fall slightly below the spread for the 52-ft pavement set (in particular, the centroid of the 60-ft set of points is located below the mean line for the 52-ft set). Since the observations for the 60-ft pavement set are clustered over a small range of signal densities, and since the mean line for this group is expected to be no worse than that for the 52-ft pavement set, the line through this set of points was obtained by hinging the line at the y intercept of the 52-ft pavement line. The data for the 36- to 42-ft pavement streets indicate that the corresponding line is translated upward at a steeper slope. This is to be expected since the number of moving lanes has been reduced from four to two. Considerably greater variation is also evident for this latter group, which is explained by the fact that the streets in this set did not have coordinated signal systems, in contradistinction with the other sets. A general relationship is thus evident between the slopes and signal density for streets with pre-timed, coordinated signals.

Further Confirmation From Another Set of Data

The relationship between the slopes and signal density has been further examined against the set of Baltimore data and the progressively phased streets from the San

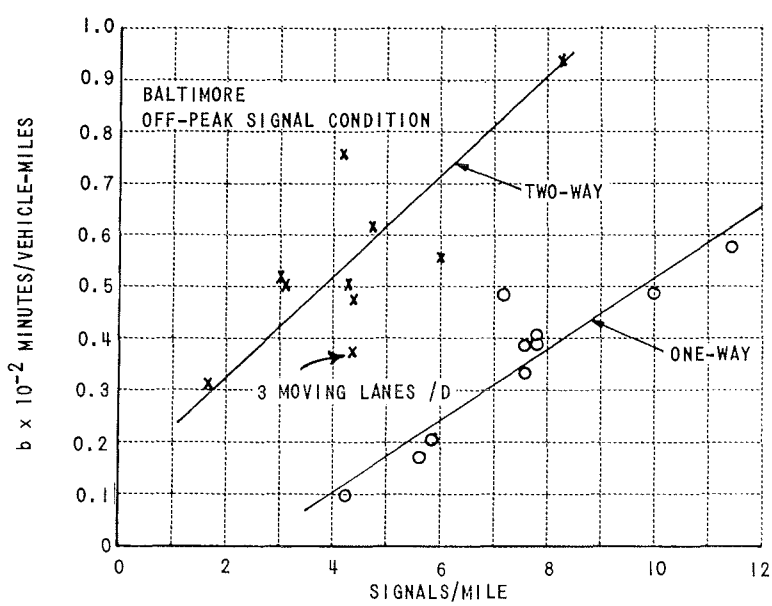

Figure 5. Relationship between slope and signal density, Baltimore—off-peak.

Diego set. The City of Baltimore has a traffic-adjusted signal system, where the progression and the cycle lengths are prescribed by the traffic conditions on strategic arterials. The signal parameters may thus vary throughout the day, which could imply difficulty of analysis.

An early inspection of the Baltimore raw data, however, revealed that the signal parameters, for most cases, can be blocked out into three distinct conditions that correspond to: a.m. peak (7a.m.-9 a.m.); off peak (9 a.m.-4 p.m.); and p.m. peak (4 p.m.-6 p.m.). During the a.m. peak, the signal progression favors the inbound traffic to the downtown area. During the p.m. peak, the signal progression favors the outbound traffic from the downtown area. The counter-favored direction suffers during the peak periods. During the off-peak period, average progression is employed where both inbound and outbound traffic have equal priority. The cycle lengths during the peak periods are typically 90 to 110 sec. For the off-peak period, the cycle lengths are typically 70 to 75 sec. By blocking out the data in the way indicated, the inferences for Baltimore are made comparable with those locations with pre-timed systems. However, this is not to say that the traffic-adjusted systems perform equivalently with the pre-timed systems. The traffic-adjusted system still has the advantages of flexibility and adaptability, which allow it to cope with variations in the traffic volume distributions.

The slope relationships were examined for the peak and for the off-peak conditions. The two peak periods (in the appropriate directions) were pooled into one group, since they constitute a symmetrical set. Further, since 1, 2, and 3-moving-lane streets were studied, the slopes were normalized in terms of 2-moving-lane streets. Figure 5 shows the relationship between the slope and signal density for two-way streets and one-way streets for the off-peak signal condition. Highly significant relationships are once again obtained, with a sharp distinction between the two sets. The distribution of points, corresponding to the two-way streets, can be superimposed right over the distribution already obtained for pre-timed signals. The mean line for the Baltimore set

