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Foreword

Traffic control devices in one form or another have probably been in use since that time when primitive man first placed a broken branch to indicate a turn in a path. Although a great deal still remains to be done in applying modern traffic control methods, researchers have already turned their sights toward more sophisticated means of control. The papers in this RECORD for the most part present new ideas relating to traffic control devices. They should be of interest to traffic engineers and administrators who have to cope with today's problems and who would perhaps like some inkling of new developments in the field.

The first paper, by a Missouri researcher, has utilized computer simulation models to see if the German traffic signal "funnel" concept could be applied on high-speed roads having intersections with semiactuated signal control. The models were validated by real-life traffic data from a typical site. It was indicated that main-road traffic flow was substantially improved without causing excessive side-road delays.

The second paper describes how an urban transportation study produced data for better management of traffic signals. A computer printout is furnished to those responsible for signals. It is useful for purposes of maintenance planning, capital improvements, and updating as well as for improving signal timing and phasing.

The next paper, by a California engineer, presents an evaluation of special sign installations used to reduce accidents. The study investigated accident prevention methods for curve warning, advisory speed, and 4-way stop sign installations. Warrants for left-turn channelization were also developed from use of before-and-after studies at 53 channelized intersections.

A team of California researchers developed a tool for traffic engineers to use in evaluating alternate traffic engineering improvements for urban networks, as the next paper points out. The research demonstrated how cathode ray tubes could visually depict what certain improvements, if made, could produce insofar as traffic volumes, travel times, and various other measurements were concerned.

The fifth paper in this RECORD, by two Ohio researchers, explores the possibilities of more efficient field measurement methods at intersections. The methods can also be used to see if the best use is being made of a traffic control device. One method uses a motion picture camera mounted on a rotating platform and the other uses mirrors to simultaneously photograph all four intersection approaches. Digital simulation models were developed to help test the two camera methods.

Two Michigan researchers produced the sixth paper, which is concerned with a mathematical procedure for optimum selection of progression on a two-way arterial street. The authors took into account constraints by car-following behavior in developing their procedure.

The last paper, by a Catholic University professor, uses queuing theory to explore the evaluation of operational efficiency of types of unsignalized intersections. Theoretical values were checked with observed data and some correlation was achieved.

Contents

Development of an Advisory Speed Signal System For High-Speed Intersections Under Traffic-Actuated Control

CHARLES E. DARE, University of Colorado, Denver Center[1]

High-speed signalized intersections are hazardous from the standpoint of causing rear-end collisions between vehicles on the same approach. The signal funnel concept developed in Germany is desirable at such locations since it substantially reduces the percent of vehicles stopping. However, the signal funnel has not been incorporated with the semiactuated traffic signal often used at intersections on major thoroughfares in the United States.

The major objectives of this investigation were to design and evaluate a speed signal system capable of functioning effectively with semiactuated control. The study involved a traffic control system for a T-junction utilizing an advisory speed signal on the main approach. The evaluation was accomplished by computer simulation models programmed in GPSS/360. The figures of merit for each model were (a) total number of vehicles stopping on the main approach during 15 signal cycles, (b) percent of vehicles forced to stop against the red signal on the high-speed route, (c) average delay incurred per side road vehicle, and (d) average delay per side road vehicle stopped.

The first simulation model described vehicle activity on a minor approach lane and a high-speed approach lane at a T-junction with a two-phase semiactuated controller. This model was validated by comparing simulation output to field data obtained at an intersection in a 45-mph speed zone. Field data were gathered for side flow ranging from 60 to 250 vph and main flow from 180 to 700 vph per lane. Linear regression equations were established relating each figure of merit to the traffic volume. Comparable data were extracted from the simulation model and regression equations involving the same variables were constructed. The corresponding equations from the field data and from the simulation were then statistically tested for equality of regression coefficients. The simulation model proved satisfactory for predicting the figures of merit for the traffic volumes involved.

The second simulation model was similar to the first, but included a main route speed advisory signal and a more elaborate side route vehicle detection system. Data obtained from the speed signal simulation model were compared to the output from the first model, thus evaluating the proposed signal funnel.

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[1]Formerly with University of Missouri-Rolla.

Paper sponsored by Committee on Traffic Control Devices and presented at the 48th Annual Meeting.

The traffic-actuated speed signal funnel stopped an average of only 2.0 percent of the main route traffic, while the conventional semiactuated controller stopped 20.9 percent. Furthermore, the speed signal system stopped only 2.8 cars on the high-speed route during 15 typical signal cycles, compared to 25.8 cars stopped in 15 cycles with the semiactuated control. The improvement in main road flow was obtained without causing excessive side road delay.

•A TRAFFIC signal located at an isolated intersection on a high-speed thoroughfare creates a hazardous situation when it interrupts the rapidly moving vehicular flow. The presence of an unexpected signal and the extreme speed differential as traffic halts combine to render vehicles highly susceptible to rear-end collisions in this situation.

Despite the undesirable effects of signalization, regulation is often necessary at high-speed intersections to provide safety for drivers desiring to enter or cross the main stream. Traffic engineers, recognizing the necessity for regulation, have devised controls that reduce the frequency of main flow interruptions. One of the more common controls in use today is the semiactuated signal which retains the main route green phase until a side arrival is detected. Semiactuated control offers the advantage of transferring the green only when required by the side traffic, but does not eliminate the problems resulting from unexpectedly halting the main flow.

THE SIGNAL FUNNEL CONCEPT

The signal funnel system developed in Germany by W. von Stein (10) is a traffic control system that provides access to a route while causing minimum disturbance to the flow. When utilized at high-speed locations, the signal funnel includes advisory speed signals preceding the signalized intersection. The operating principle is illustrated in Figure 1 for a simplified case involving a pretimed traffic signal and one advisory

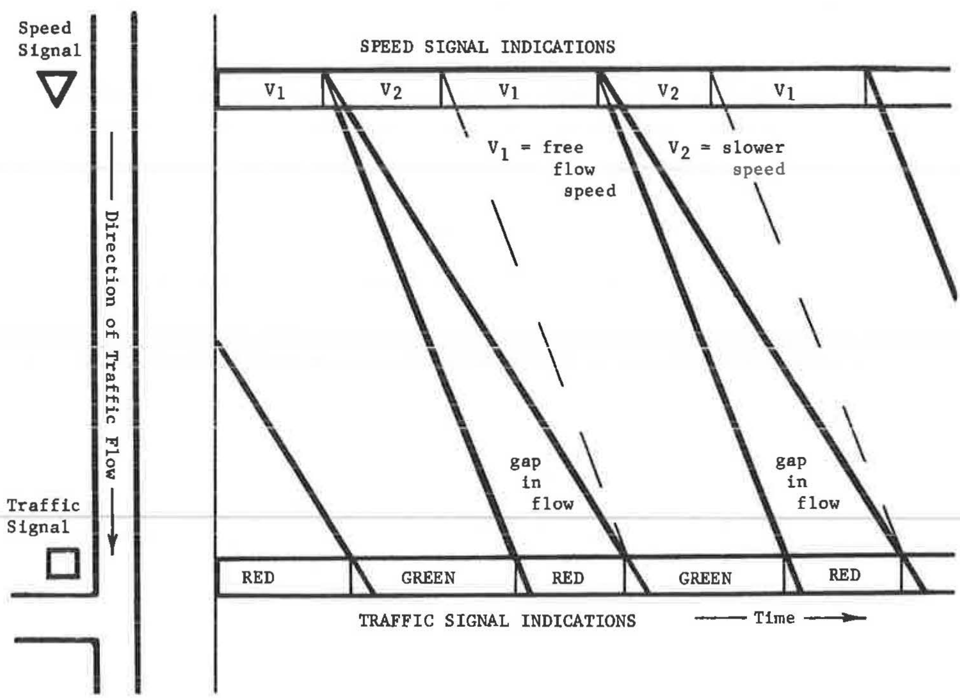

Figure 1. Speed signal operation with pretimed controller.

speed signal. The speed signal displays speeds to approaching traffic, thus regulating vehicle progress to avoid arrival at the intersection during the red phase.

Signal funnel systems have been used with success in Germany since 1954. One of their initial installations permitted 90 to 97 percent of the traffic to pass without stopping during moderate flow, even though the green phase was only 47 percent of the cycle. Up to this time, signal funnels have seen limited use in the United States, the first system being the Traffic Pacer (1, 7) developed at the General Motors Research Laboratory.

The signal funnel concept seems to offer great potential for reducing vehicle stoppages at intersections. Furthermore, one situation where this control would be especially useful is the high-speed location where cross traffic is relatively light compared to the main route volume. If an advance speed signal could be coordinated with a semiactuated controller, smooth and efficient regulation of such an intersection could be achieved. The traffic-actuated controller would cause interruptions only when necessary, and the speed signal would minimize the number of vehicles stopping during the main route red phase.

Schmarsel (8) has indicated the extent of signal funnel application in Germany and relates (9) that it has been combined with traffic-actuated control in few instances. However, no data are available concerning funnel system performance with traffic-actuated signals and the problem of coordinating advance speed signals with traffic-actuated equipment has not been extensively explored. It is, therefore, the purpose of this investigation to design and evaluate a control system with main route advisory speed signals and a traffic-actuated intersection signal.

SCOPE AND METHODOLOGY

The location involved is a high-speed T-junction, where regulation is by semiactuated signals. The analyses concern a single lane on a main approach and a single lane on the minor approach, assuming the results could be generalized to any number of lanes. Since this is a preliminary investigation, the evaluation is based on performance predicted by computer simulation models.

The investigation was conducted in three stages. First, field data were gathered at an intersection with semiactuated control. Next, an initial computer model was programmed to simulate the intersection observed in the field studies. This initial model was validated by comparing its output to the field data. Finally, the traffic-actuated speed signal funnel was developed, simulated by a second computerized model, and evaluated by comparing the output from this model to that from the semiactuated control simulations.

FIELD STUDY

The field study site was a T-intersection formed by a four-lane, two-way highway with a 45-mph speed limit, and a two-lane, two-way street in a 25-mph speed zone. The semiactuated signal at this location was linked to a magnetic detector under the side road pavement 25 feet preceding the stop line. The controller settings granted 45-sec minimum main green and 6-sec amber. The side route settings were 5-sec initial interval, 5-sec vehicle extension, 24-sec maximum side green, and 3-sec amber. The duration of each study was 15 complete signal cycles rather than a predetermined time period, thus avoiding inclusion of incomplete phases. Main route data pertained to one direction of flow, the direction selected to eliminate disturbances caused by left-turning vehicles.

Field data were gathered concerning these figures of merit:

1. The total number main route cars stopped in 15 cycles,
2. The percent main route traffic stopping at the light,
3. The average delay per side road vehicle, and
4. The average delay per side road vehicle stopped.

The high-speed route stoppage characteristics were of primary interest in this study. The side road delay measures were included since an effective system should not cause

TABLE 1

TRAFFIC VOLUME SUMMARY: FIELD DATA

Approach	Flow Range (vph)	Mean (vph)	Std. Dev. (vph)
Side road	59.6 - 249.0	134.67	47.69
Main road			
Outside lane	234.0 - 769.0	448.0	132.93
Inside lane	103.0 - 543.0	313.0	126.47
Average per lane	178.0 - 663.0	381.0	128.94

TABLE 2

MAIN ROUTE STOPPAGE CHARACTERISTICS: FIELD DATA

Lane	Percent Stopped			Number Stopped		
	Range	Mean	Std. Dev.	Range	Mean	Std. Dev.
Outside lane	11.9-39.7	23.3	6.09	6.0-52.0	32.8	14.0
Inside lane	8.9-34.2	20.4	6.41	11.0-69.0	20.8	12.0
Average per lane	10.4-37.1	22.2	5.61	8.5-60.5	26.8	12.8

excessive delay to a particular traffic movement. Data regarding traffic volume were also recorded during the studies.

Results of Field Studies

A total of 72 studies were initiated, of which 15 were discarded due to unusual traffic circumstances. The 57 retained studies represent 1,064 minutes of traffic observation. The volume summary is presented in Table 1. The side road volume, ranging from 59.6 to 249.0 vph, covers light flow through conditions approaching congestion. The average flow per main route lane is reported since subsequent analyses concern one typical approach lane.

It should also be pointed out that the following data analyses are based on mean values of the variables as computed for each study. This was necessary since the figures of merit must be related to the corresponding traffic volume.

Table 2 gives the stoppage characteristics on the high-speed route. A disadvantage of semiactuated control is that an average of 22.2 percent of the main road vehicles were stopped due to side street demands. Furthermore, the total number of vehicles stopping in 15 cycles averaged 26.8 per lane for the field studies.

The figures of merit regarding side road delay are summarized in Table 3. The delay per vehicle stopped averaged 23.4 sec, while the delay per vehicle was 19.0 sec.

Linear Regression Analysis: Field Data

The linear regression equations and statistical measures for the field data are given in Table 4. The statistical measures are as follows:

1. The standard error $S_{y \cdot x}$, which measures the residual variability of the data points around the regression line,
2. The coefficient of multiple determination R^2, indicating the percent of sample sum of squares explained by regression, and
3. The value of the F ratio from an analysis of variance testing the significance of the regression equation.

The main route stoppage characteristics are given by Eqs. 1 and 2, and are shown in Figure 2. Both responses in Figure 2 exhibit a definite increasing trend with respect to traffic volume. The significance of the relationships is indicated by the high F ratio of 301.90 for the number stopped and 37.85 for the percent stopped.

The side road delays plotted in Figure 3 correspond to Eqs. 3 and 4. The average delay per side road vehicle was not significantly related to side volume as indicated by the F ratio of 2.11. The delay per vehicle stopped has a more significant trend, the F ratio being 12.69.

TABLE 3

SIDE ROAD DELAY MEASURES: FIELD DATA

Figure of Merit	Range (sec)	Mean (sec)	Std. Dev. (sec)
Average delay per vehicle	11.4-25.6	19.0	2.76
Average delay per stopped vehicle	14.5-29.4	23.4	3.33

SIMULATION OF SEMIACTUATED CONTROL

The simulation model concerning semiactuated control is programmed in GPSS/

TABLE 4

LINEAR REGRESSION EQUATIONS AND STATISTICS: FIELD DATA

Equation No.	Equation	$s_{y.x}$	R^2	$F_{(df)}$
(1)	$Y_1 = -8.911 + 0.0693\, X_1$	5.06	0.846	$301.90^a_{(1, 55)}$
(2)	$Y_2 = 0.113 + 0.211(10^{-3})\, X_1$	0.0436	0.408	$37.85^a_{(1, 55)}$
(3)	$Y_3 = 11.43 + 0.113\, X_2 - 0.443(10^{-3})\, X_2^2 + 0.427(10^{-6})\, X_2^3$	2.68	0.107	$2.11_{(3, 53)}$
(4)	$Y_4 = 15.60 + 0.045\, X_2 - 0.378(10^{-3})\, X_2^2 - 0.169(10^{-5})\, X_2^3$	2.60	0.418	$12.69^a_{(3, 53)}$

[a]Significant at the 0.1 percent level.

where

Y_1 = number stopped per main road lane in 15 cycles;
Y_2 = percentage of vehicles stopped per main road lane;
Y_3 = average delay (sec) per side road vehicle;
Y_4 = average delay (sec) per side road vehicle stopped;
X_1 = sum of side road and main road flow (vph); and
X_2 = side road flow (vph).

360 (5). The program simulates vehicle activity on a high-speed lane for 1,150 ft preceding the intersection while a minor approach is simulated for 650 ft. Vehicle interarrival times are specified by shifted exponential distributions. The program incorporates signal control settings duplicating those at the field study site. The major assumptions regarding vehicle flow follow:

1. Traffic is composed of passenger cars only,
2. Turning movements from the main route are not permitted,
3. Vehicles do not pass in the vicinity of the intersection,

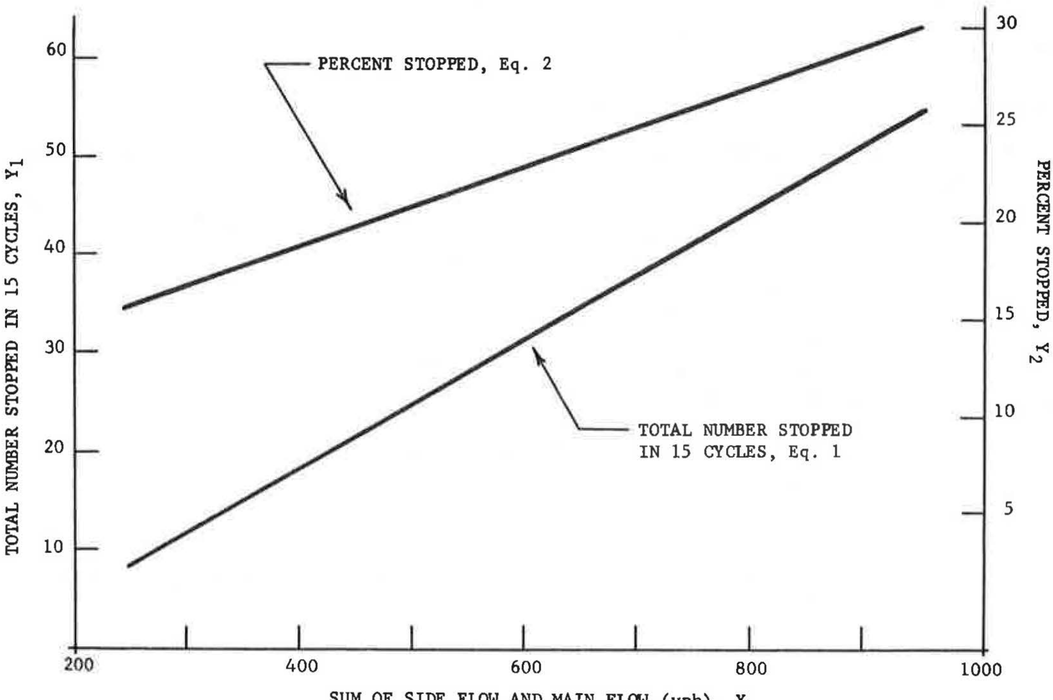

Figure 2. Main route stoppage characteristics, field data.

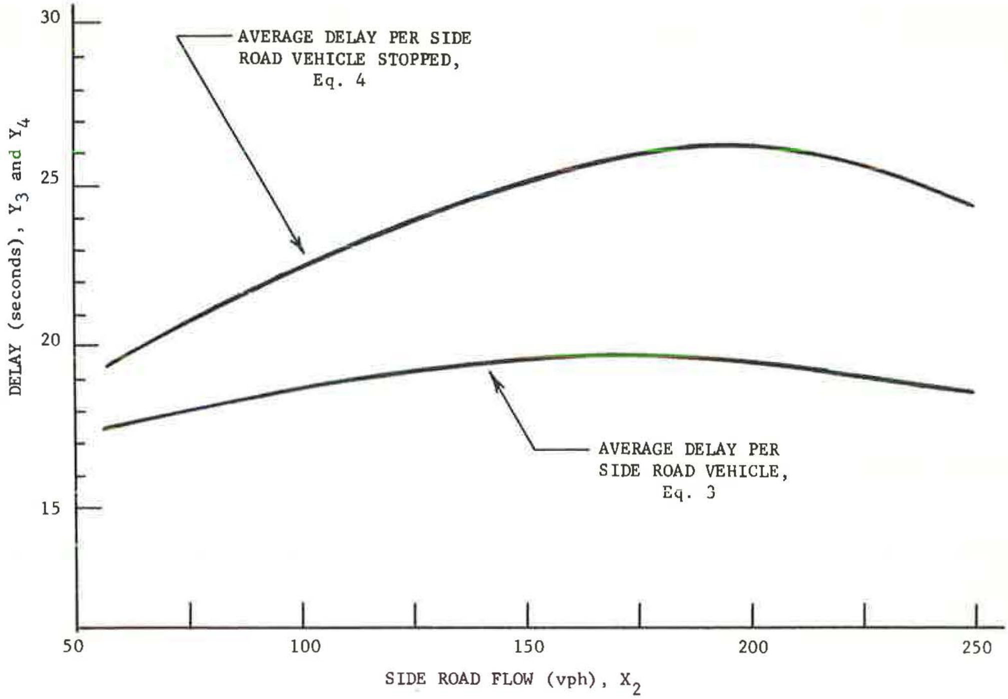

Figure 3. Side road delay measures, field data.

4. Stopping on amber is a simple probabilistic decision, and
5. Vehicles entering the system are assigned an initial velocity equal to their respective speed limit; the vehicle speed is altered as it proceeds, depending on conditions encountered.

Results of the Semiactuated Control Simulation

A total of 132 simulations were processed, each representing 15 cycles of the signal. From these, 57 runs were selected by matching simulated traffic volume to the traffic flows observed in the field studies. The 57 simulated studies represent 1,030 minutes of system observation. Table 5 summarizes simulated flow rates. The mean side flow of the simulation studies, 134.73 vph, is similar to the mean in the field studies, 134.67 vph. Since the road flows were matched first, it was difficult to attain the same agreement for the main road volumes. The mean flow in the field studies was 381.00 vph; the simulation mean is higher at 401.99 vph.