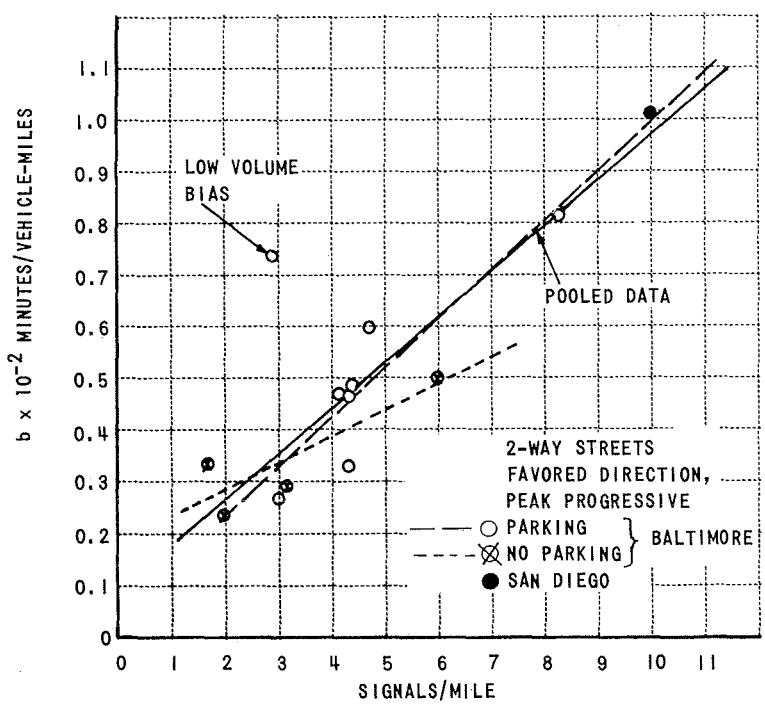

Figure 6. Relationship between slope and signal density, two-way streets.

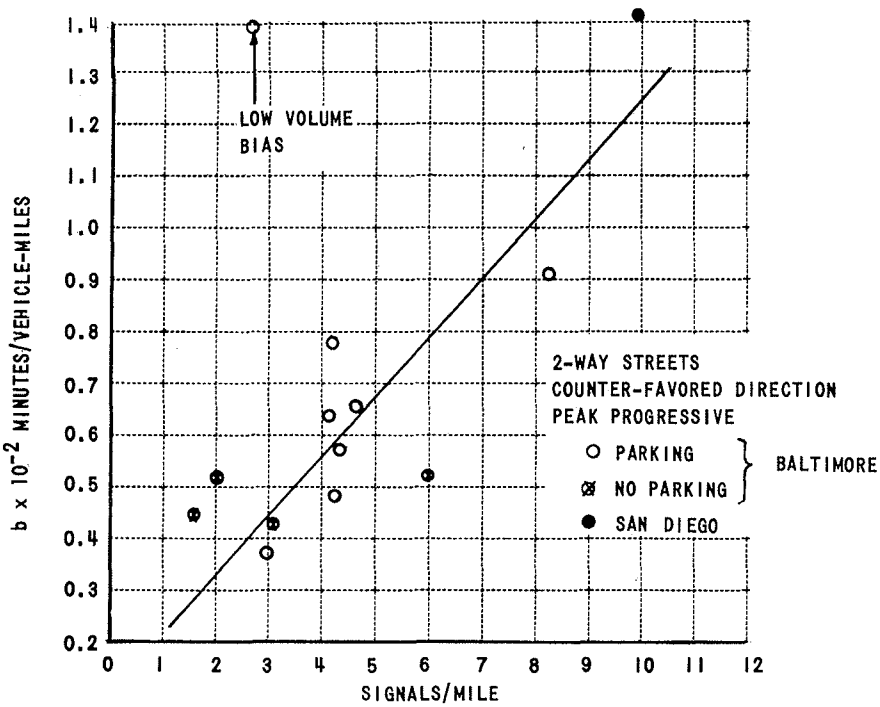

Figure 7. Relationship between slope and signal density, two-way streets.

coincides almost exactly with the mean line for the group of streets with pre-timed signals. Furthermore, the dispersion of the points around the mean line is also comparable for both sets.

The set of one-way streets (for the off-peak condition) is indicated to have very low dispersion around the mean line. The slope of the mean line through this set of points appears to be about the same as the two-way street set, but the intercept at the b-axis is considerably lower.

The relationships for the two-way streets for peak signal conditions are shown in Figures 6 and 7. Figure 6 shows the relationship along the favored direction of flow. Streets with parking or standing restrictions have been distinguished from those which do not have such restrictions, and a difference in the slope relationships is indicated. The relationship for the counter-favored direction is shown in Figure 7. Greater dispersion is evident for this direction and the mean line is displaced upward, as expected.

The relationships for the one-way streets for the peak signal conditions are shown in Figure 8. The slopes for the relationships are displaced downward from that for the two-way streets. The favored direction is shown to have a steeper slope than that for the counter-favored direction. Here, again, the data from the two different localities (Baltimore and San Diego) are consistent within the general relationship.

These results show strong relationships obtained between the slope coefficients for each street and signal density, for coordinated signal systems. The analyses have demonstrated that signal density has an overwhelming effect on the variation. Reliable relationships are obtainable once this factor is accounted for. The effect of locality has been shown to be not significant. Consistent relationships were obtained after pooling data from different cities. Pavement width (or, rather, lane width) and parking have been indicated to have an effect; however, the effect introduced by these factors is secondary. Cycle length and signal split are also secondary factors.

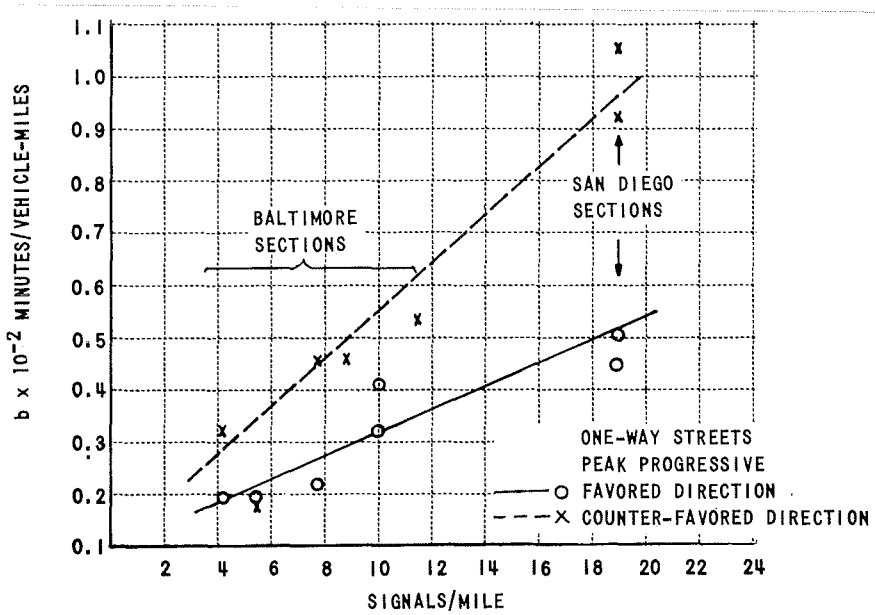

Figure 8. Relationship between slope and signal density, Baltimore and San Diego one-way streets.

The Effect of Signal Split

The Detroit street sections have been shown to have slope coefficients which are appreciably lower than those for the other streets considered. This appears to be due to the larger number of moving lanes (four in each direction during the peak periods), larger average lane width (10 ft), and the presence of a left-turning lane, besides the fact that a progressive signal system is employed.

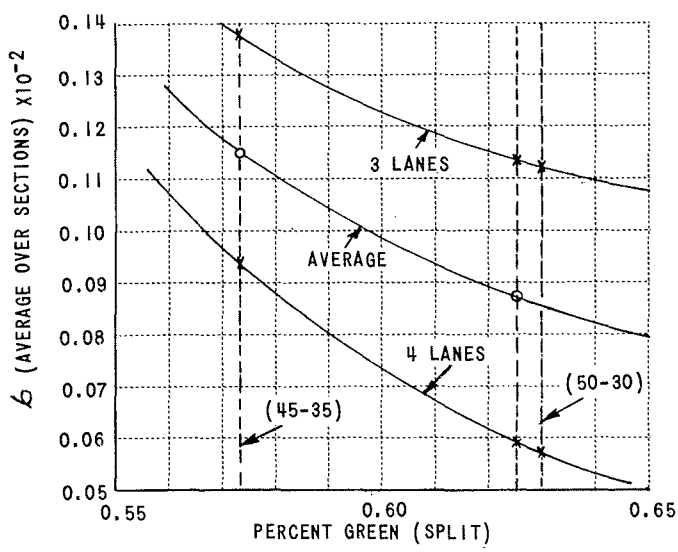

Figure 9. Relationship between slope and split, Detroit data—favored direction.