Table 6 summarizes figures of merit pertaining to the main approach. The mean percent main route traffic stopped in the 57 simulation studies was 20.9; the field data indicated 22.2. The total number of vehicles stopping during 15 cycles averaged 25.8 in the simulation and 26.8 in the field. The difference noted in this response is less than 0.1 car per cycle.

TABLE 5

TRAFFIC VOLUME SUMMARY: SIMULATION OF SEMIACTUATED CONTROL

Approach	Flow Range (vph)	Mean (vph)	Std. Dev. (vph)
Side road	63.8-244.0	134.73	47.90
Main road	204.0-684.0	401.99	118.59

TABLE 6

MAIN ROUTE STOPPAGE CHARACTERISTICS: SIMULATION OF SEMIACTUATED CONTROL

Figure of Merit	Range	Mean	Std. Dev.
Percent stopped	8.3-37.5	20.9	5.87
Number stopped	7.0-48.0	25.8	11.4

Table 7 summarizes the simulated minor approach delays. The delay per stopped vehicle averaged 23.1 sec, while the delay per vehicle was 18.2 sec. Both measures from the simulation closely approximate the corresponding means listed in Table 3 for the field data.

TABLE 7

SIDE ROAD DELAY MEASURES: SIMULATION OF SEMIACTUATED CONTROL

Figure of Merit	Range (sec)	Mean (sec)	Std. Dev. (sec)
Average delay per vehicle	12.2-23.0	18.2	2.78
Average delay per stopped vehicle	14.0-28.2	23.1	3.47

Linear Regression Analysis: Semiactuated Control

The regression equations and statistical measures associated with the simulation results are given in Table 8. These equations are plotted in Figures 4 through 6, which also show results from the field study to facilitate comparison.

The figures of merit for main road stoppages are in Figures 4 and 5. In both instances, the simulation results exhibit an increasing trend similar to the field study results, but at a lower level. The discrepancy in either case, however, is less than the standard error associated with the field data equation.

The side road delay figures of merit for both simulation and field study are plotted in Figure 6. Comparisons of these responses and the statistical measures in Tables 8 and 4 indicate excellent conformity.

Statistical Validation of the Simulation Model

The major step in the validation procedure was application of an F test that evaluates the closeness of paired regression lines (3). The linear regression equations as determined from the field data and from the computer simulation were evaluated in pairs by this F test (Table 9). This test indicated that differences between the pairs of regres-

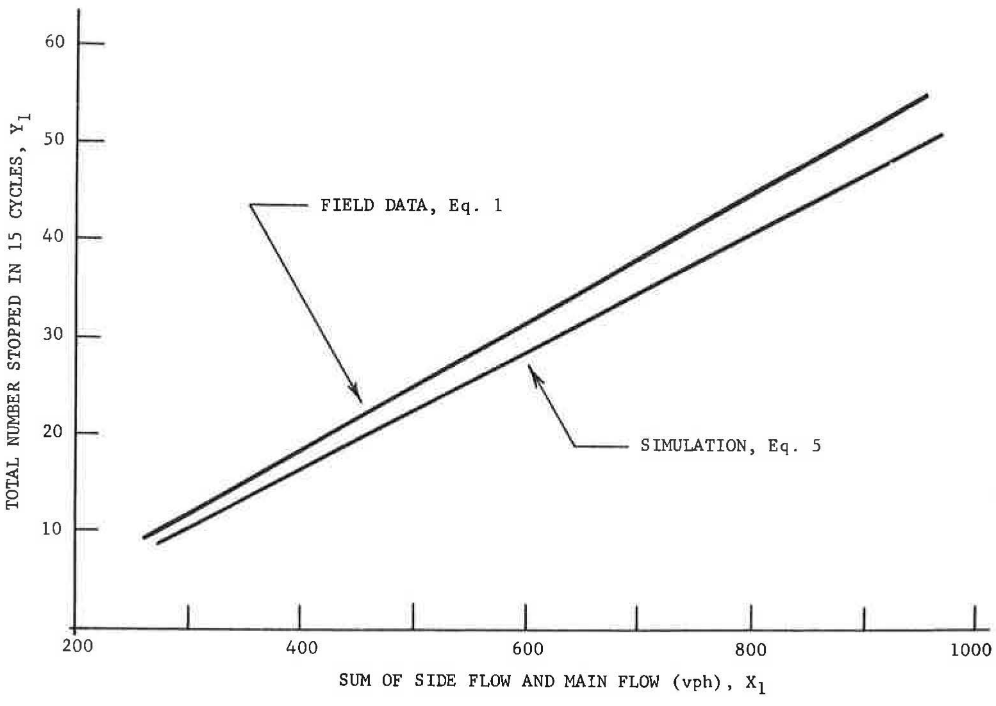

Figure 4. Total number stopped in 15 cycles, semiactuated control.

TABLE 8

LINEAR REGRESSION EQUATIONS AND STATISTICS: SIMULATION OF SEMIACTUATED CONTROL

Equation No.	Equation	$s_{y.x}$	R^2	$F_{(df)}$
(5)	$Y_1 = -9.409 + 0.0657 X_1$	5.16	0.800	$219.85^a_{(1, 55)}$
(6)	$Y_2 = 0.088 + 0.224(10^{-3}) X_1$	0.0477	0.353	$29.94^a_{(1, 55)}$
(7)	$Y_3 = 6.90 + 0.214 X_2 - 0.125(10^{-2}) X_2^2 + 0.232(10^{-5}) X_2^3$	2.74	0.075	$1.43^a_{(3, 53)}$
(8)	$Y_4 = 0.77 + 0.405 X_2 - 0.231(10^{-2}) X_2^2 + 0.437(10^{-5}) X_2^3$	3.02	0.286	$7.08^a_{(3, 53)}$

[a]Significant at the 0.1 percent level.

where

Y_1 = number stopped per main road lane in 15 cycles;
Y_2 = percentage of vehicles stopped per main road lane;
Y_3 = average delay (sec) per side road vehicle;
Y_4 = average delay (sec) per side road vehicle stopped;
X_1 = sum of side road and main road flow (vph); and
X_2 = side road flow (vph).

sion equations were not sufficient to warrant rejection of the simulation model as a tool for predicting intersection performance. Thus, the GPSS/360 model was accepted for investigating the figures of merit for the traffic volumes involved.

DEVELOPMENT OF THE TRAFFIC-ACTUATED SPEED SIGNAL SYSTEM

The purpose of a speed signal is to alter progress of certain approaching vehicles, thus creating substantial gaps within the main stream. These gaps then serve as the time intervals for the red phase. Thus, coordination between the speed signal and the traffic controller is imperative so the gaps will coincide with the red phase. Coordi-

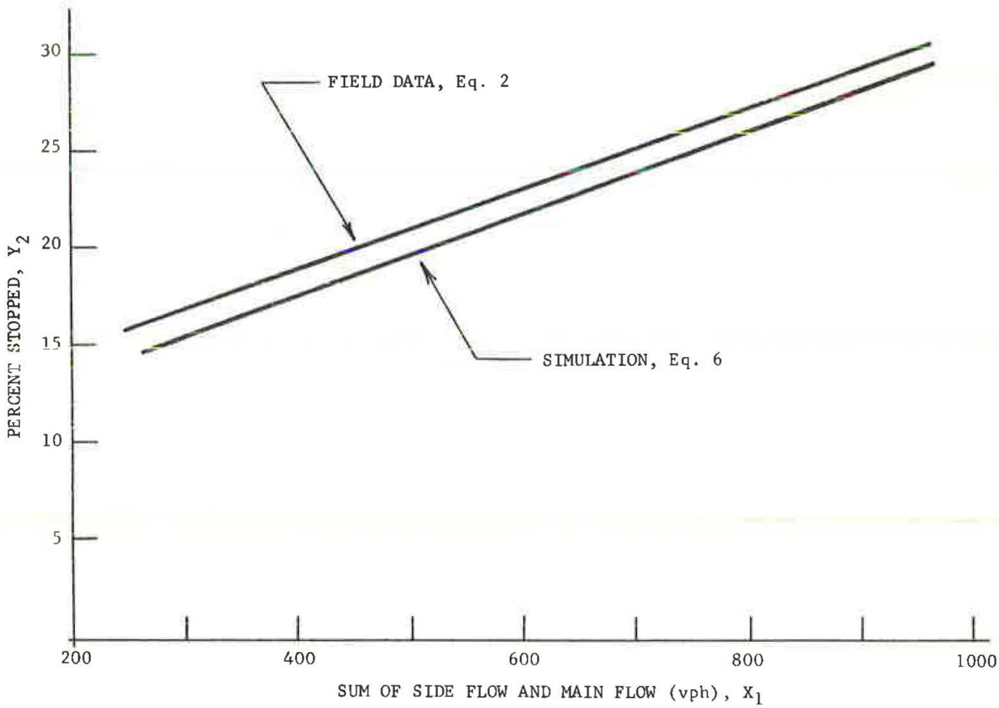

Figure 5. Percent stopped, semiactuated control.

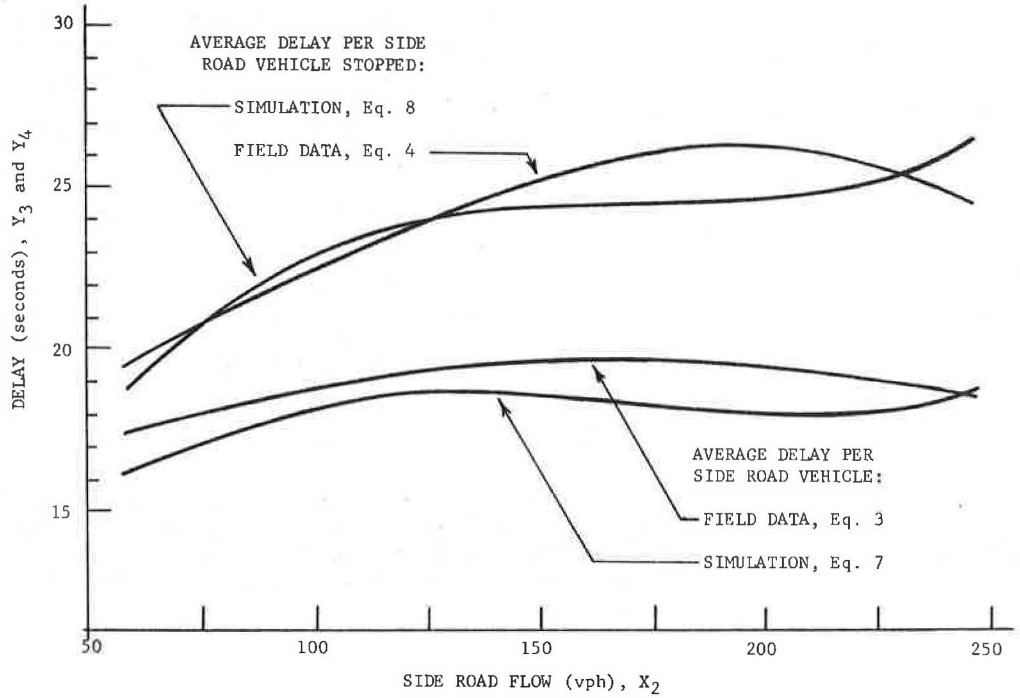

Figure 6. Side road delay measures, semiactuated control.

nating an advance speed signal with a semiactuated traffic controller is difficult since this controller does not function on a completely regular cycle. Figure 7 is a space-time diagram illustrating this problem for a hypothetical traffic-actuated speed signal system. As the traffic signal proceeds through the cycles, times of phase changes do not occur at regular intervals. The start of a main red phase, as shown by Point A, is not known until a side arrival is detected, subject to the minimum main green time restriction. Further, the red termination time noted by Point B is not specified until the last vehicle crosses the side detector or the maximum limit expires.

Speed Signals in Fixed Cycle Systems

Before developing the traffic-actuated speed signal system, it is advisable to consider the principles of speed signal coordination in fixed-cycle signal funnels. This principle is illustrated in Figure 8. The advisory signal in this example alternately shows one of two possible speeds to approaching vehicles. As outlined by Breuning (2), the following expression may be derived based on Figure 8, assuming vehicles adopt speed changes at the speed signal:

$$s = 1.467\ T_r\ V_1 \left(\frac{V_1}{\Delta V} - 1\right) \qquad (9)$$

where

s = funnel length, in ft;
T_r = duration of main red phase, in sec;
V_1 = free flow speed, in mph; and
ΔV = $(V_1 - V_2)$ the speed difference required for creating the gap, assumming V_2 is the slower speed indication.

TABLE 9

TEST FOR EQUALITY OF REGRESSION MODELS

Figure of Merit	Computed F Ratio	Degrees of Freedom	F Table Value (0.01, df)
Number stopped on main road	3.32	(2, 110)	4.81
Percent stopped on main road	2.00	(2, 110)	4.81
Side road delay per vehicle	0.622	(4, 106)	3.50
Side road delay per stopped vehicle	0.837	(4, 106)	3.50

Eq. 9 emphasizes the influence of free-flow speed on the funnel length and shows that for any given free-flow speed the funnel length may be reduced only by increasing the speed differential utilized or by decreasing the gap duration.

Coordinating the Traffic-Actuated Speed Signal

To establish coordination between the advance speed signal and the traffic signal in a traffic-actuated system, the following information must be obtained:

1. Advance notice of the forthcoming side road green, and
2. An estimate regarding the duration of the next side green.

Advance notice of the side green would activate the speed signal and the main route gap could begin to form before the side route traffic reaches the intersection. The estimate regarding the side green duration would be used by the speed signal to create a gap of appropriate size. The speed signal would have this capability if options were available for the lower speed indication to be shown.

Side Route Detection System

The necessary information concerning the time and duration of the next side green could be obtained by installing auxiliary detectors on the side approach (Fig. 9). De-

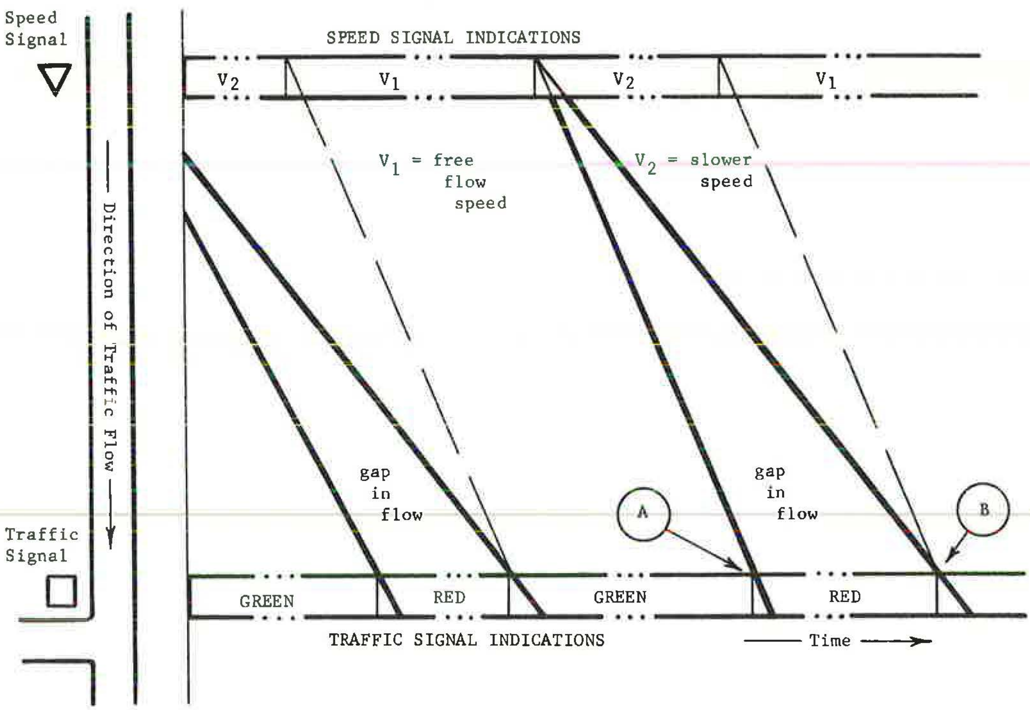

Figure 7. Space-time diagram: speed signal in a traffic-actuated system.

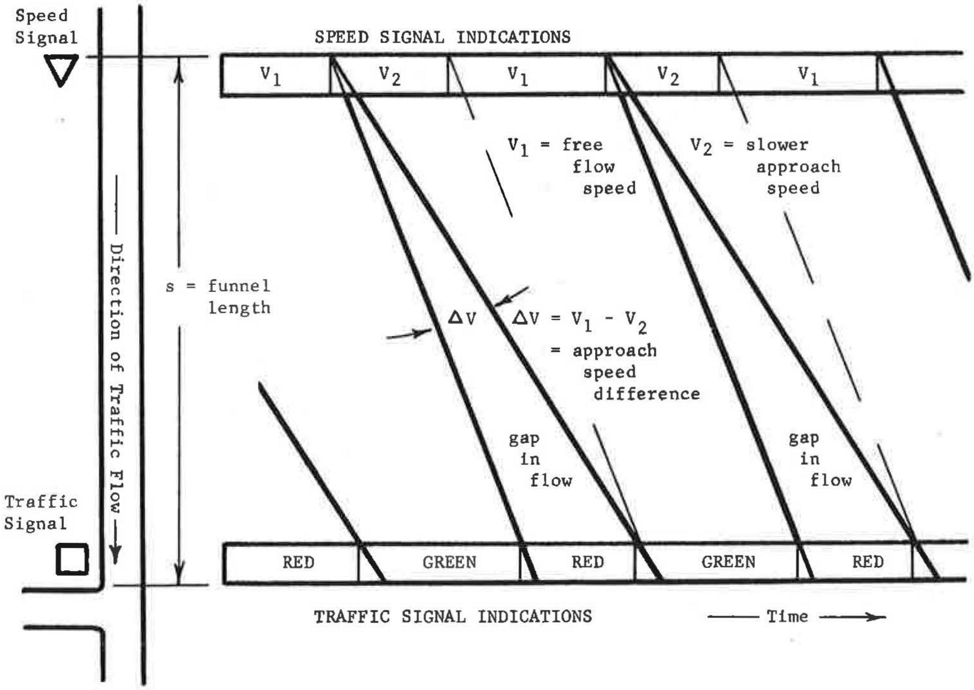

Figure 8. Space-time diagram: speed signal in a fixed-cycle system.

tector 2 provides the advance warning of vehicle arrivals to the main route speed signal. Detector 1, nearest the stop line, has two functions:

1. Places calls in the traffic signal for phase changes and vehicle extensions, and
2. Detects vehicles trapped on the side approach due to overloading the side green phase.

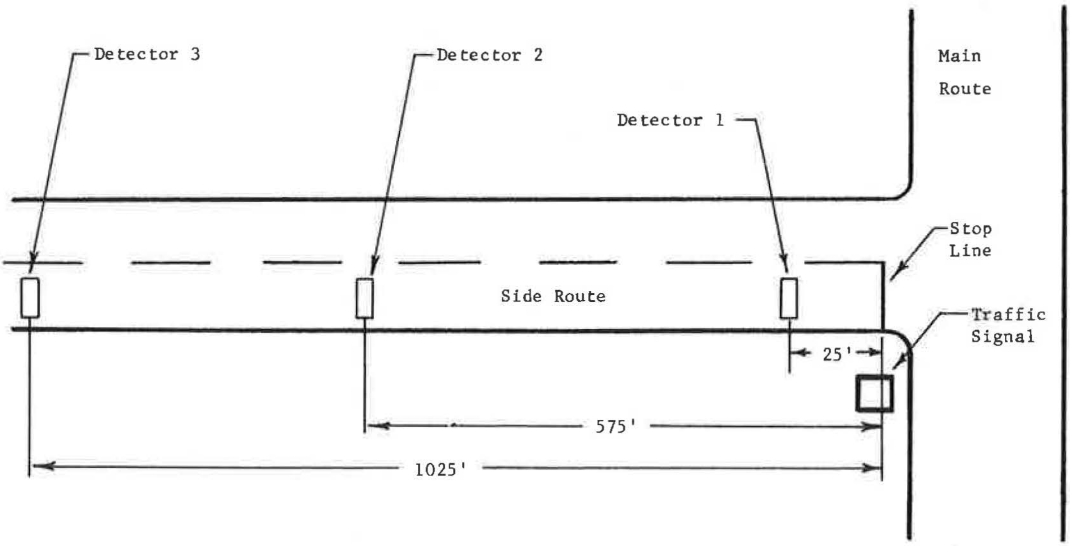

Figure 9. Proposed side road detector arrangement.

The third detector is used in conjunction with detector 1 to indicate the probable length of the next side green phase. Detector 3 maintains a count of vehicles passing in a specified time period and detector 1 reports congested conditions in the vicinity of the intersection stop line.

With adequate information available from the side route, attention is now focused on the main route speed signal system. Unfortunately, there are several important design considerations that prohibit development of the main route components on a general basis. These factors all have significant influence on the design, but each one varies from one location to another, or depends on the desired system complexity.

1. The gaps required for processing the side flow at the site,
2. The intersection controller settings,
3. The free-flow speeds (speed limits) on the approaches,
4. The number of speed signals utilized on the main approach,
5. The number of lower speed options provided each speed signal,
6. Vehicle deceleration distance for various speeds and sites, and,
7. The maximum differential in main road speed indication to be tolerated.