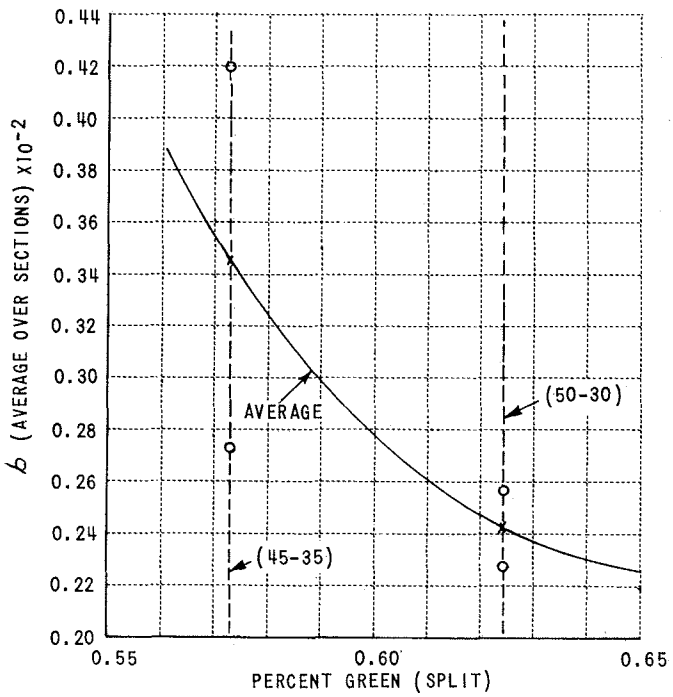

Figure 10. Relationship between slope and split, Detroit data—counter-favored direction.

For this set of streets, it was possible to conveniently and reliably investigate the effect of signal split. This effect is shown in Figures 9 and 10. The relationship between the slope b and signal split is expected to be a curve which is concave upward, since b can be seen to tend to infinity as percent green approaches zero, b can be seen to tend to some fixed lower bound as percent green tends to 100 percent, and the curve is expected to be continuous between the two extremes. It is noted for the favored direction case shown in Figure 9, that the 4-lane curve for the slope is indicated to be significantly lower than the 3-lane case. Further, the effect of signal split is evident. The relatively low volumes that are characteristic of the counter-favored direction do not allow a significant comparison between the 4-lane and 3-lane cases. The average curve through the pooled counter-favored set of points is shown in Figure 10.

In both the favored and counter-favored cases, signal split is shown to have a significant effect. A general relationship is indicated between the slopes and signal split. The favored direction curves appear to have a higher confidence relationship. The curves also show that the travel time loss in the counter-favored direction tends to increase more rapidly than along the favored direction as the signal split (percent green) is reduced.

Effect of Commercial Vehicles

The effect of the proportion of commercial vehicles has also been tested. Commercial vehicles are believed to introduce an effect on the dispersion of the travel time observations and consequently of the slope of the mean line. It is expected that as the proportion of commercial vehicles (dual-tired vehicles) increases, the standard error will increase. The hypothesis on this effect by commercial vehicles has been confirmed with the Pittsburgh data (Fig. 11). The Pittsburgh data were selected for this demonstration since the sections of this set had large contrasts in the number of commercial vehicles. A line is drawn through the set of points that correspond to the two

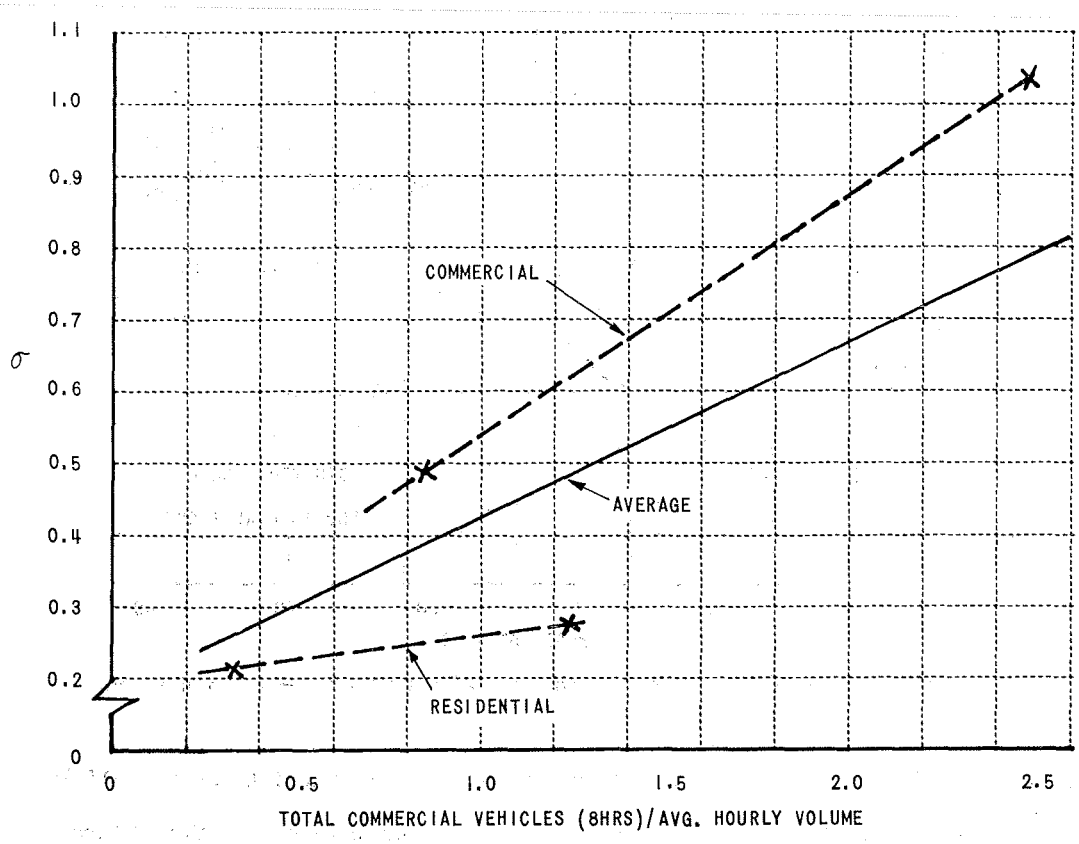

Figure 11. Relationship between standard error and commercial vehicles, Pittsburgh data.

residential street sections. A second line is drawn through the set that corresponds to the two commercial street sections. A third line is drawn through the pooled set of points.

The Slope Relationships for Traffic-Actuated Signal Systems

The factors that affect the responses of streets with traffic-actuated signal systems were expected to differ from those for streets with pre-timed signal systems. In fact, from the nature of traffic-actuated systems, it was expected that the greatest effect on the variation would come from the cross-traffic volume at the signalized intersections. The higher the volume on the cross streets, the greater the likelihood of having traffic-induced interruptions. Consequently, on the average, the delays on the through street would increase as the cross traffic increased.

This hypothesis was tested and confirmed against the set of Dallas data. All the street sections in the Dallas group have fully actuated signals and are all similar in their geometric characteristics. Average daily traffic (ADT) for the pertinent cross streets was obtained from the 1964 Dallas ADT map. The independent variable was selected to be the sum of the ADT's from all the cross streets that corresponded to the signalized intersections on each sample street section.

Figure 12 shows the relationship obtained between the slope and cross-ADT, when the directions are pooled together. Low dispersion is evident for this group. However, Sections 4 and 7 appear as outliers, which appears to be because traffic in both directions carries approximately the same volume even during the peak periods; no one direction is able to dominate the progression of flow. Also shown in Figure 12 are the indicated differences between streets with parking and streets with no parking permitted.

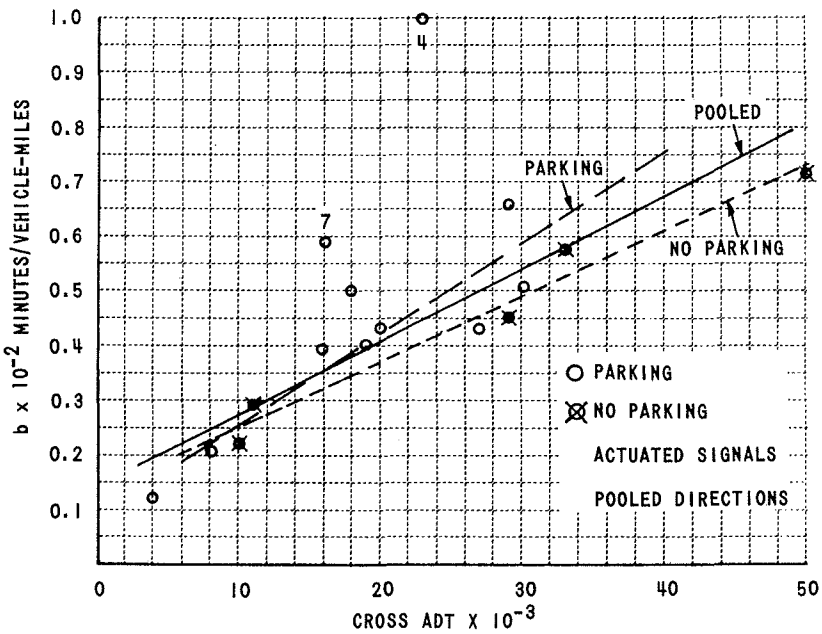

Figure 12. Relationship between slope and cross-ADT, Dallas data—pooled direction.