Considering the number of design variables and their complicated interrelationships, it was decided to limit the development to a basic funnel system suitable for the field study site. Therefore, the following design criteria were assumed:

1. The free-flow speed indication would be 45 mph,
2. Only one advance speed signal will be placed on the main approach,
3. The number of options available for lower speed indications on that signal will be limited to two,
4. The speeds shown on the speed signal must all be a multiple of 5 mph for driver convenience, and
5. The maximum differential used must not exceed 20 mph.

It is now possible to evaluate Eq. 9 using the 45-mph main approach speed, giving:

$$s = 66.0 \; T_r \; (\frac{45}{\Delta V} - 1) \tag{10}$$

Eq. 10 indicates numerous system design alternatives still exist for the given approach speed, depending on gap size required and the speed differential selected for the advisory signal.

Next, the intersection controller settings were reviewed to set limits on the gaps to be considered. For the field study location, the minimum main red phase is 13 sec, with a maximum of 30 sec. An additional 3 sec added to these values allowed for the portion of main amber considered an effective red period. Consequently, the gaps required ranged from 16 to 33 sec.

Preliminary computations were then performed using Eq. 10 and the 13-sec minimum gap requirement. It was found that even with the minimum gap the only feasible lower approach speeds were 25 and 30 mph, since excessive funnel length was required when either 35 or 40 mph were utilized.

Having determined the feasible lower speed indications, it became apparent that the length of the maximum gap had to be restricted to avoid the extremely long funnel. Analysis of the field data revealed that a 22-sec main red phase was usually adequate for processing side flows as high as 200 vph. Since side route volumes exceeding 200 vph were seldom observed at the study site, 22 sec was then selected as the maximum gap. The computation performed using the 22-sec gap and the greatest tolerable speed differential (ΔV = 20 mph when 25 mph is shown) in Eq. 10 yields a funnel length of 1,815 ft. An additional 770 ft was added to this length allowing for vehicle deceleration (6) from 45 mph to 25 mph, thus specifying a tentative funnel length of 2,585 ft.

Assuming this distance, the only other feasible speed differential (ΔV = 15 mph when 30 mph is shown) was evaluated to verify the minimum gap adequacy. The result was a 15.1-sec gap. Since this approximated the minimum main route gap requirement, the tentative length of 2,585 ft was accepted. The speed signal system design parameters were thus established as:

1. Funnel length (s) = 2,585 ft;
2. Speed differential ΔV = 15 mph, yielding a 15.1-sec gap; and
3. Speed differential ΔV = 20 mph, yielding a 22.0-sec gap.

These parameters were included in the speed signal simulation discussed in the following section. Two additional restrictions were also incorporated to establish adequate coordination between the speed signal and the traffic signal:

1. The traffic signal change to main red was delayed until the last vehicle showed a free-flow speed on the main route had sufficient time to clear the intersection, and
2. Speed signal operation was synchronized with the traffic signal so the main road minimum green control setting was recognized.

SIMULATION OF THE TRAFFIC-ACTUATED SPEED SIGNAL SYSTEM

The initial simulation model for semiactuated control was modified to include the speed signal on the high-speed route, a series of three detectors on the minor route, and statements regarding the control and effects of the advisory speed signal. Data were gathered from the speed signal simulation model for the same main and side road flows previously studied.

Results of the Speed Signal Simulation

A total of 132 simulations were run, of which 57 were retained on the basis of matching the vehicular flows to those previously simulated. The studies retained for analysis represent 1,130 minutes of traffic observation. Table 10 sumarizes the flow values from the speed signal simulations. The average side flow of 134.85 vph is closely matched to the 134.73 vph in the semiactuated control simulation. The average main flow in the speed signal analysis, 392.27 vph, is slightly less than the 401.99 vph in the previous simulation model.

The figures of merit regarding main route stoppage characteristics in the speed signal funnel are summarized in Table 11. These measures are extremely important since the ultimate objective of the funnel is reducing the number of main route vehicles stopping. Table 11 indicates the percentage of vehicles stopping is slightly less than 2 percent, considerably below the 20.9 percent in Table 6 for the standard semiactuated signal. Furthermore, the total number stopping during 15 cycles of the intersection signal dropped from an average of 25.8 with semiactuated control to an average of 2.75 for 15 cycles with the speed signal system. Moreover, Table 11 shows the maximum number observed to stop in 15 cycles was only 14 vehicles, or slightly less than one vehicle per signal cycle.

Table 12 summarizes the side road delay measures from the speed signal model. The mean values, 21.2 and 25.9 sec, are both approximately 3 sec greater than the corresponding averages reported in Table 7 for semiactuated control.

LINEAR REGRESSION EVALUATION: SIGNAL FUNNEL VS SEMIACTUATED CONTROL

Equations and statistical measures representing the figures of merit from the speed signal simulation are given in Table 13. These equations are plotted in Figures 10 and

TABLE 10

TRAFFIC VOLUME SUMMARY: SIMULATION OF SPEED SIGNAL FUNNEL

Approach	Flow Range (vph)	Mean (vph)	Std. Dev. (vph)
Side road	60.0-247.0	134.85	47.88
Main road	202.0-709.0	392.27	116.91

TABLE 11

MAIN ROUTE STOPPAGE CHARACTERISTICS: SIMULATION OF SPEED SIGNAL FUNNEL

Figure of Merit	Range	Mean	Std. Dev.
Percent stopped	0.0-7.49	1.99	1.59
Number stopped	0.0-14.0	2.75	2.58

TABLE 12

SIDE ROAD DELAY MEASURES: SIMULATION OF SPEED SIGNAL FUNNEL

Figure of Merit	Range (sec)	Mean (sec)	Std. Dev. (sec)
Average delay per vehicle	16.6-26.0	21.2	2.59
Average delay per stopped vehicle	20.8-29.5	25.9	2.17

13 along with corresponding results from the semiactuated control simulations.

The total number of vehicles stopping in 15 cycles is illustrated in Figure 10 for both types of control. These responses each increase as flow increases, however, the plot for semiactuated control averages 23 units greater than with the speed signal and exhibits a greater slope. The R^2 statistic

TABLE 13

LINEAR REGRESSION EQUATIONS AND STATISTICS: SIMULATION OF SPEED SIGNAL FUNNEL

Equation No.	Equation	$s_{y.x}$	R^2	F(df)
(11)	$Y_1 = -2.453 + 0.987(10^{-2})X_1$	2.07	0.363	$31.32^{a}_{(1,\ 55)}$
(12)	$Y_2 = 0.186(10^{-2}) + 0.413(10^{-4})X_1$	0.0146	0.168	$11.10^{a}_{(1,\ 55)}$
(13)	$Y_3 = 23.67 + 0.284(10^{-2})X_2 - 0.289(10^{-3})X_2^2 + 0.896(10^{-6})X_2^3$	2.46	0.149	$3.084^{b}_{(3,\ 53)}$
(14)	$Y_4 = 20.21 + 0.111X_2 - 0.109(10^{-2})X_2^2 + 0.240(10^{-5})X_2^3$	2.20	0.027	$0.494_{(3,\ 53)}$

[a]Significant at the 0.1 percent level. [b]Significant at the 5 percent level.

where

Y_1 = number stopped per main road lane in 15 cycles;
Y_2 = percentage of vehicles stopped per main road lane;
Y_3 = average delay (sec) per side road vehicle;
Y_4 = average delay (sec) per side road vehicle stopped;
X_1 = sum of side road and main road flow (vph); and
X_2 = side road flow (vph).

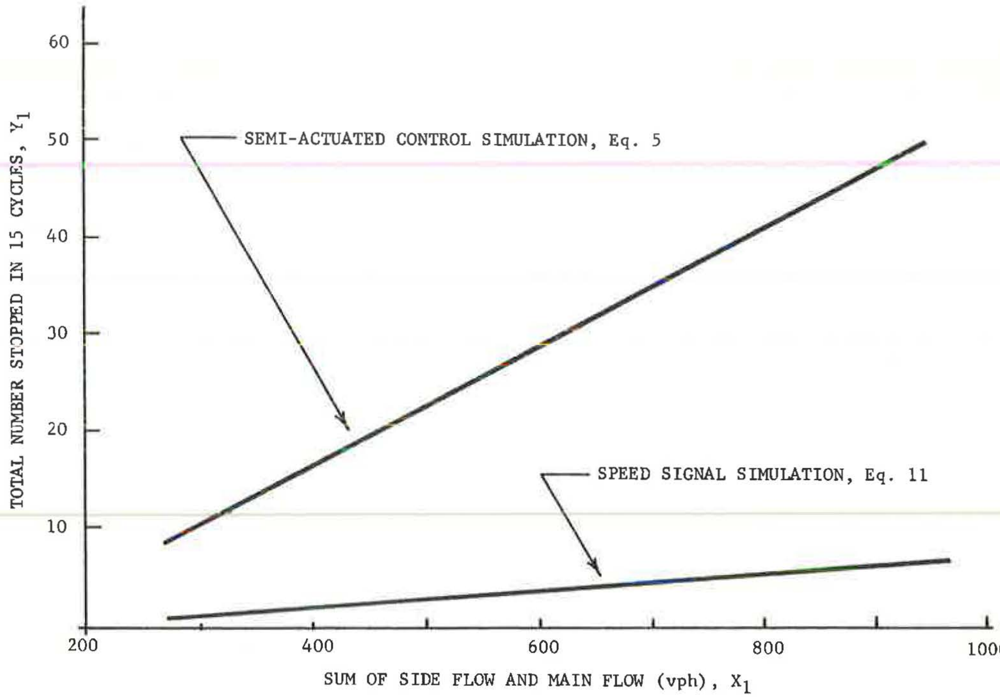

Figure 10. Total number stopped in 15 cycles, speed signal vs semiactuated control.

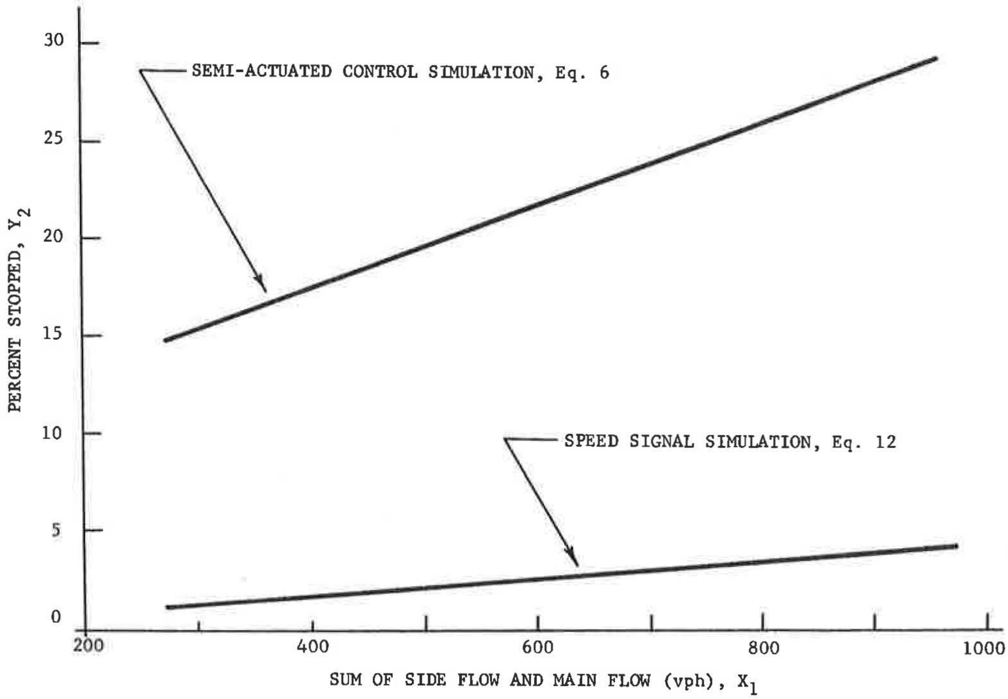

Figure 11. Percent stopped, speed signal vs semiactuated control.

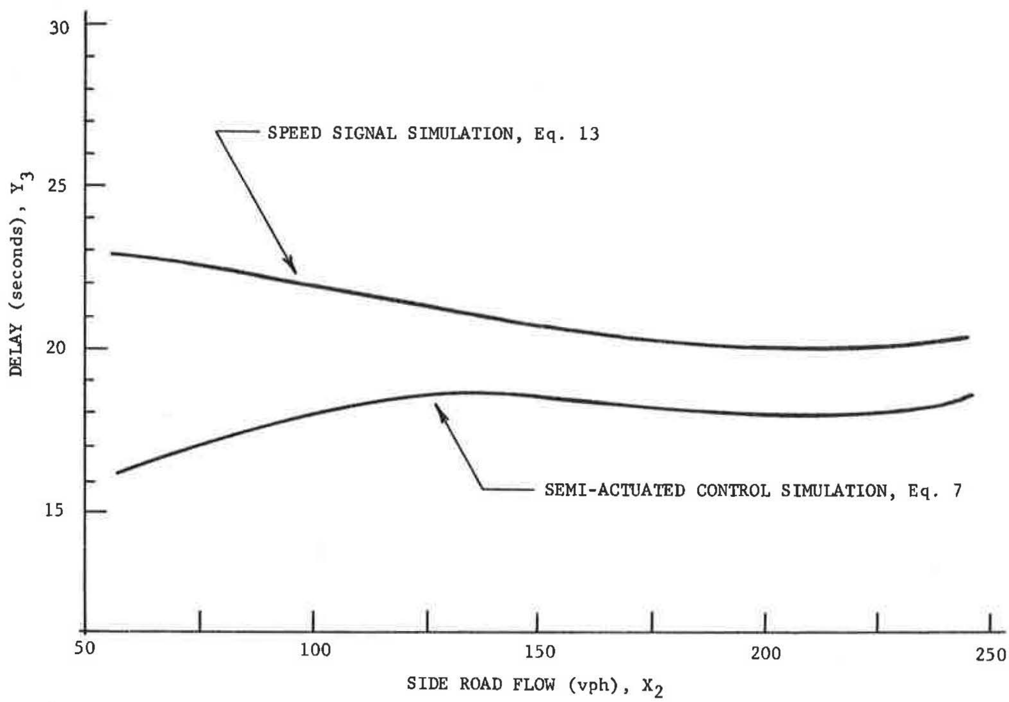

Figure 12. Average delay per side road vehicle, speed signal vs semiactuated control.

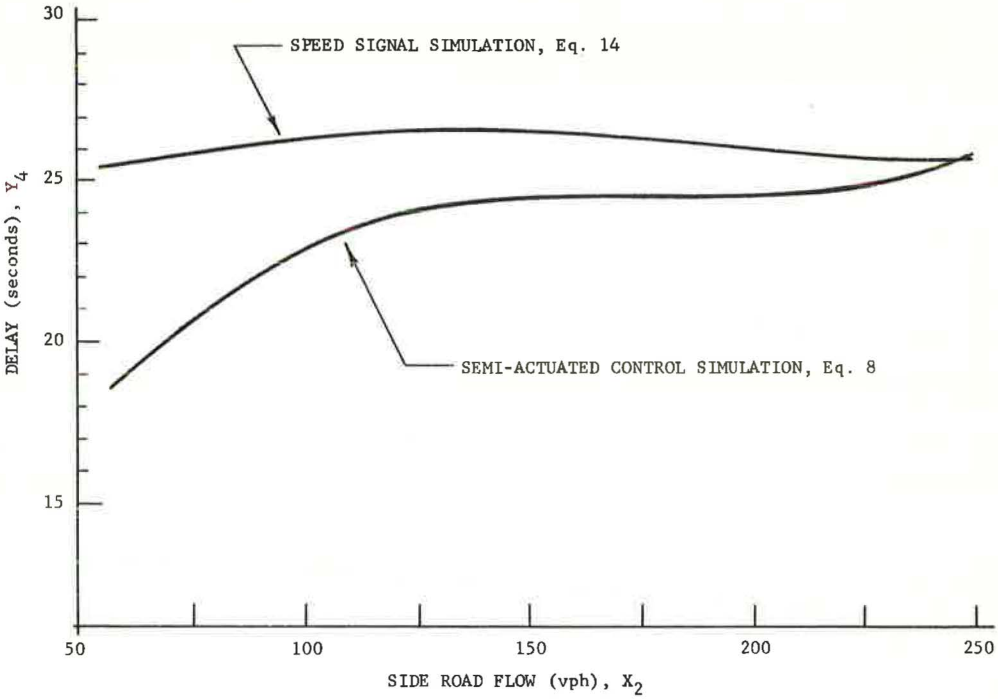

Figure 13. Average delay per side road vehicle stopped, speed signal vs semiactuated control.

with speed signal control is 0.363 which indicates the trend is not as definite as that observed with semiactuated control, where R^2 is 0.800.

Figure 11 concerns the percent vehicles stopped on the main route for the two types of control. Both plots increase with flow; however, the signal funnel averages 19 percentage units less than that indicated for the semiactuated controller. The R^2 value of 0.168 for the speed signal equation indicates a large portion of the variability in this response is random rather than a result of the trend.

The average delay per side road vehicle in Figure 12 shows the speed signal system inflicts higher average delays, especially in the lower flow range. It should be pointed out that above 170 vph the difference between the two curves diminishes to about 2 sec per vehicle.

The delay per side road vehicle stopped is described by Figure 13. This measure also indicates the signal funnel causes longer delays in the lower volumes than does the semiactuated controller. As the side road volume approaches 240 vph, this delay is essentially the same with either type of control.

SUMMARY AND CONCLUSIONS

This investigation has proposed and evaluated a traffic-actuated speed signal funnel for high-speed intersections. The main route speed signal has options available for lower approach speed indications and obtains information regarding side flow from multiple detectors on the side route. Side road volume ranged from 60 to 250 vph, main volume was 180 to 700 vph per lane.

When the advance speed signal was employed, an average of only 2 percent of the main route traffic was forced to stop against the red phase, compared to 21 percent with conventional semiactuated control. Furthermore, the speed advisory system halted only 2.8 vehicles during a typical study of 15 cycles, while the semiactuated controller stopped an average of 25.8 vehicles in that number of cycles. Figures of merit regarding side road delays averaged only 3 sec longer with the speed signal control than with the semiactuated.

The funnel system analyzed in this project is a preliminary design and it is possible that certain modifications would result in further improvement of system effectiveness. For example, several advance speed signals could be placed on the main route, thus providing more control over vehicle approach speed. Certainly, there are other means available for measuring the side route flow conditions.

The author believes the traffic-actuated speed signal concept offers great potential for relieving the access problem on high-speed routes where grade separations cannot be justified due to low side route volume. This control system would limit interruptions to those which were necessary, and would create a minimum disturbance in the flow during these interruptions.

ACKNOWLEDGMENTS

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A Traffic Signal Data Management System for Urban Transportation Planning

BARTON W. FENTON, Franklin County Regional Planning Commission, Ohio

This paper is a summary of research and development conducted by the Transportation Section of the Franklin County Regional Planning Commission directed toward the design and completion of a data management system for traffic signals. In performing one of its main functions, urban transportation planning, the Commission has need of current traffic signal data to determine link service volumes for network traffic assignments, to analyze problem intersections, and to assist in the development of TOPICS programs for municipalities within the study region.

The design of this data management system is predicated on the utilization of a System/360-Model 30 digital computer for data manipulation, data storage, and for the printing of permanent output records. Descriptions of the phasing, timing, hardware, and location of each traffic signal are coded in symbol form and keypunched as input data cards. A system program converts these symbols to recognizable form and generates the printed output record. Procedures for updating are included in the system.

The end result is that all traffic signals in Franklin County are described in the system by jurisdiction (municipality or township). Each agency will receive updated copies of the printed output record for such uses as the planning of routine maintenance programs, the planning and programming of capital improvement projects designed to update each traffic signal to national standards, and the development of signal timing and phasing revisions to improve traffic flow. The output record will also serve as a communications device between the planning agency and the municipalities.

•THE Franklin County Regional Planning Commission (RPC) has as one of its major responsibilities the completion of a transportation plan for Columbus and Franklin County. A major step in the transportation planning process is to develop simulation models for regional travel characteristics. The basis for development of such models is data collected in various inventories related to physical characteristics and use of the major highway network. The magnitude and importance of these data for a region containing 800,000 people justifies development and utilization of electronic computers for data storage and retrieval.

The Transportation Section of RPC has designed and applied a data management system for traffic signals. Data contained in this system will be maintained on a current basis so that staff members can calculate valid link service volumes, analyze critical signalized nodes on the regional highway network, and so that it can be applied to varied uses within other local governmental agencies.

Paper sponsored by Committee on Traffic Control Devices and presented at the 48th Annual Meeting.

This report discusses development of a data management system and applications of the inherent detailed inventory of traffic signals. The system employs a digital computer for processing data and includes procedures for updating. An example is included for illustrative purposes.

SYSTEM DESIGN AND OPERATION

System Hardware

The traffic signal data management system, similar to all data systems developed by RPC's transportation section, is geared to electronic data processing. General processes included within it are shown in Figure 1. Traffic signal data are added to field inventory work sheets (keyed to coding sheets) which are then coded and keypunched.

The system utilizes a 32K IBM System/360-Model 30 digital computer for data processing. Output devices include a 2415 tape drive for use in preparation of magnetic tape storage of data and a 1404 printer for preparation of printed reports containing traffic signal information.