Highly significant relationships are evident between the slope and the cross-ADT for these data groups.

Figure 13 shows the relationship corresponding to the favored direction of flow (heaviest volume) during the peak periods. The slope relationship for the off-peak periods is shown in Figure 14. Greater dispersion is evident for these relatively

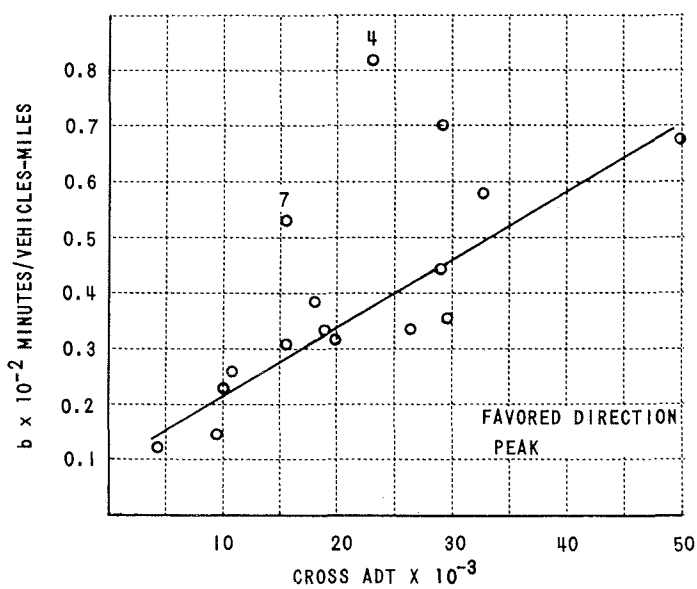

Figure 13. Relationship between slope and cross-ADT, Dallas data—peak.

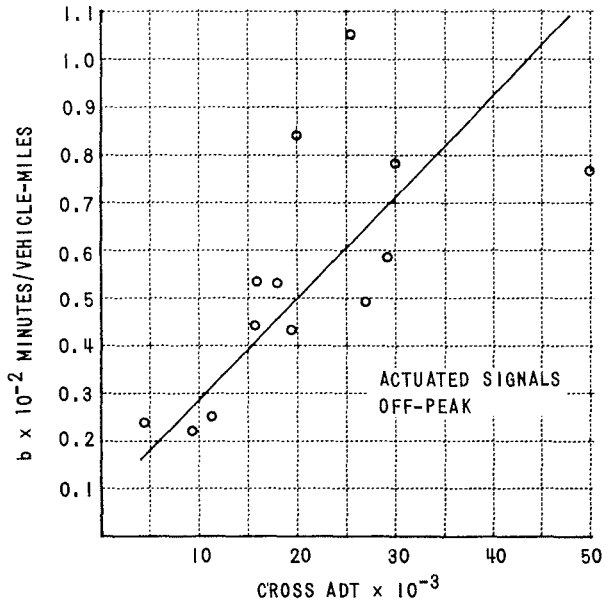

Figure 14. Relationship between slope and cross-ADT, Dallas data—off peak.

low-volume conditions, which was expected beforehand. The classification according to peak favored and off peak has thus also yielded highly significant relationships.

Slope Relationships for Traffic-Supervised Systems

The traffic-supervised, semiactuated set of streets from the San Diego sample was also expected to be principally affected by the cross-ADT. In order to obtain a stable