System Programs and Processes

Two system programs are in operation; they were developed to perform the task of building an initial data file and to perform regular updating. The computer language utilized is BAL (basic assembly language).

Input data cards contain alphanumeric symbols which the program, through a series of testing procedures, recognizes and converts to specified formats on the output report. This procedure retains the use of a single 80-column data card but greatly increases program complexity and computer core storage requirements. Data, once processed, are stored on magnetic tape which will be updated at regular intervals depending on data flow.

Organization of data is accomplished by machine sorting data cards prior to input. They are sorted first by jurisdiction and then by intersection code within each jurisdic-

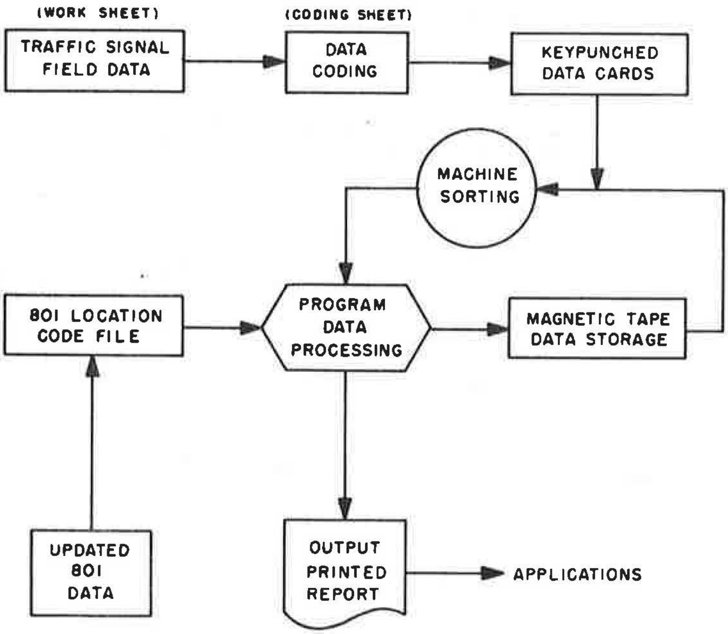


Figure 1. Schematic data flow.

tion. Traffic signals maintained by Ohio Department of Highways, Columbus, and Franklin County are organized by sorting on the "ownership-maintenance" field because of other requirements of the output report format.

SYSTEM INPUT FORMAT AND CODING

General

The underlying objective of this data management system is to provide a complete description of each traffic signal including the location, phasing of traffic movements, associated green times allotted, hardware utilized, and various other facts which are important to the agency charged with maintenance responsibilities.

Field information is, therefore, divided into general intersection data and specific approach leg data. This organization is applied to system input techniques and format as field observations are condensed for transmission to data processing.

Intersection Data Coding

The 80-column data cards are delineated into fields depicting various aspects about each traffic signal. The number of data cards prepared for an intersection is equal to the number of legs forming that intersection. The following portion of the card schematic describes coding techniques used to transform overall intersection field observations to system input data (see Appendix B).

INPUT CARD SCHEMATIC (Entire Intersection)

Even though the study region encompasses only Franklin County at this time, provisions have been made in all data systems to include other counties. Upper case descriptions in parentheses throughout this schematic define the output format of a particular coded item.

Col. 1 - County

Enter the code representing the county in which the signal is located:

1 - Delaware (DEL)
2 - Fairfield (FAI)
3 - Franklin (FRA)
4 - Licking (LIC)
5 - Madison (MAD)
6 - Pickaway (PIC)
7 - Union (UNI)

Cols. 2-4 - Jurisdictions

Enter a code representing the Franklin County municipality or township in which the intersection is located. With two exceptions this is the first three letters of the municipality or township name.

Cols. 5-14 - Location Code

Enter an 801 code (1) for the jurisdiction in which the intersection is located.

Col. 15 - Route System

Enter a code for the highest order system:

S - State highway system (IR, US & SR)
C - County road system
T - Township road system
M - Municipal street system
U - Ohio State University streets
O - Private roads and all others

Col. 16 - Ownership and Maintenance

If the same jurisdiction owns and maintains a traffic signal, enter the proper alpha code:

S - State Highway Department (ODH)
C - Franklin County Engineering Department (FRA)
M - Municipal
P - Private (PRI)
T - Township

Otherwise, enter the respective numeric code:

1 - Municipal owned, county maintained by contract (FRA)
2 - Township owned, county maintained by contract (FRA)
3 - Privately owned, county maintained by contract (FRA)

Col. 17 - Signal System (applies only to Columbus at present)

If the traffic signal is associated with a designated system, enter the corresponding code (if not applicable, leave blank):

1 - Northeast (NE)
2 - Northwest (NW)
3 - CBD
4 - Southeast (SE)
5 - Southwest (SW)
6 - Cycle selector (CS)

Cols. 18 & 19 - Installation Year

Code the last two digits of the year during which the signal was installed.

Cols. 20 & 21 - Revision Year

Code the last two digits of the year during which the signal was last revised.

Col. 22 - Power Company

Enter a code for the company supplying current to the signal:

1 - Columbus Municipal Electric Light Plant (MUNI)
2 - Columbus and Southern Ohio Electric Co. (CSOE)
3 - South Central Power Co. (SCPC)

Col. 23 - Type Power

Code the type of contract under which current is supplied:

M - Metered
F - Flat rate

Col. 24 - Number of Legs

Code the number of legs at the intersection.

Col. 25 - Number of Signal Heads

Code the total number of signal heads used to display the various indications for the intersection as a whole (a two-way adjustable head is coded as one even though two one-way heads are connected):

1 to 9 heads, code direct; A = 10, B = 11, C = 12, D = 13, E = 14

Col. 26 - Number of Pedestrian Units

Code the total number of pedestrian units present (if none, leave blank).

Col. 27 - Number of Signal Phases

Code the number of signal phases used to control traffic movements through the intersection. Right-turn overlaps are not to be considered as separate signal phases. Only left-turn and through movements comprise phases.

Col. 28 - Mode of Operation

Enter a code describing operation of the signal:

P - Pre-timed (FIXED)
S - Semi-actuated (SEMI)
F - Full-actuated (FULL)
T - Part-time (PTIME)
C - Semi-actuated ped signal (PEDS)
V - Volume-density (VDENS)

Col. 29 - Type of Coordination

Enter a code describing the type of coordination under which the signal operates:

I - Isolated
P - Pre-timed (Master)
T - Traffic-adjusted (Master)
O - Other (Timer)

Cols. 30 & 31 - Manufacturer's Name

Code the name of the controller manufacturer:

AS - Automatic Signal
CH - Crouse-Hinds
EA - Eagle
EC - Econolite
FP - Fisher & Porter
GR - General Railway Signal
MA - Marbelite
TS - Traffic Signals, Inc.
WM - Winko-Matic Signal
GA - Gammatronix
GE - General Electric
DA - Darley (Tokheim)

Cols. 32-39 - Controller Code Number

Enter the coded controller identification assigned by the manufacturer.

Approach Leg Data Coding

The second half of each input card is designed for coding leg data according to the following format.

INPUT CARD SCHEMATIC (Each Approach Leg)

Col. 40 - Approach Direction

Enter a code representing the direction of travel on this approach:

N - Northbound (NB)
S - Southbound (SB)
E - Eastbound (EB)
W - Westbound (WB)

Col. 41 - Number of Signal Faces

Code the number of signal faces displayed to traffic on this approach.

Cols. 42-44 - Basic Description

Enter each code which describes a different type of signal face that is present (maximum of three). (Arrow types and sizes are described in cols. 45-48; consider through arrows as green indicators.)

1 - 8 inch standard (RYG)
2 - 12 inch standard (RYG)
3 - 12 inch red only, remainder 8 inch
4 - 12 inch green only, remainder 8 inch
5 - 8 inch standard "T"-head
6 - 12 inch green only, remainder 8 inch "T"
7 - 8 inch standard with arrow
8 - 12 inch standard with arrow
9 - 8 inch single green
0 - 12 inch single green
A - 8 inch four section
B - 12 inch four section
C - 8 inch six unit "T"
E - Pedestrian unit

Col. 45 - Number of Turn Arrows

Code the total number of turn arrows facing this approach.

Cols. 46-48 - Description

Enter each code which describes a different type of turn arrow facing this approach (maximum of three).

1 - 8 inch left
2 - 12 inch left
3 - 8 inch right
4 - 12 inch right

Col. 49 - Possible Movements Through Approach Leg

1 - No movements provided (one-way in opposite direction)
2 - Through only
3 - Right only
4 - Left only
5 - Through and right
6 - Through and left
7 - Right and left
8 - All movements
9 - Pedestrian crossing only

Col. 50 - Number of Approach Leg Phases

Enter the total number of approach phases including left turn auxiliary phases.

Col. 51 - Parent Phase Movements

Enter a code as defined for column 49 which describes the movements controlled by the parent phase (basic from controller).

Cols. 52-55 - Auxiliary Phase Description

Enter a pair of codes or code that describes auxiliary turn phases (auxiliary phases may or may not be controlled by special equipment) for traffic on this approach (enter left description first):

AL - Advance left only
AT - Advance left with through
A5 - Advance left with through and right
AR - Advance right only
OR - Overlapping right
LL - Lagging left only
LT - Lagging left with through
L5 - Lagging left with through and right
LR - Lagging right only

Cols. 56-58 - Total Cycle Time

Enter total elapsed time (seconds) for one complete sequence of signal phases.

Cols. 59 & 60 - Parent Phase Greentime

Enter total greentime (seconds) if no auxiliary phases are provided. If auxiliary turn phases exist, enter (in column 51) only the greentime allotted to traffic movements, right-adjusted with leading zeros.

Cols. 61 & 62 - Left Turn Phase Greentime

Enter greentime (seconds) for a left turn phase movement, right-adjusted with leading zeros.

Cols. 63 & 64 - Right Turn Phase Greentime

Enter greentime (seconds) for a right turn phase movement, right-adjusted with leading zeros.

Cols. 65-70 - Detector(s) and Movements Detected

Enter code(s) for type of detector(s) and traffic movements detected:

Cols. 65, 66, 68 & 69 - Type of Detector

RO - Radar overhead (RADOVHD)
RS - Radar sidefire (RADSIDE)
PR - Pressure (PRESSUR)
LP - Loop presence (LOOPPRE)
LM - Loop motion & multi-lane (LOOPARE)
LL - Loop motion & lane (LOOPLNE)
PB - Pedestrian push button (PUSHBUT)
MP - Magnetic presence (MAGPRES)
MM - Magnetic motion & multi-lane (MAGAREA)
ML - Magnetic motion & lane detection (MAGLANE)
UO - Ultrasonic overhead (ULTROVHD)
US - Ultrasonic sidefire (ULTSIDE)
CD - Call detector (CALLDET)

Cols. 67 & 70 - Movements Detected

Refer to definitions under column 49 for the appropriate code.

Cols. 71-73 - ODH Street Number

Enter a street number for the principal street (from the 803 or 820 File). Right-adjust and add leading zeros.

Cols. 74-77 - Log Station

Enter a log station from the road and street inventory field sheets, right-adjusted with leading zeros.

Cols. 78-80 - Data Management System Number

Code the number 830.

Coding techniques resolve the basic problem of condensing field data to a form more amenable to system procedures. In addition, they provide more efficient use of the input data card and can often preclude the use of "trailer cards." However, as illustrated in the schematic, this creates a complex set of coding definitions. Due to initial file size (3200 cards), our choice was to maintain a single card record.

Illustrative Example

For illustrative purposes, assume intersection X-Y to be in Clinton Township, Franklin County. Traffic movements through the intersection are controlled by an Eagle Signal, EF20 controller which is not interconnected. Installation date was 1952 and in 1967 the Franklin County Engineer, who maintains the signal by contract, added signal faces to bring the installation up to national standards. The signal faces, phasing, and timing are as shown in Figure 2. Power is purchased from Columbus and Southern Ohio Electric Company.

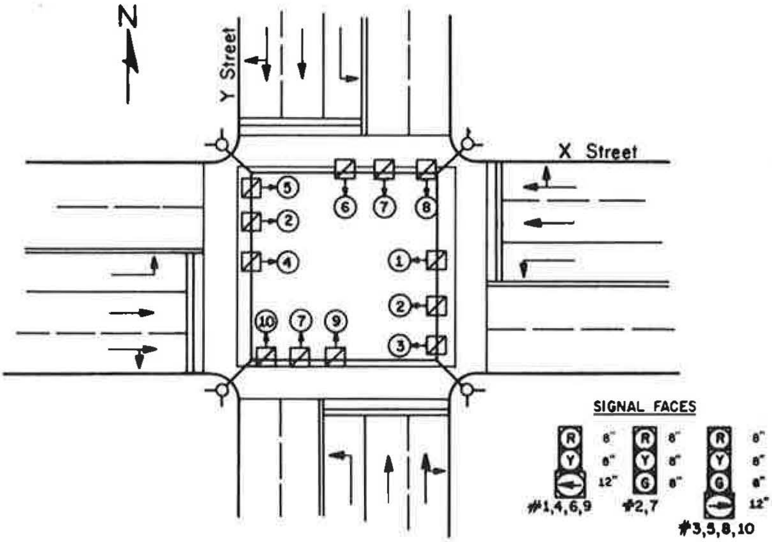
A complete field work sheet representing the installation shown in Figure 2 is shown in Appendix A. Input data have been transposed from this work sheet to the coding sheet shown in Appendix B. At this point keypunching can take place after which the data are routed through the program for storage and output.

SYSTEM OUTPUT FORMAT

General

As mentioned previously, coded input data are converted to recognizable form by the system program before an output report is generated.

Printed output sheets are formatted to facilitate understanding by individuals with varied technical backgrounds who will be using them. The principal underlying design of the printed report is to provide easy recognition of all aspects of its format. Simplicity must be maintained so that maximum use will be made of the reports thereby returning a large initial investment and generating the interest necessary to prompt regular submission of updated information.



SIGNAL DISPLAY & PHASING SCHEDULE — TOTAL CYCLE = 90 sec.

PHASE & TIME	A (9 sec.)		B (27 sec.)		C (9 sec.)		D (31 sec.)	
SIGNAL #	MVT.	SIG. DISP.	MVT.	SIG. DISP.	MVT.	SIG. DISP.	MVT.	SIG. DISP.
1	↰	←	—	R	—	R	—	R
2	—	R	↑	G	—	R	—	R
3	—	R	↑→	G	↱	R →	—	R
4	↰	←	—	R	—	R	—	R
5	—	R	↑→	G	↱	R →	—	R
6	—	R	—	R	↰	←	—	R
7	—	R	—	R	—	R	↑	G
8	↱	R →	—	R	—	R	↑→	G
9	—	R	—	R	↰	←	—	R
10	↱	R →	—	R	—	R	↑→	G

EAGLE EF20 CONTROLLER — 4 PHASE QUAD LEFT TURN

Figure 2. Traffic signal example.

Alphanumeric Conversions

Coded input data are converted to output specifications according to Tables 1 and 2.

Table 1 lists the various types of signal faces and their corresponding printed output descriptions. Other combinations are possible and will be generated from the information given if they are required.

Table 2 lists possible parent and auxiliary phase movements with their specified output conversions for a given approach leg. This information will enable leg-by-leg calculation of service volumes once proper greentime ratios are known.

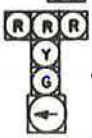
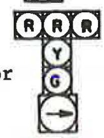
Information in these tables will resolve questions concerning interpretation of the data output.

The Printed Report

Organization of the printed report includes column headings printed only once at the top of each page. Each output list is structured to show intersection information on the first line and specific approach leg data on succeeding lines, one line per approach. Appendix C shows output for the previously given example. A new page is started for each jurisdiction, thereby separating data for each governmental agency.

TABLE 1

SIGNAL FACE CONVERSIONS

Type	Code	Description	Printed Output Format BASIC	Printed Output Format TURN ARROW
	1	8" standard	3SECT8	
	2	12" standard	3SECT12	
	3	8" with 12" red	3SECT12R	
	5	8" standard "T"	5UNIT8	
 or 	9	8" single green	1SECT8	8LT or 8RT
 or 	0	12" single green	1SECT12	12LT or 12RT
 or 	A	8" standard with 12" left or right arrow	4SECT8	12LT or 12RT
 or 	B	12" standard with 12" left arrow	4SECT12	12LT or 12RT
 or 	C	8" standard "T" with 12" left or right arrow	6UNIT8	12LT or 12RT
DONT WALK	E	Pedestrian "walk" unit	PED-SIG	

Intersecting street names are printed from the 801 intersection code file (1) on magnetic tape which is interfaced with the traffic signal data management file. When an intersection code contained in columns 3 through 14 is matched to the same code in the 801 file, then street names are printed directly from the 801 file. If no match is possible, an entry in column 5 is tested for a match in Table 3. A match in Table 3 permits conversion of the alpha code and a corresponding message is printed in the location name field. This method is utilized to process mid-block signal installations. The condition of "no match" in either the 801 file or Table 3 must be resolved manually because it is symptomatic of an input error.

TABLE 2

MOVEMENT CONVERSIONS

Movements	Description	Code	Printed Output Format
	Parent Phase		
	One-way in opposite direction	1	NONE-ONE WAY IN OPPOSITE DIRECTION
	Through only	2	THRU
	Right only	3	RGHT
	Left only	4	LEFT
	Through and right only	5	TH & RT
	Through and left only	6	TH & LT
	Right and left only	7	RT & LT
	ALL	8	ALL
	Ped. crossing only	9	PEDS
	Auxiliary Phase		
	Advance left	AL	ADV. LEFT
	or		
	Lagging left	LL	LAG. LEFT
	Advance left with through	AT	ADV. LT W/THRU
	or		
	Lagging left with through	LT	LAG. LT W/THRU
	Advance left with through and right	A5	ADV. LT W/TH & RT
	Lagging left with through and right	L5	LAG. LT W/TH & RT
	Advance right	AR	ADV. RGHT
	or		
	Lagging right	LR	LAG. RGHT

Illustrative Example

Using the same traffic signal installation in Figure 2, the resulting printed report is included as Appendix C. The next signal listed in the report would not have column headings but would be printed according to identical format specifications.

Since the total value of any management tool is considerably reflected by its degree of utility to various groups, it is appropriate to conclude with a discussion of the applications of this data management system.

TABLE 3

MID-BLOCK LOCATION CONVERSIONS

County	Juris.	Code	Printed Output Format
3	GRO	A	E. MAIN ST AT GRO-MAD HIGH SCHOOL EXIT
3	MIF	B	SUNBURY RD AT OHIO DOMINICAN PKG LOT
3	COL	C	PARSONS AV AT AM. BLOWER EXIT
3	COL	D	E. MAIN ST AT XWALK WEST OF ALLEN
3	COL	E	LOCKBOURNE RD AT XWALK SO. OF SMITH
3	COL	F	E. LIVINGSTON AV AT XWALK EAST OF HEYL
3	COL	G	W. MOUND ST XWALK WEST OF WHEATLAND
3	COL	H	GREENLAWN AV AT FIRE COMPLEX EXIT
3	COL	I	W. FIFTH AV AT XWALK WEST OF EDGEHILL
3	COL	J	N. HIGH ST AT BLIND SCHOOL XWALK
3	COL	K	INDIANOLA AV AT XWALK OLYMPIC POOL
3	COL	L	E. FIFTH AV AT TIMKEN XWALK
3	COL	M	E. WEBER RD AT XWALK WEST OF REIS

APPLICATIONS

Transportation Study Agency

Network Analysis—Now that RPC's traffic signal data management system is in operation, analysis of the street network will be the next step. Detailed knowledge of traffic signals is of considerable importance to transportation planners primarily because network analysis and evaluation can then proceed based upon known factors and not upon "scientific" assumptions. Valid information on signal phasing and timing at a signalized intersection, for instance, permits determination of reliable greentime percentages for traffic movements. Therefore, calculation of intersection service volumes and the important process of existing network evaluation can proceed within an acceptable level of confidence, void of doubts typically created by the nearly universal insufficiency of data.

A regional traffic signal data file also permits analysis of traffic flow along a continuous route, combining signal information for all municipalities along that route. Data can be arrayed along a continuous route (from west to east or south to north) by sorting the file on street number and log station. Experience indicates that municipal boundaries, inadvertently, represent increases in travel friction along a major arterial. This happens because some municipalities provide progressive movement through signal interconnection and others allow their signals to operate independently for economic or other reasons. Traffic engineers, once given continuous route signal data, can assist smaller municipalities toward achieving eventual interconnection with the signal system of an adjacent central city.

TOPICS—The 1968 Federal Highway Act sets the stage for financial assistance in fiscal years 1969 and 1970 to implement TOPICS (Traffic Operations Program to Increase Capacity and Safety) on city streets. Some types of improvements eligible for aid under TOPICS include: the installation of traffic signal control systems, upgrading of traffic control signs and signals, and various other types of improvements to urban street networks. In many cases, transportation study staffs will act as regional coordinators for the planning of improvements to be financed under TOPICS, thereby requiring specific information as to the present status of traffic signals. Given such information, they can make preliminary recommendations to community officials on those signal improvements required to increase street capacity and safety. RPC's traffic signal data file is organized by municipality and is, by design, well suited to use in the TOPICS program for central Ohio.