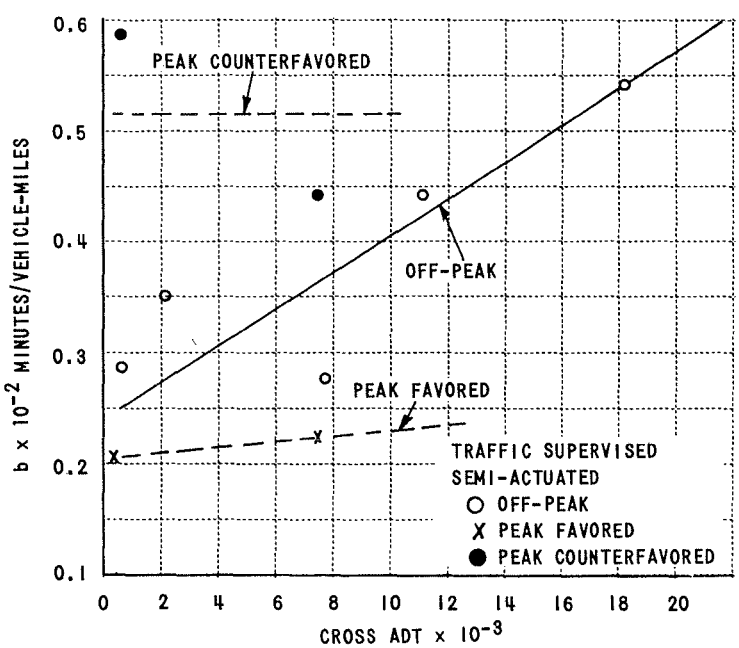

Figure 15. Relationship between slope and cross-ADT, San Diego data.

set of data that would not be confounded by extraneous variables, the signal system conditions were held fixed at either the peak settings or the off-peak settings during each set of experimental runs. Thus, the flexible features of the traffic-supervised system were not tested. Once again, since semiactuated signals are employed, the likelihood of traffic interruptions would increase as the volume on the signalized cross streets increases. This suggested the cross-ADT as the principal variance-introducing factor.

The relationships obtained for this type of system are shown in Figure 15. The off-peak condition shows a significant relationship. It is further noted that a comparison of this data with the data for the actuated signal systems (Dallas) reveals that the data sets roughly overlap at the low cross-ADT region, suggesting quite similar relationships.

CONCLUSIONS

A measure of traffic operational performance was proposed which is expressed in terms of four driver satisfaction factors: travel time, driver discomfort, driving hazards, and direct vehicle running costs. Travel time was discussed in great detail. An extensive investigation was performed on this factor, based on the slope relationship between travel time and volume. By accounting for the effect of volume variation in this way, it was possible to perform a more efficient investigation of the effects of the other geometrics and signalization factors. This led to the demonstration of the operational effects of the most significant street characteristics.

Signal density was shown to have a highly significant effect on the variation of travel time through analyses of variance and multiple regression analyses performed on several factorial experimental designs (Tables 1, 2 and 3). Road width (lane width), percent green time, and parking had a lesser effect. Analysis of covariance confirmed the conclusion that the major variance-producing factors had been identified, and that, consequently, consistent general relationships could be obtained between the slopes and the geometrics and signalization factors. With the satisfactory results obtained from these preliminary steps, the way was then clear for the final and key step—the determination of general prediction relationships between the geometrics and signalization factors and the slopes.

The strong effect of signal density in coordinated signal systems was shown in Figures 4 to 8. Lane width and parking were indicated to have a secondary effect. Attention was called to the relatively low dispersion of the observations around the mean line for each homogeneous group. Also, the effect of locality was shown to be not significant. Consistent relationships were obtained after pooling the observations from different cities.

The effect of signal split and number of lanes was shown in Figures 9 and 10; the effect of commercial vehicles was shown in Figure 11.

The significant factor in traffic-actuated and traffic-supervised systems was found to be the relative number of signalized intersections, weighted by the cross-ADT at those intersections. Figures 12 to 15 show these relationships. A contrast was observed between peak and off-peak operations. Parking, once again, was found to have a secondary effect.

The figures in the foregoing discussion provide a set of general estimating relationships for the travel time factor that covers large classes of arterial streets. Consistent and reliable estimates of the travel time slopes can be obtained for specific street sections, given the geometrics and traffic control characteristics. These estimates can then be applied to the traffic evaluation problem, given the volume distribution.

Although it is believed that the most reliable estimates of the operational performance of streets should also make use of the other three driver satisfaction factors, first order estimates of street performance can be obtained through the use of travel time. Travel time is recognized as the major factor from the drivers' viewpoint. The performance, in terms of the one factor, would be obtained by making use of the first term in Eq. 1, thus

$$
\begin{aligned}
M &= E\{vt\} \\
&= E\{v(a + bv)\} \\
&= aE\{v\} + b\,E\{v^2\} \\
&= aE\{v\} + b\,[\mathrm{Var}\{v\} + E^2\{v\}]
\end{aligned}
$$

Hence, given a street section whose slope relationships have been predetermined, and knowing the street characteristics of the section, such as type of signalization, signal density, number of moving lanes, etc., . . . , the slope b could then be obtained. This would be done, for example, through the application of the figures presented in this paper. And, given the speed zoning or speed of progression of the street, the coefficient a would be determined. The volume distribution would give E{v} and Var {v}. Finally, the determination of the performance measure, M, would be a simple arithmetical computation involving the quantities noted.