Municipalities

Even though the traffic signal data file is designed primarily for planning applications, the needs of municipal applications have been incorporated into file development. Each governmental agency charged with traffic signal maintenance responsibilities will receive a list of its traffic signals, oriented toward ready evaluation of these data so that needed improvements can be incorporated into annual capital programs.

A quick review of the printed output will indicate which traffic signal installations do not meet national standards requiring a minimum of two signal faces per intersection approach. This knowledge of existing signal hardware will lead to the reliable tabulation of additional equipment needed to conform to such standards.

Because complete records on each traffic signal are a prerequisite to efficient maintenance operations, information in the signal file will be of considerable use in developing traffic signal maintenance programs. For instance, priorities of maintenance items can be established based on the type of route system, functional classifications of intersecting streets, or any one of the parameters contained in the file. A preventive maintenance program for actuated equipment would make use of those items describing controller type and detection equipment as well as signal location.

As municipalities become involved in TOPICS programs, the signal data file will assist in the establishment of program priorities, equipment refinements, and other project guidelines. Even though the full impact of this file is, at this time, somewhat speculative, one conclusion is apparent: transportation planners, traffic engineers, and city officials in Franklin County will have a common framework providing the impetus for cooperative efforts toward improving regional transportation facilities.

ACKNOWLEDGMENTS

The author gratefully acknowledges the work of the data processing group within the Transportation Section of the Franklin County Regional Planning Commission. Special appreciation goes to Ervin J. Erlanger, Systems Coordinator, whose relentless efforts have made the signal data management system a reality and to Eileen Flowers for her valuable data processing skills. All work was conducted under the auspices of William C. Habig, Chief Transportation Planner.

Additional acknowledgments are extended to the Columbus Division of Traffic Engineering and Parking for assistance in the field inventory of the traffic signals within the City of Columbus, to the Franklin County Engineers' Office and to Division Six of the Ohio Department of Highways.

The Subcommittee on Regional Data Management of the Technical Advisory Committee is responsible for providing coordination necessary to insure that the data items included comprise a set which is acceptable and useful to all traffic engineering agencies within the county.

Preparation of this publication was financed by appropriations from Franklin County, Municipalities in Franklin County, and other local public and private agencies together with planning funds from the U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads in conjunction with the Ohio Department of Highways.

REFERENCE

1. Habig, W. C., and Erlanger, E. J. Location Coding in the Central Ohio Region. Franklin County RPC, Aug. 1968.

Appendix A

830 FILE--TRAFFIC SIGNAL INVENTORY

WORK SHEET

PART I--INTERSECTION DATA

Location X STREET at Y STREET County (1) FRA

Jurisdiction (2-4) CLI 801 Intersection Code (5-14) 2618 - 3759

Jurisd. Route System (15) COUNTY Ownership & Maint. (16) 2 System (17) ——

Year Installed (18-19) 57 Year Revised (20-21) 67 Power Co. & Type (22-23) CSOE-F

No. of Legs (24) 4 No. of Sig. Heads (25) 12 No. of Ped Units (26) 8

No. and Description of Signal Phases (27) 4 PHASE QUAD LEFT TURN

Mode of Controller Operation (28) PRETIMED Type CO-ORD (29) ISOLATED

Controller Manufacturer (30-31) EAGLE SIGNAL Code No. (32-39) EF 20

ODH Principal Street No. (71-73) 016 Log Station (74-77) 06.69

Additional Information:

PART II--LEG DATA

Card Column	Data	APPROACH DIRECTION (40)			
		Northbound on Y STREET	Southbound on Y STREET	Eastbound on X STREET	Westbound on X STREET
cc 41	No. of Traffic Sig. Indications	3	3	3	3
cc 42 to 44	Description of Indications: Lens size, no.of Sections, etc.	8" 3 SECTION 8" 4 SECTION	8"-3 SECTION 8"-4 SECTION	8"-3 SECTION 8"-4 SECTION	8"-3 SECTION 8"-4 SECTION
cc 45	No. of Arrows	2	2	2	2
cc 46 to 48	Type & Size of Turn Arrows	12" LEFT & 12" RIGHT	12" LEFT & 12" RIGHT	12" LEFT & 12" RIGHT	12" LEFT & 12" RIGHT
cc 49	Possible Traffic Movements on Approach Leg	ALL	ALL	ALL	ALL
cc 50	Number of Ø's	2	2	2	2
cc 51	Parent Phase Movements	THRU & RIGHT	THRU & RIGHT	THRU & RIGHT	THRU & RIGHT
cc 52 to 55	Description of Mvts. on Aux. Turn Ø's	ADVANCE LEFT LAGGING RIGHT	ADVANCE LEFT LAGGING RIGHT	ADVANCE LEFT LAGGING RIGHT	ADVANCE LEFT LAGGING RIGHT
cc 56 to 58	Total Cycle Length (sec)	90	90	90	90
cc 59 to 60	Basic or Parent Green	31	31	27	27
cc 61 to 62	Left Turn Phase Green	09	09	09	09
cc 63 to 64	Right Turn Phase Green	09	09	09	09
cc 65 to 70	Type Veh. Detectors and Mvts. they Detect	—	—	—	—
	COMMENTS				

Recorder: BWF Date: 8/19/68

Franklin County Regional Planning Commission
Central Ohio Transportation Study
6/28/68

TRAFFIC SIGNAL INVENTORY (830 FILE)

CODER: BWF
DATE: 8/19/68

INTERSECTION DATA

COUNTY (1)	JURISDICTION (2-4)	801 FILE INTERSECTION CODE / URBAN LOC. CODE (5-9)	STATION NO. (10-14)	JURIS. SYS. (15)	OWNERSHIP (16)	COL. SYS. (17)	INSTALL YEAR (18-19)	LATEST REV YEAR (20-21)	CUR-RENT BY (22)	CUR-RENT TYPE (23)	NO. OF LEGS (24)	NO. SIG. HEADS (25)	NO. PED UNITS (26)	CONTROLLER INFORMATION: NO. PHASES (27)	MODE (28)	TYPE COORD. (29)	MANUFACTURER NAME (30-31)	MANUFACTURER CODE NO. (32-39)
3	CL1	2618	3759	C	2		57	67	2	F	4	C	8	8	F	I	EA	EF20
3	CL1	2618	3759	C	2		57	67	2	F	4	C	8	8	F	I	EA	EF20
3	CL1	2618	3759	C	2		57	67	2	F	4	C	8	8	F	I	EA	EF20
3	CL1	2618	3759	C	2		57	67	2	F	4	C	8	8	F	I	EA	EF20

LEG DATA / SPECIAL IDENT.

APPR. DIR. (40)	SIGNAL INDIC. NUMBER (41)	BASIC CROSS SECTION (42-44)	TURN ARROWS NO. (45)	TURN ARROWS DESC. (46-48)	MVT. THRU LEG (49)	APPR. Ø'S NUMBER (50)	PAR. Ø MVT. (51)	AUXIL. PHASES (MAX. OF 2 DESC.) (52-55)	TIMING (SEC.) TOTAL CYCLE TIME (56-58)	GREENTIME (59-60)	AUX. L.T. Ø GREENTIME (61-62)	AUX. RT. Ø GREENTIME (63-64)	DECTECTORS TYPE 1 TYPE DET. (65-66)	TYPE 1 MVT. (S) (67)	TYPE 2 TYPE DET. (68-69)	TYPE 2 MVT. (S) (70)	ODH IDENT. (INTERS. INVENT.) STREET NO. (71-73)	LOG STATION (74-77)	CARD NO. (78-80)
N	3	1	2	24	8	2	5	ALLR	090	31	09	09					016	0669	830
S	3	1	2	24	8	2	5	ALLR	090	31	09	09					016	0669	830
E	3	1	2	24	8	2	5	ALLR	090	31	09	09					016	0669	830
W	3	1	2	24	8	2	5	ALLR	090	31	09	09					016	0669	830

CENTRAL OHIO TRANSPORTATION STUDY
FRANKLIN COUNTY REGIONAL PLANNING COMMISSION 7/1/68

FRANKLIN COUNTY REGIONAL PLANNING COMMISSION
CENTRAL OHIO TRANSPORTATION STUDY
TRAFFIC SIGNAL INVENTORY
DATA MANAGEMENT FILE (NO.830)

DATE 08/19/68

CTY	JURIS	INTERSECTION OR MIDBLOCK LOCATION	JUR SYS	MAINT	COL SYS	YR. INS	YR. REV	CURRENT BY TYPE	L G S	H D S	P E D	P H S	MODE	CONTROLLER INF. O R D	A M E	CODE NUMBER	ODH STR NO.	ODH LOG STATION
FRA	CLI	ST Y ST	C	FRA		57	67	CSOE F	4	12	8	4	FIXED	I	EA	EF20	016	06.69

APPR. DIR.	NO. SIG. IND	SIZE & TYPE OF SIGNAL	SIZE & TYPE OF INDICAT.	TURN-ARROWS NO.	TURN-ARROWS SIZE & TYPE		MVMT. THRU LEG	NO.OF APPR. PHSES	PARENT PHASE MVMTS.	AUXILIARY PHASE MOVEMENT		CYCLE LNGTH (SEC)	GREEN TIME (SEC)	AUX LFT GRN	AUX RGT GRN	TYPE OF DETECT	MVMTS DETC.	TYPE OF DETECT	MVMTS DETC.
NB	3	3SECT8	4SECT8	02	12LT	12RT	ALL	2	TH&RT	ADV.LEFT	LAG.RGHT	90	31	09	09				
SB	3	3SECT8	4SECT8	02	12LT	12RT	ALL	2	TH&RT	ADV.LEFT	LAG.RGHT	90	31	09	09				
EB	3	3SECT8	4SECT8	02	12LT	12RT	ALL	2	TH&RT	ADV.LEFT	LAG.RGHT	90	27	09	09				
WB	3	3SECT8	4SECT8	02	12LT	12RT	ALL	2	TH&RT	ADV.LEFT	LAG.RGHT	90	27	09	09				

Evaluation of Minor Improvements

CHARLES G. HAMMER, JR., California Division of Highways

This investigation evaluated the effectiveness of left-turn channelization in reducing reported accidents. The before-and-after study method was used to evaluate 53 left-turn channelized intersections. Warrants for this type of improvement were developed and accident predictive parameters investigated. The effectiveness of specific sign installations in reducing reported accidents was also investigated. The before-and-after study method was used to evaluate 34 sign installations. Accident predictive methods were investigated for curve warning, advisory speed, special combination advisory speed, and 4-way stop sign installations.

•ALTHOUGH new or improved freeway systems are decreasing the pressure on the presently overtaxed streets and roads, the fact remains that the motorist must drive on the conventional road system for at least part of his travel. The California Division of Highways, for many years, has channeled certain funds into a "minor improvement program" to increase safety to the conventional road user.

The Evaluation of Minor Improvements study was designed to develop objective criteria for the evaluation of minor improvements, and thereby permit maximum safety benefits per dollar spent in the minor improvement program. The objectives of the overall study are as follows:

1. To determine the effectiveness of minor improvements in reducing traffic accidents.
2. To determine what conditions are susceptible to improvement and how much improvement can be expected.
3. To determine methods and measures for predicting the magnitude of the accident reduction on proposed minor improvement projects.
4. To review present improvement warrants for validity and adequacy and to determine if new warrants are required.

METHODOLOGY

Before and after periods of equal length were compared. To avoid bias due to seasonal fluctuations in accidents, the same number of each calendar month was used in each pair of before and after periods when fractional parts of a year were used (e.g., May 1961 to December 1962-before, and May 1963 to December 1964-after).

The periods used were, insofar as possible, immediately before and after the improvement construction to reduce the influence of any general trend in accident rates. An investigation of a possible increasing or decreasing trend showed no such phenomenon.

The before and after accidents and accident rates were compared. The "accident rate" is simply the number of accidents related to vehicle exposure (the total entering volume when considering intersections). When accident breakdowns are shown in tables, fatal and injury accidents were combined since the number of fatal accidents were few.

Paper sponsored by Committee on Operational Effects of Geometrics and presented at the 48th Annual Meeting.

SIGNIFICANCE TESTING

The chi-square test was generally employed to establish whether the reductions in accidents were statistically significant (1). Generally a confidence level of 0.10 was used. In other words, any significant difference would not be expected to have occurred by chance more than ten times out of one hundred.

The statistical reliability of observed changes is often indicated by $P < 0.10$. This means that we can be 90 percent confident that the difference observed was a true difference and not one due to random sampling fluctuations.

Because of relatively few (generally less than 20) accidents occurring in the before period for any one location, a reduction even as high as 50 percent for the after period is rarely statistically significant. That is, for such a small sample, this amount of reduction could have occurred because of chance variation. Therefore, under these conditions, the hypothesis that the highway improvement caused the accident reduction cannot be accepted with confidence. However, a large sample which is the sum of several projects may show a significant reduction from before to after because of the added power due to the increased sample size.

LEFT-TURN CHANNELIZATION

The purposes of left-turn channelization are (a) to increase intersection capacity by removing stopped vehicles awaiting a left-turning opportunity; (b) to provide a refuge for the stopped vehicle from rear-end accidents; and (c) possibly to provide refuge at the islands for pedestrians crossing the highway. This channelization can be accomplished by painted stripes, raised bars with painted outline, and curbs.

The protection for the left-turning vehicle consists of a bulbing of the channelization upstream of the storage pocket.

The intersections with raised-bar channelization and those with curbed channelization were combined for analysis since, in both cases, a positive physical barrier was used to protect the left-turning vehicles. Also, the accident reductions noted were approximately the same. The two groups used are paint and physical protection (raised bars and curbs).

The relative effectiveness of the channelization was judged solely on an accident basis. This assumes that no appreciable difference in design policy existed between the respective types of channelization. The assumption appears to be valid since for many years California has used the same equation to determine the required length of the storage lane.

Fifty-three left-turn channelized intersections were evaluated. While only 10 intersections showed a statistically significant reduction, 42 of these intersections did experience some reduction in accident rates.

Left-turn channelization as a whole has been effective (34 percent reduction) with a 48 percent reduction at unsignalized and 17 percent reduction at signalized intersections.

TABLE 1

LEFT-TURN CHANNELIZATION PROJECT SUMMARY

Channelization	No. Projects	Accidents		Percent Change
		Before	After	
Unsignalized.				
Paint	27	157	106[s]	32
Physically protected	13	156	56[s]	64
Subtotal	40	313	162[s]	48
Signalized:				
(No L. T. ↑)	13	283	234[s]	17
Total	53	596	396[s]	34

[s]Significant at 0.10 level using the chi-square test.

TABLE 2

LEFT-TURN CHANNELIZATION

Intersection	Sing. Veh.	Multi. Veh.	PDO	I+F	Day	Night	Total	
13 Signalized (284 MV):								
Before	19	264	176	107	178	105	283	37.8
After	25	209	136	98	140	94	234	41.9
40 Unsignalized (275 MV):								
Before	29	284	182	131	203	110	313	42
After	28	134	104	58	94	68	162	36

Painted left-turn channelized projects experienced a 32 percent accident reduction, and physically protected intersections had a 64 percent reduction in accidents (Table 1).

Pedestrian accidents were few (dropping slightly from 20 to 14 for all 53 projects) and are not discussed further.

Accidents in lieu of accident rates are tabulated throughout the report (except as noted) since the exposure remained essentially the same—not having increased more than 10 percent in any comparison group in the after period. The chi-square calculations are, therefore, based on an assumption of equal volumes in both periods.

A summary of the accident breakdown for 13 signalized (with no left-turn phase) and 40 unsignalized intersections is given in Table 2. Generally, when left-turn channelization is provided at signalized intersections on state highways, a signal phase is also provided for the movement. The small sample size available of this unphased movement is an indication of this policy.

The signalized intersections are highly urbanized and have high entering volumes. Total accidents were reduced in both groups (signalized, $P < 0.05$; unsignalized, $P < 0.001$) with greater reductions at unsignalized intersections. Channelization did not appreciably reduce night accidents at signalized (lighted) intersections.

Single-vehicle accidents remained about the same. Multiple-vehicle accidents were reduced for both categories although multiple-vehicle accidents ($P < 0.001$) at unsignalized intersections were reduced more than at signalized intersections ($P < 0.02$). Even greater reductions would have been realized if crossing or broadside accidents in the unsignalized category had not increased. These broadside accident increases occurred at urban painted and physically protected intersections.

Left-turn and rear-end accidents are given in Table 3 with a signalized and unsignalized intersection breakdown.

At the signalized intersections, the left-turn accidents were reduced although rear-end accidents increased slightly. This reduction apparently occurred during the daytime with nighttime accidents remaining about the same. All of these signalized intersections had safety lighting although the level of illumination is not known. It is also interesting to note that the more severe accidents (injury and fatal) remained the same with a significant decrease in the PDO ($P < 0.05$) category.

TABLE 3

LEFT-TURN AND REAR-END ACCIDENTS

Intersection	Left Turn	Rear End	PDO	I+F	Day	Night	LT+RE
13 Signalized:							
Before	101	92	125	68	121	72	193
After	46	106	90	62	88	64	152
40 Unsignalized:							
Before	52	164	122	94	139	77	216
After	33	24	33	24	36	21	57

TABLE 4

PAINT VS PHYSICALLY PROTECTED LEFT-TURN CHANNELIZATION (Unsignalized)

Intersection	Sing. Veh.	Multi. Veh.	PDO	I+F	Day	Night	Total
27 Ints.—Paint (134± MV):							
Before	15	142	84	73	98	59	157
After	18	88	64	42	58	48	106
13 Ints.—Physical Protection (140± MV):							
Before	14	142	98	58	105	51	156
After	10	47	40	16	36	20	56

At the 40 unsignalized intersections, both left-turn ($P < 0.10$) and rear-end ($P < 0.001$) accidents were reduced with greater reductions noted in rear-end accidents (85 percent). All severity classes were reduced, with similar reductions noted ($P < 0.001$) in injury and fatal accidents (-74 percent) and in PDO accidents (-73 percent). This is similar to the day-night breakdown.

NONSIGNALIZED INTERSECTIONS

Effect of Physical Protection

Data were available for 27 painted left-turn channelized intersections and 13 physically protected left-turn channelizations. Although the latter group was considerably smaller in number, the total number of accidents in both groups in the before period is approximately the same as is the total exposure (Table 4).

Both groups of intersections show statistically significant reductions in total accidents ($P < 0.01$). The group of intersections with the physically protected channelization had twice the accident reduction as did those with paint.

Effect of Urbanization

The unsignalized intersections were divided into urban and rural groups (Table 5). When the relative exposures are applied to the total accidents, the accident rates are almost identical.

Painted channelization accident experience was divided into urban and rural intersections as shown in Table 6.

One thing that stands out immediately is that despite equal exposure and accidents in the rural and urban groups, painted channelization yields much greater accident reductions (of all types) in rural areas than in urban areas. Total rural accidents were significantly reduced ($P < 0.02$) as were rural multiple vehicle, PDO, I + F, and daytime accidents, whereas no type of accidents (shown in this table) were significantly reduced in the painted urban channelization groups. The data indicate that much greater accident reductions can be expected from painted left-turn channelization in rural areas than in urban areas.

In urban painted channelization although the total number of accidents dropped only 15 percent (79 to 67), left-turn and rear-end accidents (susceptible to correction) were halved from 52 to 25 (Table 7), thus indicating success in protecting the left-turning vehicles.

TABLE 5

ACCIDENT RATES OF URBAN-RURAL INTERSECTIONS

Time	Urban	Rural
Before	1.16	1.17
After	0.57	0.55

Physically protected left-turn channelization is divided into rural and urban groups in Table 8. Both urban and rural intersections show significant reductions ($P < 0.01$) in total accidents when physically protected left-turn channelization is used. Both painted ($P < 0.10$) and physically protected ($P < 0.01$) urban intersections have significant broadside accident increases. These accident increases are given in Table 9 with a further breakdown of the type or location of crossing accidents by direction of travel of the crossroad vehicle.

TABLE 6

PAINTED CHANNELIZATION (URBAN—RURAL)

Location and Time	Sing. Veh.	Multi. Veh.	PDO	I+F	Day	Night	Total
12 Ints.—Urban (46 MV):							
Before	8	71	43	36	45	34	79
After	8	59	43	24	33	34	67
15 Ints.—Rural (67 MV):							
Before	7	71	41	37	53	25	78
After	10	29	21	18	25	14	39

TABLE 7

PAINTED CHANNELIZATION:
LEFT-TURN AND REAR-END ACCIDENTS

Location and Time	Left Turn	Rear End	PDO	I+F	Day	Night	LT+RE
Urban:							
Before	18	34	29	23	30	22	52
After	14	11	14	11	11	14	25
Rural:							
Before	19	34	26	27	35	18	53
After	10	6	8	8	14	2	16

TABLE 8

PHYSICALLY PROTECTED LEFT-TURN CHANNELIZATION

Location and Time	Sing. Veh.	Multi. Veh.	PDO	I+F	Day	Night	Total
9 Ints.—Urban (40± MV):							
Before	10	100	70	40	79	31	110
After	7	29	26	10	26	10	36
4 Ints.—Rural (40±MV):							
Before	4	40	28	18	26	20	46
After	3	17	14	6	10	10	20

TABLE 9

URBAN BROADSIDE ACCIDENTS

Channelization	Type A		Type B		Total	
	Before	After	Before	After	Before	After
Painted	5	11	9	15	14	26
Physically protected	1	8	0	4	1	12
Total	6	19	9	19	15	38

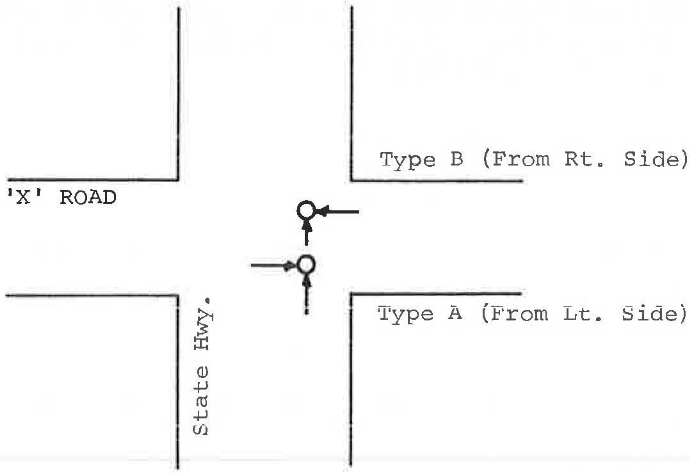


Figure 1.