To illustrate an application of the results presented in this paper, a sample problem is discussed. Say that it has been proposed that a certain one-mile section of an arterial street have two additional intersections signalized in order to assist the cross traffic which has increased in the two corresponding cross streets. This would raise the number of signalized intersections over the section from an initial four to six. The street section is further characterized as having a 52-ft pavement width, a pre-timed coordinated signal system, and it is zoned for a 30-mph speed. It is desired to know how this signalization change will affect the through traffic on the arterial section.

To make the evaluation, the directional volume characteristics E {v} and Var {v} are required. These two parameters are obtained from a sample directional volume (say, 15-min volumes) distribution for the section. The first parameter is simply

$$E\{v\} = \frac{\mathrm{ADT}}{(2)(4)(24)}$$

Assuming now that the street section under study has the following volume parameters:

$$E\{v\} = \frac{16{,}000}{(2)(4)(24)} = 83.5 \text{ vehicles/15 min}$$

$$\mathrm{Var}\{v\} = 3480$$

then all the necessary information is now available for performing the evaluation.

The expected, or average, travel time per 15-min period is determined for each of the two signal conditions. It is recalled that the equation for M will be used. First, it is observed that, since the section is zoned for 30 mph, then

$$a = \frac{60}{30} = 2.0 \qquad \text{(min/mi)}$$

Further, since the section has a pre-timed coordinated signal system with a 52-ft pavement, the solid line in Figure 4 is to be employed. For the first signal density condition, we enter the graph in the abscissa at the value of 4 signals per mile and read out the slope $\beta = 0.5 \times 10^{-2}$. For the second (the proposed) signal condition, the slope of $\beta = 0.67 \times 10^{-2}$ is obtained.

Letting M_1 be the expected travel time for the street with signal density of 4, and M_2 for the street with signal density of 6, we then obtain

$$M_1 = (2)(83.5) + (0.005)[3480 + (83.5)^2]$$

$$M_2 = (2)(83.5) + (0.0067)[3480 + (83.5)^2]$$

Hence

$$M_2 - M_1 = (0.0017)\,[3480 + (83.5)^2]$$

$$= 17.8 \text{ min per directional mile per 15-min period}$$

This represents the increase in travel time per mile for the drivers using the street section in one direction over a 15-min period. Dividing this number by the average 15-min volume, we obtain

$$\frac{17.8}{83.5} = 0.213 \text{ min} = 12.8 \text{ sec/mi/veh}$$

Thus, by increasing the number of signalized intersections from four to six, the travel time per vehicle is increased by 12.8 sec while traveling over the one-mile section.

Concluding Comment

Our research has included the other three driver dissatisfaction factors; each has been investigated to a lesser degree than the travel time factor. The preliminary findings on these other three factors suggest that they can be treated in a similar fashion to the travel time factor. Further research, however, is required to establish the relationships between these factors and volume. We believe that the use of this overall methodology will result in objective and rational judgments of traffic operational performance. And, it is stressed, the suggested procedure is practicable.

ACKNOWLEDGMENTS

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2. Davis, O. L. Design and Analysis of Experiments. Hafner, 1960.

Appendix

COLLABORATING AGENCIES ON TRAVEL TIME TASK

Agency	Contact
New York State Department of Public Works, Subdivision of Transportation Planning and Programming	Roger L. Creighton, Director
Baltimore Department of Transit and Traffic	James L. Foley, Commissioner
Dallas Department of Traffic Control	Winston H. Carsten, Director
Detroit Department of Streets and Traffic	Alger F. Malo, Director
Pittsburgh Bureau of Traffic Planning	Anthony F. Miscimarra, Traffic Engineer
San Diego Division of Transportation and Traffic Engineering	Martin J. Bouman, Transportation and Traffic Engineer
Buffalo Division of Safety	Henry W. Osborne, Traffic Engineer
Erie County Department of Public Works	H. Dale Bossert, Commissioner
Buffalo Department of Police	William H. Schneider, Commissioner
Town of Amherst Highway Department	George Austin, Superintendent