It is interesting to note that both Type A broadside (Fig. 1) accidents (involving crossing street vehicles approaching from the left of the vehicle on the state highway) and Type B broadside accidents (involving crossing street vehicles approaching from the right of the vehicle on the state highway) were significantly increased (Type A, $P < 0.05$; Type B, $P < 0.10$).

It is possible that after the intersections were channelized, thus opening up through traffic to two lanes, that speeds on the state highway through the intersection increased. If this were the case, the cross road driver might not have adequate sight distance to judge the speed of vehicles on the state highway and thus broadside accidents could ensue.

Left-Turn Channelization Warrants

An investigation was made to determine the best criteria for establishing warrants for left-turn channelization of intersections. Various trial warrants were applied to the unsignalized intersection accident data (Table 10). These were compared to the results of all the intersections as constructed.

It is necessary to compare only accident frequency since the vehicular exposure is equal for all warrants. To compare the effects of different warrant criteria on all of the projects, it was necessary to estimate the accident experience in the after period for the unwarranted intersections. Since the before and after exposures were approximately the same, the number of before accidents was assumed to remain the same in the after period for projects not meeting a specific trial warrant. Therefore, the total accident experience in the after period was composed of the actual accidents of the warranted projects plus the before accident experience of the unwarranted intersections.

TABLE 10

EVALUATION OF WARRANTS—UNSIGNALIZED INTERSECTIONS

Warrant[a] Description	No. Warr. Projects	Estimated Reduction		
		L.T.	R.E.	Total
All Projects	40	20	140	151
3 LT+RE/Year[b]	22	24	119	141
4 LT+RE/Year[b]	13	9	96	98
5 LT+RE/Year[b]	10	6	93	93
4 LT+RE/Year or 6 LT+RE/2 Years	19	12	116	136

[a]Accidents involving channelized leg traffic.
[b]Average minimum number of accidents per year.

Note: The after accident experience for "Unwarranted" projects was estimated by assuming the number of before accidents would remain the same in the after period (before and after exposure approximately equal).

The accidents in Table 10 are total accidents. The projects declared unwarranted, however, were removed on the basis of the specific number and type of accidents of the warrant being considered. Since the before and after periods are equal for all projects, it is not necessary to compare accidents on a per year basis. In fact, by using the total periods, the changes are greater and more obvious.

When an average of three left-turn plus rear-end accidents was tried (Table 10), the total number of accidents reduced was only 10 less for these 22 projects than for all 40 projects. If the remaining 18 projects were constructed, a saving of less than 0.6 of an accident per project would be realized. This appears to be the best warrant evaluated. However, when a small separate study was made of 100 intersections with three accidents in the first year, only 40 of these had a two-year average of three accidents per year or greater; whereas, 66 of 100 intersections examined having four accidents in the first year had a two-year average of three or more accidents per year. For this reason, the last warrant which reduced 136 accidents while building 19 of the 40 projects is recommended.

The accident warrants as developed are intended as a guide to determine what corrective measure should be made relative to a specific accident pattern at a problem location. If a potential hazard is detected, improvements should be made without waiting for any specific number of accidents to occur.

The percents of accidents reduced were grouped relative to the zoned speeds on the state highway through the 19 warranted intersections. It was determined that at locations having zoned speeds of 55 mph or greater, painted left-turn channelization projects experienced slightly greater accident reductions than the physically protected intersections which were more effective at slower speeds (Table 11). These changes are all significant at $P < 0.001$ with the exception of painted channelization at the slower speeds which is significant at $P < 0.10$. It is felt that where physical protection is needed, the use of raised bars with a painted outline would in most instances do a good job in reducing accidents by economically providing an excellent delineator of the protective portion of the pocket in day or night. Also in night rainy weather, the bars provide reflective surfaces for headlights and make the channelization visible to drivers. One type of raised traffic bar used in California is only 1½ in. high and could be traversed in an emergency situation with little loss of vehicle control. These bars are an inexpensive and easily added-on device.

The number of projects are insufficient for warrant analysis at signalized intersections, especially since 9 of the 13 intersections were built as one painted channelization project and tend to bias the sample disproportionately.

Findings and Recommendations (Left-Turn Channelization)

1. Both painted and physically protected channelization were effective in reducing accidents (32 percent for paint and 64 percent for physically protected).

2. Painted left-turn channelization was more effective in reducing total accidents in rural areas than in urban areas. Paint and physical protection are equally effective in reducing total accidents in rural areas.

3. Significant total accident reductions were found in urban areas for physically protected intersections.

TABLE 11

MAIN LINE ZONED SPEEDS VERSUS TYPE OF CHANNELIZATION
(Warranted Projects Only)

Zoned Speed	Paint			Physically Protected		
	Acc.		Percent Change	Acc.		Percent Change
	B	A		B	A	
35-50	63	43	-32	46	11	-77
55-65	31	7	-77	87	31	-64

4. Painted intersections had a significant reduction in rear-end accidents under urban conditions. There were no indications that vehicles were driving over the paint and causing accidents.

5. Significant broadside accident increases were found at urban intersections at both painted and physically protected intersections.

6. The following warrants for left-turn channelization are recommended:

> The use of left-turn channelization as a traffic control device should be considered at unsignalized intersections having a total of 4 or more left-turn plus rear-end accidents in 12 months (involving vehicles from intersection legs to be channelized), or six or more left-turn plus rear-end accidents in a 24-month period. If the state highway is zoned for speeds 55 mph or greater, the use of painted channelization should be considered. If the zoned speeds are less than 55 mph, the use of physically protected form of channelization is suggested.
>
> Intersections with borderline warrants, where the improvement can be made at a reasonable cost, should also be considered on a cost-per-accident reduced basis.

7. It is recommended that for unsignalized intersections an average accident rate reduction of 60 percent or an average after base rate of 0.5 be used to estimate the numbers of future accidents. The preferred method is the use of the after base rate.

SIGNS

In California, signs are used only when justified by a factual review of field conditions. They are used where special regulations apply, where unusual conditions are not self-evident, and to furnish directional information.

Four general types of traffic signs are warning, regulatory, guide and construction. Specific warning and regulatory sign installations have been studied to determine their effect on safety.

Twenty-six of the 34 projects evaluated showed a reduction in accident rates with 8 of these projects having a significant accident reduction. There were no projects having significant accident increases.

The analysis of each type of sign installation is discussed separately. Before and after data have been reviewed at curve warning, curve warning plus advisory speed and at four-way stop signs. The total accident experience of these projects is given in Table 12.

Although there is an almost equal percent reduction in accidents when curve warning signs are used without advisory speed signs as with them, these reductions are not significant. This is an example of the unreliability of percent change of accidents as a criterion for comparing alternates.

Since the installation costs for these signs are generally minimal, no cost per accident reduced analysis was made.

TABLE 12

SIGN PROJECT SUMMARY

Sign	No. of Projects	Accidents		
		Before	After	Percent Change
Curve warning	4	22	18	-18
Advisory speed	24	209	162[s]	-22
4-Way stop	6	100	27[s]	-73
Total	34	331	207[s]	-37

[s]Significant at 0.10 level.

W3R
(W1-1)

W5R
(W1-2)

Figure 2. Curve warning signs.

Figure 3. Standard advisory speed sign (special sign in background).

Curve Warning Signs

The use of curve warning signs has been standardized consistent with the degree of hazard at any particular location by considering speed of approach and the reading on a standard ballbank indicator when traversing the curve (2).

The W3R sign is placed 250 to 750 ft in advance of a curve that has a 10-deg ballbank indicator reading at 30 mph or less. The W5R is used when the ballbank indicator reading is 10 deg at speeds of between 30 and 60 mph. These signs are shown in Figure 2.

Accident statistics at four rural two-lane highway curve locations were reviewed having either W3R or W5R ground-mounted signs. (One location had reversing 1000-ft radius curves with four W5R signs.) Only slight insignificant reductions in total accidents were obtained at curves having these signs (Table 13).

Susceptible to correction accidents involving vehicles exceeding the safe speed into the curve were also reviewed. These accidents were not significantly reduced when only curve warning signs were present.

It is felt that there were too few projects in the curve warning category for meaningful analysis although slight accident reductions were noted. Curve warning signs alone may not convey enough information to enable the driver to make an intelligent estimate of the safe speed of the curve.

Advisory Speed Signs

W46R (W13-1) Signs Ground Mounted in Standard Position—When additional warning is required at curves to reduce approach speeds for safety, an advisory speed sign (W46R) is installed on the same post below the curve warning sign (Fig. 3).

The speed shown on the sign is the multiple of five nearest to the lowest safe speed found for the condition. Each direction of travel is considered separately (2).

No attempt was made to evaluate the advisory speed placed on the individual curves. As a portion of another study, the safe speed of a random sample of 200 curves throughout California was determined. Over 90 percent of these curves were found to have safe speeds equal to or within 5 mph of the advisory speed.

The effects of advisory speed signs on accidents have been studied at 15 curves having ground-mounted signs in the standard position. The accidents given in Table 14 are total accidents (regardless of type) occurring at the curve locations.

Total accidents ($P < 0.10$) were reduced as mentioned before. These were mainly single-vehicle ($P < 0.02$), and property damage only accidents ($P < 0.10$). There were no

TABLE 13
GROUND-MOUNTED W3R AND W5R CURVE WARNING SIGNS

Time[a]	Sing. Veh.	Multi. Veh.	PDO	I+F	Day	Night	Total
Before	15	7	12	10	11	11	22
After	11	7	11	7	12	6	18

[a]Before and after exposure approximately equal.

TABLE 14

TOTAL ACCIDENT EXPERIENCE AT CURVES WITH CURVE WARNING AND ADVISORY SPEED SIGNS[a]

Time	MV	Sing. Veh.	Multi. Veh.	PDO	I+F	Day	Night	Total
Before	62.7	95	47	76	66	68	74	142
After	67.6	68	55	58	65	61	62	123

[a]Ground mounted in standard position.

TABLE 15

ADD W46R TO EXISTING CURVE WARNING SIGN

Time	Accidents	Exposure (MV)	Rate (Acc/MV)
Before	52	14.6	3.56
After	43[a]	18.9	2.28
Percent rate change			-36

[a]$P < 0.05$.

significant changes in multiple-vehicle accidents or in daytime or night accidents. The more severe (I+F) accidents also remained the same. Susceptible to correction accidents (exceeding the safe speed) were also examined. No appreciable difference from the total accident experience was found. These data were obtained from 13 locations where both curve warning and advisory speed signs were installed at the same time and two locations where advisory speed signs were added to existing curve warning signs.

To obtain a better evaluation of the advisory speed sign, data are given in Table 15 for 15 curves at which an advisory speed plate was added to an existing curve warning sign.

Where single-vehicle accidents were reviewed, the majority of accidents were of the run-off-the-road type, some hitting a fixed object. Fixed object and ran-off-the-road accidents occurring at night were significantly reduced.

Figure 4. Oversize special sign placed in head-on position.

Special Combination Signs

A large special sign having both a curve arrow and advisory speed has been used successfully in California. It is placed at the head-on position at curves where there is a severe ROR (ran-off-the-road) accident problem. It is generally placed where standard curve warning and advisory speed signs are already present. An example of this sign is shown in Figure 4.

A study of accident data at six special sign locations is given in Table 16.

Although available data are somewhat limited, total accidents (notably the single-vehicle type) were significantly reduced ($P < 0.01$). All other subgroups were significantly reduced with the exception of multiple-vehicle accidents. These signs apparently are doing a good job when additional emphasis of the safe speed beyond the standard W46R is needed by the motorist.

TABLE 16

ACCIDENT DATA FOR SPECIAL CURVE ARROW AND STATED SPEED SIGN

Time	MV	Sing. Veh.	Multi. Veh.	PDO	I+F	Day	Night	Total
Before	3.5	18	4	12	10	11	11	22
After	3.8	2	3	2	3	1	4	5

TABLE 17

TOTAL ACCIDENT EXPERIENCE ILLUMINATED OVERHEAD SIGNS

Time	MV	PDO	I+F	Day	Night	Total
Before	19.5	23	22	19	26	45
After	20.4	18	16	25	9	34

TABLE 18

FOUR-WAY STOP SIGNS (TOTAL ACCIDENTS)

Time	MV	Sing. Veh.	Multi. Veh.	PDO	I+F	Day	Night	Total
Before	50.8	4	96	53	47	66	34	100
After	59.3	0	27	18	9	11	16	27

Overhead Signs

Accident data from three projects having dual curve warning signs with an advisory speed sign mast arm mounted (Fig. 5) were examined. These signs were illuminated and placed on highways having four lanes.

Table 17 gives a breakdown of total accidents at these locations before and after the improvement. Although reductions were noted in total accidents, only night accidents were significantly changed ($P < 0.01$).

Four-Way Stop Signs

Four-way stop signs were installed at six intersections having stop signs facing minor road traffic in the before period. A four-way stop sign installation is a useful traffic control measure, and is most effective when the volume of traffic on the intersecting roads is about equal. The four-way stops were effective in reducing total accidents (Table 18).

Broadside accidents, a major problem at these intersections, were reduced from 78 to 12. The accident rate was reduced from 1.97 to 0.46 accidents per million vehicles, a 77 percent reduction.

Figure 5. Dual overhead curve warning and advisory speed signs.

Sign Warrants

Inasmuch as the cost of installing advisory speed and curve warning signs is

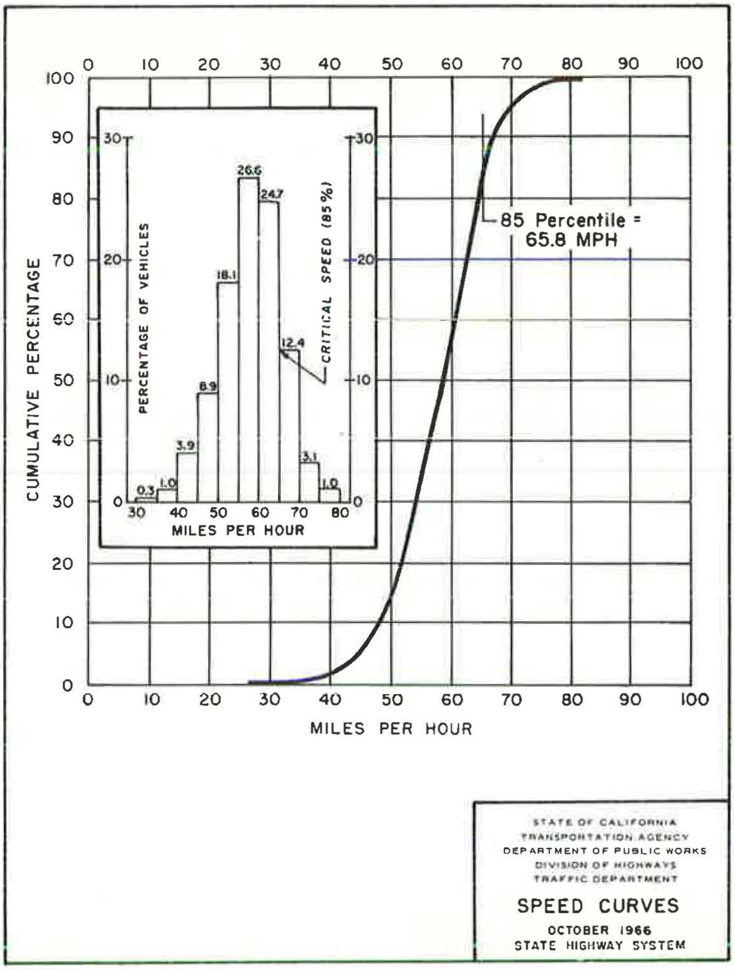


Figure 6.

very small (excepting the dual overhead signs), these signs could be quickly installed at a problem curve having a run-off-the-road accident problem.

Figure 6 is a speed curve for passenger cars from a statewide speed check in 1966. Some segments of traffic are exceeding the 85 percentile (critical) speed by approximately 17 mph. A portion of these fast vehicles would have difficulty negotiating a curve designed for the critical speed. If the safe speed on the curve is 5 mph slower than the critical speed on the curve approach, the upper segment of drivers are at least 20 mph over the safe speed.

Since the addition of an advisory speed sign to the standard curve warning sign resulted in greater accident reductions, consideration should be given to placing a W46R sign at locations with a 5-mph speed differential that requires a curve warning sign.

All of these six 4-way stop sign installations studied met the present warrants for the State of California. Because of the dearth of information, no warrant analyses for 4-way stop signs were made although it appears that present warrants are satisfactory.

Findings and Recommendations (Signs)

1. Curve Warning Signs (ground mounted in standard position)—Although the number of projects reviewed were few (four), present data indicate that no significant accident reductions occurred after placing either W3R or W5R (Curve Warning) signs.

2. Advisory Speed Signs (ground mounted in standard position)—When an advisory speed sign (W46R) was added to the curve warning sign, significant accident reductions were found. Specifically, ran-off-road accidents occurring at night were reduced at these ground-mounted signs. It is recommended that a W46R sign be placed at all locations requiring a curve warning sign. The placement of these signs should be considered when the safe speed is 5 mph less than the critical (85th percentile) speed of the approach.

3. Special Oversize—Combination curve warning and advisory speed signs placed in the head-on position at curves with a serious accident problem were quite effective in reducing single-vehicle accidents both in daytime and at night. When the standard curve warning and advisory speed signs are not totally effective and when approach speeds exceed 50 mph it is recommended that this special sign be used.

4. Illuminated dual overhead curve warning arrows with an advisory speed sign were effective in reducing night accidents on four-lane expressway curves.

5. Four-way stop signs were effective in reducing multiple-vehicle, namely right-angle or broadside, accidents. It appears that present warrants are satisfactory.

ACKNOWLEDGMENT

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Computer Displays for the Traffic Engineer

RICHARD C. SANDYS and JOHN L. SCHLAEFLI, Stanford Research Institute

This paper describes and demonstrates the application of visual display techniques generated by a digital computer to the problem of evaluating alternative traffic-engineering improvements in congested urban networks. Stanford Research Institute has developed a tool to be used by the traffic engineer in such evaluations. It is a computer simulation called the Dynamic Highway Transportation Model (DHTM). Problems arise when a user tries to analyze the large amounts of multidimensional simulation output produced by DHTM. To make the DHTM method practical for the traffic engineer, cathode-ray tube display equipment, in conjunction with a CDC 3100 computer located at SRI, and a computer programming language designed for use by a traffic engineer are applied to the evaluation problem. Maps with overlays of volumes, speeds along streets, delays at intersections and routes between selected origin-destination pairs can be called for from the display console and shown instantaneously. Plots of demand and delay for all or selected elements of the traffic networks can be called. A number of measures of the quality of traffic service can be specified and evaluated easily while the traffic engineer is sitting at the console.

Technical feasibility of the display concepts is demonstrated. The cost of operating the CDC 3100 display system located at SRI is about $80 per hour. With the technical and economic feasibility in hand, further demonstrations with practicing traffic engineers using the tools of the evaluation methodology should be forthcoming.

•DHTM (Dynamic Highway Transportation Model) is a computer model that dynamically simulates traffic flow in a network. The inputs to this simulation and the resulting outputs involve large amounts of data. The quantity of these data makes it difficult for the traffic engineer to evaluate the outputs and to reduce them to a usable form. For example, a San Francisco test network is divided into 100 origin or destination zones. In general, each origin has demand to about 60 destinations. DHTM generates up to 10 routes between each O-D pair where demand exists and thus an array that contains about 60,000 routes, each involving 4 to 7 links, is used in the simulation. The output of these routes in numerical form (or conventional computer output) would require over 1000 pages of printing. Evaluating these routes for logic and completeness is not practical when they are left in numerical form. The resulting communication problem is alleviated through the use of computer display techniques.

DEMONSTRATION OF COMPUTER DISPLAYS

The displays presented here are the results of an internally sponsored Stanford Research Institute (SRI) project (1), using data gathered as part of a U.S. Department of

Paper sponsored by Committee on Quality of Traffic Service and presented at the 48th Annual Meeting.

Transportation research project (2, 3). The displays were generated from output of SRI's DHTM (4), and from field studies of traffic parameters representing the morning rush hour in a large test network in San Francisco.

Map Display

Figure 1 shows a computer-generated display of a map of approximately one-fifth of San Francisco (2, 5). About 400 links and 100 signalized intersections of the test network used in DHTM are included. Each line represents one arterial street, that, in general, consists of two links. A link is defined as a single direction of a street in the network connecting two intersections. Links and intersections are assigned reference numbers that are not reproduced on the display. A total of 100 O-D zones has been defined relative to the test area.

The map is a common tool for the traffic engineer. This two-dimensional representation of a network provides a frame of reference for evaluating various traffic parameters such as demand and travel times, and gives relative distances, physical barriers, and orientation at a glance. To plot such information from conventional hard-copy computer output would be a time-consuming and costly process. Since the map display is based on computer input numbers, it also provides a cross-check for the inputs used. For example, if a street is input with an incorrect length, a localized section of the map will become skewed, thus indicating the error. The error can be corrected immediately by using the bug to relocate the intersection.

Peak-Hour Volume

Maps of peak-hour volume have (Fig. 2) been used extensively in traffic engineering and transportation planning evaluations (6, 7). The DHTM generates flows, travel times, delays, routes, and other data on traffic conditions throughout the network. Flows and travel times are broken down for short time intervals (of the order of 15 minutes) to

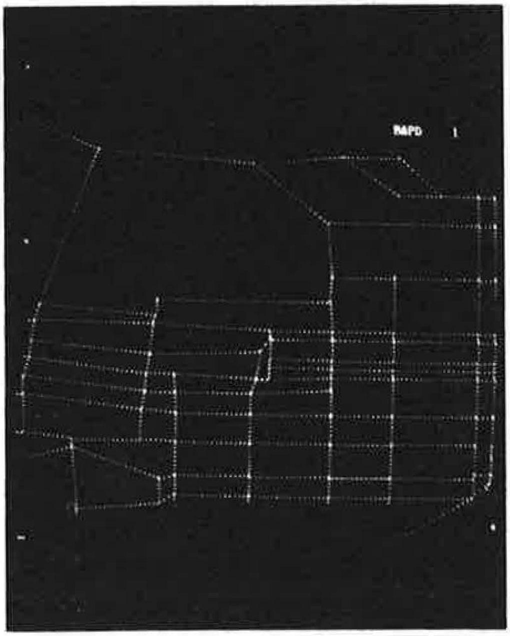

Figure 1. Computer-generated display of arterial street network in western section of San Francisco.

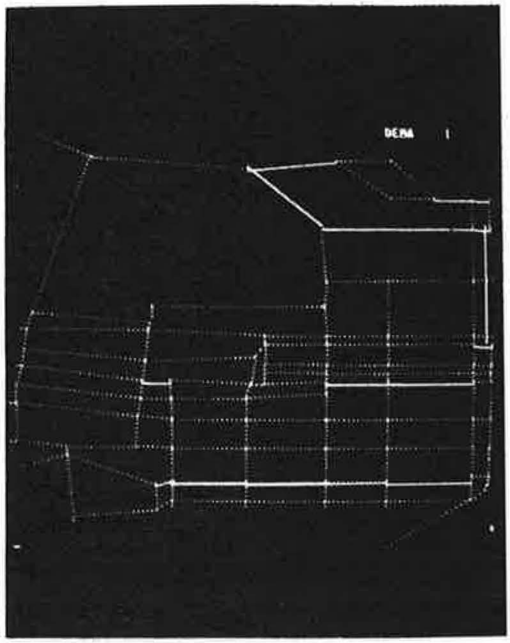

Figure 2. Heavy lines show streets with average volumes greater than 3000 vehicles in two hours (i.e., 0700 to 0900).

treat peaking in the demand patterns. Information for each time period is available for each iteration of the simulation. Peak 2-hr or 1-hr summaries or even a 15-min performance of a network can be shown on the display. A 2-hr representation of volume was chosen for Figure 2, with a threshold concept of presentation used. Links with volumes above 3000 cars in two hours are shown by a heavy line. Similarly, links with more than 6000 cars in two hours are accented by two heavy lines. Using this display the traffic analyst gets a feel for which links are major carriers, to what degree, and where they approach capacity. The value chosen as a threshold, the 2-hr time period, or simple functions of the volume (e.g., straight-, right-, or left-turners) can be designated from the console.

Peak Hour Delay

Travel time and/or delays along streets and at intersections are important traffic operational parameters. With the map as background the user can analyze these parameters from the display console. Figure 3 shows time parameters in the San Francisco network. Average delays of 30 sec per vehicle or more over a 2-hr period are represented by solid lines. The average shown is a summary of the straight, right-, and left-turn delays associated with each intersection. This display is useful in identifying trouble. It should be noted that trouble spots can be caused by an input data error or a simulation error, depending on the source of the data being used to generate the display. On the other hand, an actual physical problem may exist at an intersection. Thus, the total system map can be used to indicate the overall quality of traffic service as a function of travel time and/or delay.

Field Data Comparison

The previous displays are generated by using simulated volume counts and delays. A comparison with field data shows the user how well the simulation is performing. Figure 4 shows a magnified map of the central section of the test network. Offset

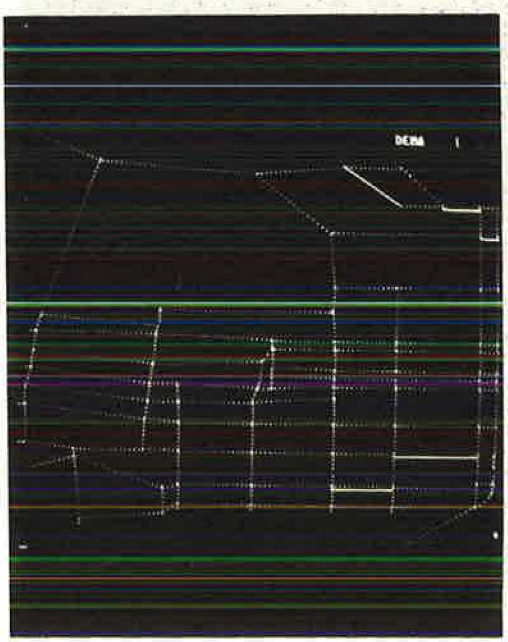

Figure 3. Heavy lines show streets with average intersection delay per vehicle of 30 sec or more.

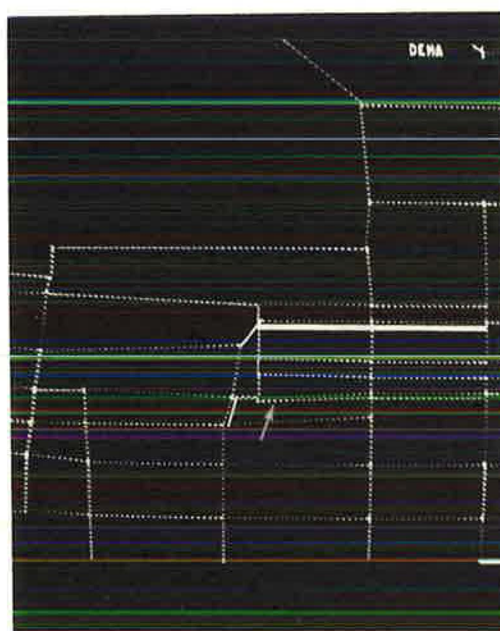

Figure 4. Magnified map display showing differences between simulated and field-measured volume data; offset lines show differences greater than 500 veh in two hours.

dotted or solid lines are used to indicate differences between simulated and field measured data; a solid line represents modeled volumes that were 500 veh less than the field volumes during a 2-hr period; an offset dotted line (arrow) represents modeled volumes that were 500 veh greater than the field volumes during a 2-hr period. A shift of simulated vehicles between two parallel links in the center would make the modeled data fit the real world more closely. Obviously this display is valuable in determining how to improve the simulation.

Magnification

Figures 4 and 5 both demonstrate the ability of the display to magnify a map. In the case where the user wishes to display the actual numerical differences between modeled and field volumes over two hours, the desired section of the map is magnified by a factor of three. The resulting display is shown in Figure 5, where it seems that for some reason the simulation has underestimated the number of vehicles on the northbound links.

Magnified Map Showing Delays

After looking at Figure 5, the user may well ask why the differences in volumes exist. Figure 6 demonstrates the ability to display average vehicle delays at each link intersection in a magnified area. Each delay is given in seconds per vehicle. In this case the delays shown on the display seem reasonable based on traffic engineering experience with the intersections being considered. Thus the analyst may conclude that the fault lies in some step in the simulation process that took place before generating delays from volumes.

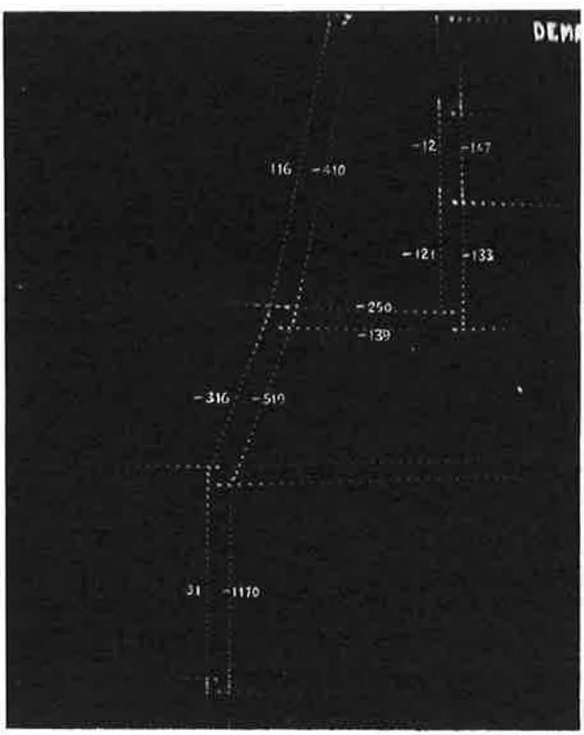

Figure 5. Magnified map display showing numerical values of the difference between simulated and measured volumes.

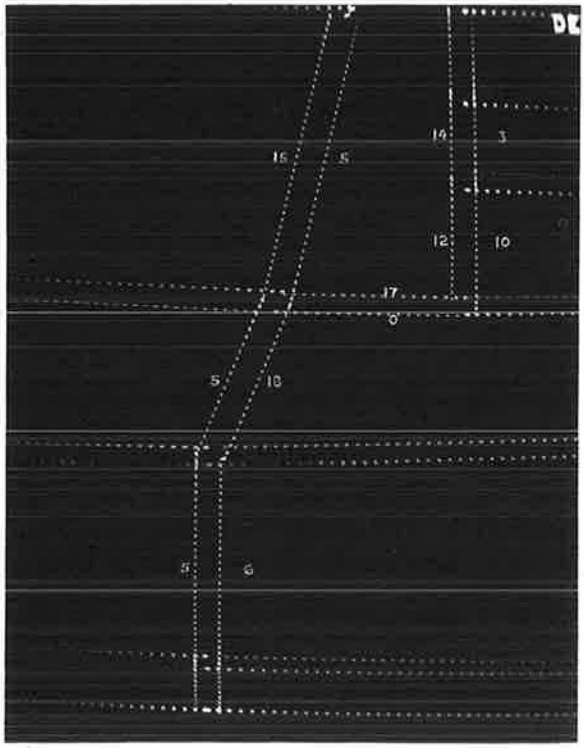

Figure 6. Magnified map display giving values of average delay (seconds per vehicle) for all links.

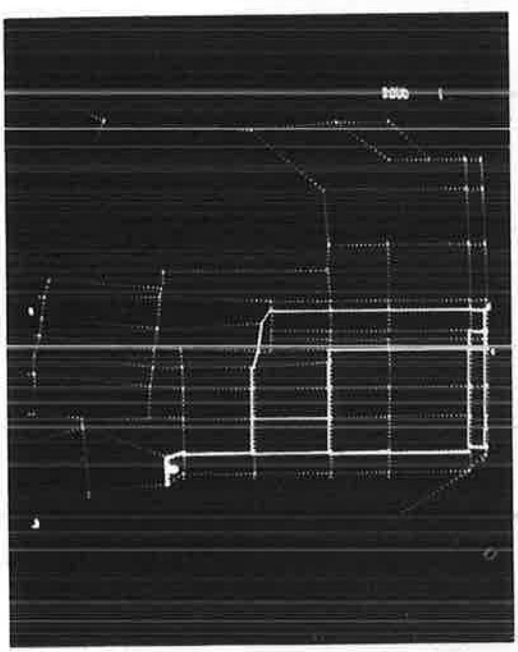

Figure 7. Alternate routes from origin zone 100 to desitnation zone 9, based on shortest distance, minimum delay, and minimum travel time.

Routes

Figure 7 demonstrates the ability to display alternate routes from origin zone 100 to destination zone 9 in downtown San Francisco (i.e., the financial district lying to the right of the picture). A traffic engineer familiar with the area would note that an important alternate has been omitted. This route is one that has a right turn at the intersection approximately at the center of Figures 5 and 6. In a later analysis of the problem, a hand calculation of the route selection algorithm shows that this route was about as good as any of the others. Thus a provision was made to include this route as one of the alternates for this O-D pair. On rerunning the simulation with the new route, a display like Figure 5 indicated more favorable simulation results.

Link and Intersection Demand And Delay

In addition to general information on the trouble spots in the network or on possible data inconsistencies, further demand-delay information may be desirable. Figure 8 demonstrates how a system of graphs can be displayed to allow the traffic engineer to compare the computer description of a particular intersection or link with his intuitive feel or knowledge of how it should function. The graph shows average delay in seconds per vehicle versus demand or volume for all traffic on link 181 at its associated output intersection. The principal demand at this intersection is for a left turn. Each demand level given for link 181 was produced as the number of vehicles being routed through the link changed as a result of more routes being generated between all O-D pairs in the network. Changes in demand level indicated by the numbers 3 through 9 on the display were produced after routes 3 through 9 had been generated. Using the physical description of this link and intersection, the traffic engineer can determine if the delay generated by the simulation is of reasonable magnitude. The plot shown is for average delay over a 2-hr period. The delay values from the plot can be output on the line printer. The straight-line portion of the plot represents a delay of about 30 sec per vehicle. The maximum delay is about 195 sec per vehicle, and the knee of the curve occurs at about 1450 veh in two hours with over 95 percent left-turners. Of course intersection performance in general is affected dynamically by all 12 flows (i.e., four directions with three turning movements each) at the intersection. Any one of those flows can change as new routes are generated. Similar plots for travel time and delay for each turning movement through the intersection can be overlayed. Obviously, many insights can be gained from these plots, such as the extent to which capacity should be increased, demand decreased, or signal parameters changed to achieve better service. One of the more important capabilities is that of determining critical demand for a particular intersection. At this point other displays, describing the cross street and opposing traffic, should be called to evaluate a possible physical change at the intersection.

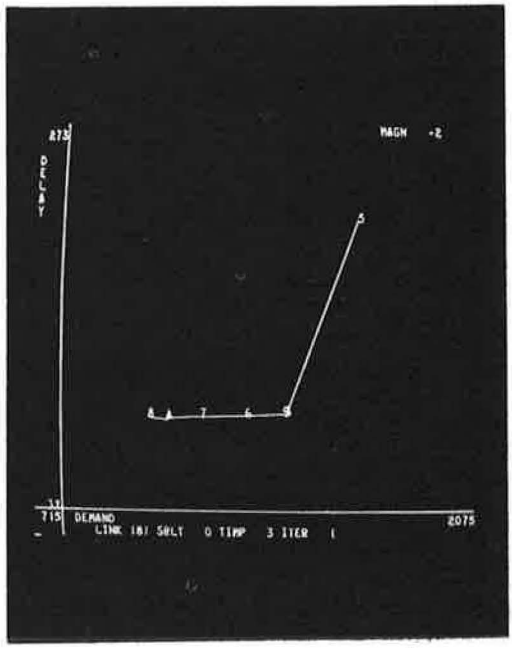

Figure 8. Computer-generated plot of delay (seconds per vehicle) versus demand (vehicles in two hours); the number of alternative routes available between each O-D pair is given for each point.

Network Demand and Delay

Figure 9 demonstrates a method for evaluating total network performance from the display console; it shows a plot of delay versus demand averaged over all intersections and links of the network. Each new point on the plot represents the demand and delay associated with increasing the number of alternative routes in the network. The results indicate that with 8 or 9 routes the simulation predicts a delay of about 19 sec per vehicle and a demand of about 800 veh on an average link of the network. The traffic engineer can use this display to evaluate measures of the quality of traffic service that include total travel time, average delay, average speed, and total vehicle-miles per hour. Sitting at the console he can define and evaluate any one of the measures in all or any part of the network. This allows him to look at only those intersections and links with demand near capacity or satisfying some other criterion or point of view taken in the evaluation.

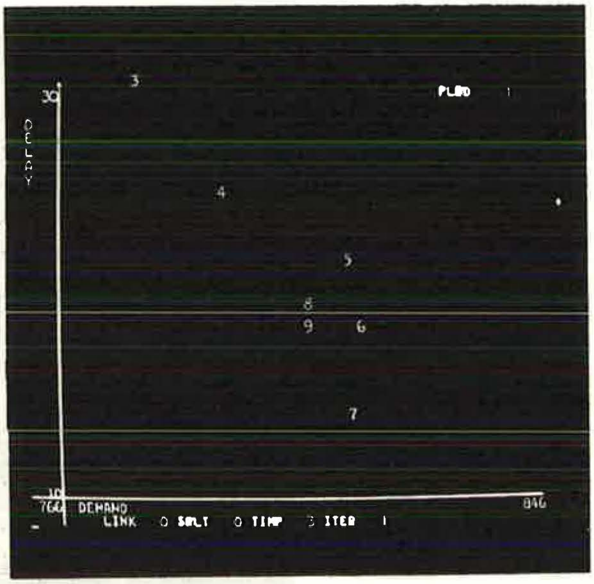

Figure 9. Plot of average delay per vehicle versus demand per link for all links of the network; the number of alternative routes available between each O-D pair is displayed as a label for each point.

THE DISPLAY SYSTEM

Physical Characteristics of the Display

The requirements for computer hardware are an important consideration to the potential user of a display system. The configuration at SRI consists of a CDC 3100 computer with three magnetic tape units, a line printer, a card reader, a 700,000 word disk, and a display system including a 512- by 512-point cathode-ray tube, a keyboard, and a cursor. The present system costs about $80 an hour to operate. Figure 10 shows the work station on the CDC 3100 system and Figure 11 shows a schematic of the computer hardware and its function. For the San Francisco example, disk memory of approximately 400,000 words was used to store 9 routes between each O-D pair, volume counts and delays on 400 links during 8 time intervals, and 10 iterations of the simulation. Any word of data can be accessed by the system in less than one-half second. This capability coupled with efficient retention of frequently referenced data in the computer core, provides the operator access to any form of the data with no discernible delay while a display is generated.

Operating the system is a simple task. Pressing three buttons on the computer console initiates the program. The traffic engineer may then sit at the keyboard before the display and make requests or write a program. Anything he types in from the keyboard appears immediately on the display. He may backspace and retype characters, delete a line, or request that the line be executed.

A "bug" is used as a pointer on the screen; it can be moved about the screen by using the "mouse" on the console table. The operator can press a button on the mouse to indicate that he is pointing to something he wishes to identify. For example, while a program is displayed on the right side of the screen, the operator may wish to change one of the instructions. To do this, he points at the instruction. The identified instruction will light up and he may type in the new instruction. The flexibility this affords is obvious: the operator can correct his mistakes and add or delete instructions—making the programming process quite simple.

Similarly, the bug can be used to identify an area of any display for magnification, to reposition links or intersections on the map, or to adjust other parameters. Data from the disk in numerical form can be displayed directly. During analysis of the

Figure 10. CDC 3100 display system work station.

routes, the link numbers making up a route can be printed. The traffic engineer can generate a new route by typing in STRAIGHT, RIGHT, or LEFT at intersections to select successive links of the new route. When he wishes to save a particular display, he can store it on magnetic tape for future plotting by a Cal Comp plotter. The operator can also interrupt a program he has written when it is no longer useful or when he desires more detail about an event that has been observed on the display.

The principal difficulty that a traffic engineer will have is remembering what instructions are available and how each should be used. To alleviate this problem, a

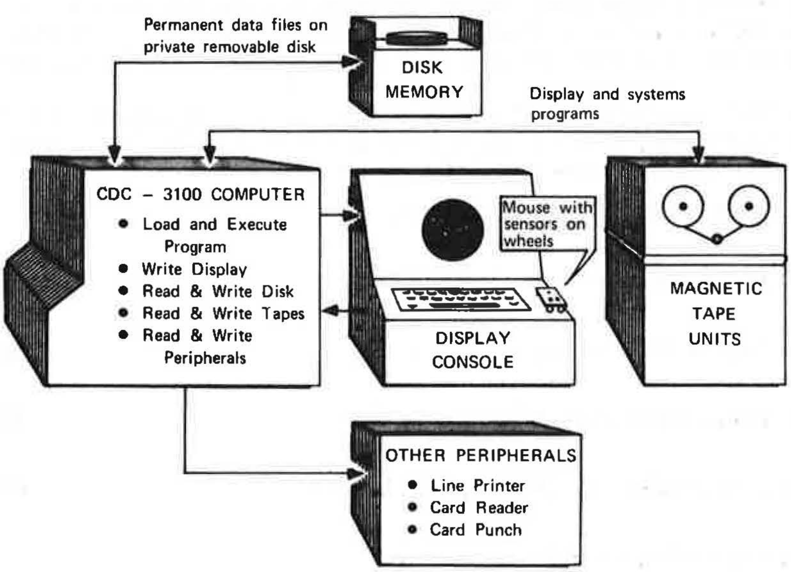

Figure 11. Schematic of CDC 3100 computer and display console.

description of each instruction is stored on the disk and may be displayed any time the operator needs to refresh his memory.

A Traffic Engineer's Display Language

A system of computer logic has been developed to provide the user with his own computer programming language for display of measured traffic networks. Display of a Measured Traffic Network (DMTN) is a computer program that implements the language. Simply defined, a language is a system for representing and communicating information or data between man and machine. As the user types at the keyboard in front of the display screen, various characters are shown. These characters can form names, numbers, or pictures. Names are strings of characters that have special meaning to the computer. For instance, in the language of DMTN, MAPD 1 typed in and executed will cause a map to be displayed. It will be displayed in the area of the screen defined as display number 1; MAPD is the name of a subroutine built into the program to perform this function.

In general the display language consists of strings of characters that have some predefined meaning (8). Remembering these meanings presents some difficulty for the user, but this difficulty is offset by his ability to write unique sequences of instructions and thus to broaden the use of the display. For convenience and ease of operation, the result of each instruction that is typed in and executed is shown on the display. Furthermore, the instruction can be added to a saved program, and a sequence of saved instructions can be executed repeatedly without the user having to retype them. Through this feature the user can look at a list of words that belongs to the language, type one in as an instruction, and see what happens. If he does not like the results, he can cancel that instruction and try another or a different sequence of instructions.

During execution of a program, branching to some alternative series of instructions may be desired. Before proceeding along a new branch, the language interpreter restores variables to their original condition. This is a definite contrast to the usual computer-programming language that consists of a series of instructions executed one after another by the computer, after which variables are left as they were at the end of each branch. The purpose of developing the display language in this unique way is to simplify programming. While programming at the keyboard of the display, the user has to remember only the higher ordered instructions above the branch being programmed. The program as it is developed at the keyboard is stored as a tree with branches and nodes. When a branch has been made, it is impossible to execute another branch of the tree at the same level without returning to a starting branch. The inability to branch to any node of the instruction tree has not proved to be detractive to the language since the user can consider each path through the tree as a logical entity in itself.

Perhaps the above description of the basic features of the language is a bit difficult to follow. For further clarification, an example has been worked out. A program to display nine routes between a specified O-D pair follows:

The first two lines represent a branch label:	III- 2
The next two set the line type for the map to dotted:	LINE 1
The next two display the link map on the display defined as 1:	MAPD 1
The next two read a value into ORIG as the origin zone:	ORIG 95
The next two read a value into DEST as the destination:	DEST 5
The next two are a branch label:	1 - 21

The next two initialize the iteration number as zero:	ITER 0
The next two call for nine repetitions of every instruction below that is part of the current branch:	DO 9
The next two increment the iteration number by one:	ITER + 1
The next two change the label so that simultaneous displays can be shown on display number 1:	LABE + 1
The next two display the route between the origin and destination zone specified on display number 1:	ROUD 1
The next two set delays of 15 seconds for keeping the display on the screen:	TIME 15
The next two lines represent a new branch with a higher number than 1 and so terminate the DO:	2 - 0
The last indicates the end of the program:	END-

Figure 12 is an actual display of the program, and the display resulting from the program is very similar to that shown in Figure 7. Various routes from origin 95 to destination zone 5 are displayed in the order they were chosen (i. e., on each iteration of DHTM). A new route is shown every 15 sec unless the operator interrupts the program.

In summary, a language for communicating with a computer on the problem of traffic on urban street networks has been developed and illustrated. The language has a unique feature of tree storage of instructions with a self-restoring feature that makes use of the display language from the console quite easy. The real utility of the display language can be proved only when analysts and engineers can use the display in a network evaluation. To date, the language and associated equipment have demonstrated many of the qualities necessary to make computer display a practical tool for these potential users.

TOWARD A PRACTICAL METHODOLOGY

The objective of developing the computer display techniques is to demonstrate the technical and economic feasibility of using these techniques as part of a practical method of measuring and evaluating traffic engineering improvements in urban street networks. The methodology requires two specific tools: a dynamic simulation model of an urban traffic network and a set of computer display techniques to be used in conjunction with the simulation. Both of these tools go beyond the present state of the art in traffic engineering.

The DHTM has been developed as a tool for comparing alternative highway improvements in terms of user benefit and cost. The model has been refined and is in the process of being calibrated by using input data gathered in a large test network in San Francisco (5). The model

Figure 12. Program to display nine routes between origin zone 95 and destination zone 5.

is unique in that it treats the dynamics of demand and delay in a congested street network at rush hour and has sufficient component detail to allow the evaluation of many of the important parameters of interest in an urban traffic network. These parameters include signal timing, signal progression, turn channels, signal phasing, and parking restrictions.

In the model, origin-destination-time (O-D-T) inputs are treated for short periods (on the order of 15 min) to account for peaking at various points within the network. A number of alternative routes between O-D pairs are developed so that the decisions available to the population of drivers making the route choice can be represented. Traffic is assigned to these routes on the basis of time, distance, and comfort; then the resulting travel times, and other quality of traffic service measures throughout the network are calculated. During the process, a submodel calculates dynamic delays at each intersection and along each street of the network. This submodel is DHTM's most rigid tie to the real world and contains much of the detail required to make the overall model responsive to the decision variables of interest.

The problem of making the results of a computer simulation model, such as DHTM, usable to the traffic engineer has been addressed here. Computer displays (DHTN) and the resulting man/computer relationship have been demonstrated to allow the traffic engineer to interpret visually the effects of changes in his network. Using this tool in conjunction with DHTM, the traffic engineer can sit at a display console and call for basic data and/or descriptive (i.e., a representation of the existing situation) simulation results from his network for use as a basis in designing and evaluating improvements. When he postulates practical network modifications, he can use the predictive simulation results to evaluate rapidly the effect of changes on network performance.

Research on a traffic-engineering evaluation methodology offers new tools to the urban traffic engineer. As envisioned, the application of the SRI methodology in an urban area will consist of the following steps:

1. Obtain a complete description of the network, including existing demand patterns.

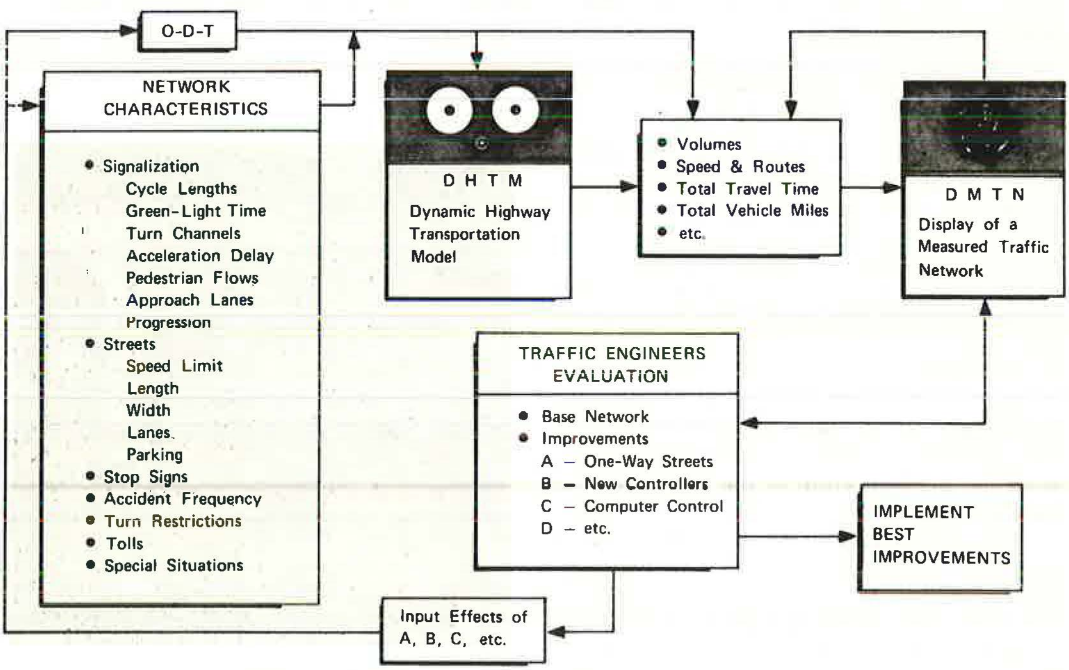

Figure 13. Methodology for traffic systems evaluation.

2. Simulate (DHTM) the existing traffic flow over the network and isolate the major problem areas by means of computer-generated displays (DMTN).

3. Develop sets of network improvements, such as one-way streets, signal timing, and parking restrictions, by using judgment and DHTM and DMTN.

4. Use DHTM to estimate the effects, appropriately measured, of each proposed set of improvements.

5. Choose a good set of changes based on the predicted improvements in traffic flow, the associated costs, and other important constraints.

6. Implement this set of changes and then measure the new volumes, delays, and demands in the network.

Steps 3 and 4 will probably have to be repeated several times to insure the consideration of all improvements of highest potential. For further clarification, Figure 13 presents a diagram of the overall process. Examples of the types of input information required and the interaction of the man and machine during the evaluation process are illustrated.

It should be emphasized that this procedure is carried out not just once but is a continuing program of improvement to the traffic network, subject to such factors as political, fiscal, and long-range planning constraints. The method has the advantage of enabling the analyst to consider a wide range of alternative solutions and to choose the best one without repeating extensive field work. The technical and economic feasibility of these tools has been demonstrated. Refinement and actual application are presently under way.

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Discussion

GORDON A. SHUNK, Alan M. Voorhees & Associates, Inc.—The computer display technique presented by the authors is yet another example of the beneficial spinoff from modern space age technology to improve the capabilities of more conventional practitioners. The concept of such displays is proving to be exceedingly valuable in many applications. The implications for the practicing traffic engineer are every bit as profound. Here is an opportunity to conduct cause and effect analysis of traffic engineering improvements without having to construct and disrupt. Here is an opportunity to examine and perturb networks and their loadings without the waiting and plotting formerly required. The effectiveness of the traffic engineer and planner can be greatly enhanced by such a technique.

The greatest asset of such a technique is the capability of immediate display. An analyst can examine and evaluate the basic system visually and decide upon measures to improve problem areas he sees. He then can make changes to effect these improvements and immediately observe the true results of his proposal. In traffic engineering, of course, such a procedure obviates costly and time-consuming construction or other trial and error solutions. The situations for which this approach is most useful are those for which standard traffic engineering analyses are not applicable or effective.

The effectiveness of the graphic display technique is a function of the simulation model or other source which supplies input to the process. The model generating input to the display must be capable of rapid response to perturbation of initial conditions. Unless such a response is possible for use with the graphic display, its greatest contribution is negated and one might as well examine printed plots output from the simulation. The model should also be capable of producing for simultaneous display several service parameters necessary for adequate system evaluation. Any display should contain such service measures as speed and delay, in the mean or on selective links, and in some cases volumes. Dynamic simulators can provide these service parameters for specific time periods. The authors' technique displays such parameters using overlays.

The previous comments regarding the requirements or desirable attributes of graphic display capabilities have been somewhat general. Specific comments were avoided because they would have borne more directly on the capabilities of the simulation model, which was not the primary element of this paper.

The authors' comments regarding cost did not deal with the economics of setting up an operating display system for a traffic engineering department. Eighty dollars per hour is certainly a reasonable figure as long as the price need be paid only when the system is in use. However, a computer facility to generate inputs and feed the display unit is not likely to be readily available to many traffic engineers outside of large cities. And even in large cities, the out-of-shop residence of the unit is sure to detract from its greatest utility. Certainly the system presented could be converted for use on other types of computers.

One solution to the problem of access to a computer facility could be using remote terminals. The traffic engineer could rent a CRT and control unit which would be connected by telephone line to a central computer, perhaps in the state highway department. The simulation display programs would be stored for direct access at the central computer facility. The traffic engineer would then merely have to call his programs and control the display from his office, using the equipment only when necessary. The non-processing cost would be nominal and the ready access would encourage more extensive use.

A centralized processor with remote display and control capability could handle input from traffic engineering agencies in many locations throughout an area.

The language provided in the subject technique to control the display program may prove difficult to use. The abbreviated commands can be referenced in tables to aid operators. But if the commands could be at least full English words, operation would be facilitated. Such a change should not prove too difficult although a translater might have to be included in the program. Display of the commands for checking is an excellent feature. The error checking of displays permitted by use of the "bug" pointer also is a valuable attribute.

In summary, the technique proposed by the authors represents a valuable contribution to the practice of traffic engineering. Using a good simulation model, graphic displays can prove a time saving tool that readily returns their cost. They can improve the effectiveness of the practicing traffic engineer by concentrating his efforts on analysis and evaluation. They can improve city traffic flow by leading to implementation of truly effective improvements. They can help to apply funds in the place and manner which will best utilize resources available.

WILLIAM C. TAYLOR, Wayne State University—The authors have described some traffic engineering functions which can be conducted utilizing computer displays as an aid. The report clearly demonstrates that the technology is available, and at a reasonable cost, to consider the benefits which one might derive from these techniques. I think the authors should be commended for the thorough presentation of some possibilities for increased use of computer displays.

I would take exception to the authors' statement that it is more difficult to evaluate the quantitative data from a standard computer output than it is to do so from a visual display. In the final evaluation, an analysis of the quantitative values is required. I do not necessarily agree that searching photographs of a visual display which is heavily laden with numbers is less difficult than the analysis of standard computer output. The potential for establishing threshold values exists in tabular data presentation as well as visual data presentation.

I do not feel that we have arrived at a standard definition of the quality of traffic flow or service, nor do I envision that we will in the near future. This implies that there are more than one, and in fact, several objectives to be considered in the selection of alternative network configurations or control strategies. The information presented in the paper illustrates the potential use of the technique for the analysis of "tradeoffs" between the satisfaction of a set of objectives.

The selection of the final set of quantitative data to be presented can be made with the use of the visual display. The iterative procedure necessary to reduce the myriad of possible combinations of system configuration, control strategies and measures of performance to a reasonable number of combinations for consideration, can be done quite readily without the necessity of quantifying all the possibilities.

The same capability exists on an operational level as exists on the broad planning level. We are all aware of the time and resource demands of a quantitative sensitivity or parameter analysis. The example of the program languages flexibility in isolating individual elements of the network for study illustrates a possible use. If we consider an individual intersection with a range of possible design features, another range of possible control strategies and are interested in the operation of the intersection as measured by a different criterion for a specified volume range, the parameter analysis requirement is already four dimensional.

The simulation procedure of fixing three of these variables and allowing the fourth to assume all the values in its range, then repeating the process for each of the other variables, and then iterating this procedure to determine the optimal set of values is quite involved. The alternative, which is constructing a model that accurately describes the relationships among all of the variables, is not always possible. The use of a visual display technique combined with the command language developed in this study offers a third, and in many cases perhaps far more attractive, alternative. The procedure would be to narrow the range of the variables qualitatively instead of quantitatively, thus reducing the matrix of possible decisions to a manageable limit. This could then be followed by a quantitative analysis of the remaining variable values.

The third possibility which occurs to me for the use of the visual display techniques is possibly the most important. The major disadvantage of tabular data is its static nature. The tables depict the status of the system at a given instant, and cannot indicate what precedes or follows that instant. Unfortunately, there is no such thing as steady-state flow in a street network. The demand flow is known to increase to some peak value in the rush periods, but the time, location and magnitude of this peak are dynamic. If we take a time slice from the system, we can portray the peak at only one location and magnitude. Likewise, a locational slice can depict only one time period and one magnitude of demand.

The simulation of flows through the network could be accomplished with the DHTM as described by the authors, with the value for some selected performance measure shown on the display. Let us assume that this performance measure is the level of service as defined in the Highway Capacity Manual. The threshold concept could be used to readily identify any link which reaches level-of-service D. This can be accomplished by tabular methods also, but there are two advantages of a visual display: (a) the operator could stop the simulation and call for level-of-service characteristics of alternative paths, and (b) the system could be continued and the duration of the low service level could be determined.

If the results must be on tabular displays, the ability to recreate the instant when level-of-service D was reached on a link would require the initializing of the complete simulation up to that point. This is the property of a dynamic system, and cannot be avoided. This is often impractical from a cost point of view. With the visual display, immediate knowledge of alternatives available allows decisions to be made without starting the simulation run again.

The duration of the low level-of-service may be just as important in the decision-making process as the fact that the level exists at all. Once again, tradeoffs can be considered at the decision-making level. In this case the decisions are between various values of the same measure of performance, where before the decision was between alternative measures.

I feel the authors have developed a tool which will be very helpful to the profession. I consider its greatest benefit to be in the role of an aid to the decision maker rather than to the analyst. The concept should be continued, as I am sure will happen at Stanford Research Institute.

RICHARD C. SANDYS and JOHN L. SCHLAEFLI, Closure—The authors would like to thank Mr. Shunk and Dr. Taylor for their favorable discussions of this paper. Through these discussions a number of important points of interpretation have become evident. The following paragraphs attempt to clarify these points.

The display language (DMTN) enables the user to communicate and program directly from a display console. To date, the interaction that takes place is with data collected in the field or with simulation (DHTM) data produced on a larger computer. In order to attain immediate interaction with simulation data, a subset of the DHTM is being considered. This simulation would run on a small computer such as the one used for the DMTN program. If this is accomplished, immediate interaction with simulated data can take place and the traffic engineer will be able to see the results of his proposal in real time.

Experience indicates that the display console can be used to manipulate data and present the most interesting results. This capability is extremely important and has been accomplished using data generated by a complete cycle of the DHTM simulation. The ability of the display language to allow the traffic engineer to sift through enormous amounts of data without pre-specifying the order or the format of the search is new and powerful. For example, a certain intersection may operate below capacity and with no large queues for all but one 15-min time period during rush hour. If a hard copy approach is used, intersection performance for all time periods must be printed out. Using the display, the particular time period of interest can be isolated and then the printout can be limited to the relevant information. This capability is very powerful in itself; when immediate interaction with simulation and/or field data is achieved, the displays become even more powerful.

The feasibility of using remote (time-share) terminals for display has not been assessed directly. However, it seems that most existing operating systems on medium- to large-scale computers are rather static as far as data retrieval and allocation of computer core storage are concerned (1). A user cannot demand the whole computer when he wishes, but must always share with other users. Also a user must pay for the maximum amount of computer storage used during the running of a program. Thus, it is very expensive to pause in the execution of the simulation to display intermediate results. What is required is an operating system that will permit dynamic core allocation. Then the simulation/display user could store the simulation program and results on a disk, while he retains the display program in core and looks at intermediate results.

Research and development on text manipulation systems have been extensive (8). Expanding DMTN commands that full English words be straightforward. One technique used at SRI is to have the computer recognize and complete a word as soon as the first characteristic letters have been typed. Thus, when the user wishes to display a route on a map he types in R and before he can type further computer responds with ROUTE, the completed English word.

A final comment on the iterative nature of the DHTM—the simulation predicts volumes on streets and the resulting delays at intersections throughout the network for a 2-hr period and then repeats itself using predicted delays to generate new volumes. Interruption during an iteration of DHTM will not prove useful to the traffic engineer because the simulation is at some intermediate point in its solution process and some of the internal data files will be for time period, i, and some time period, i + 1, and the user would have difficulty identifying which. On the other hand, interruption of the program after an iteration is complete allows the user to see vehicles avoid or become attracted to a previously "improved" element of the network. This capability would be of great value to the traffic engineer in his evaluation of a particular improvement alternative.

A New Intersection Study Technique

ZOLTAN ANTHONY NEMETH and JOSEPH TREITERER, The Ohio State University

The objectives of this research were to explore possibilities of more accurate and efficient methods of field measurements at intersections and to study new techniques that would assist in determining whether the best use is being made of traffic control devices. Two data collecting methods were developed utilizing time-lapse cameras suspended over the center of intersections. One method involved a remote-controlled 16-mm Bolex camera installed on a rotating platform. The second method, based on a 70-mm Maurer P. 2 camera, used four mirrors to bring all four approaches into the field of vision. Simultaneous traffic data could thus be obtained on all approaches. A digital simulation model was developed and programmed for the IBM 7094 computer. The technique, involving the combined use of the split-image camera and a simulation model of signalized intersections, was tested on a limited scale. The model has been validated by data obtained at a signalized intersection. The effect of changes in the timing of the semiactuated signal has been simulated. The model can also be used to simulate pretimed and fully actuated traffic signals. The three different signal types were compared. Under the traffic conditions investigated, the semiactuated signal was found to be the least efficient. The pretimed signal resulted in reduced stopped-time delay rates, but the most efficient operation was provided by the fully actuated signal.

PERSONAL mobility provided by private automobiles is a prized element in the quality of American living. The increasing rate of urbanization, however, tends to concentrate traffic demands and accelerate problems of roadway transportation. Difficulties are compounded at intersections. The function of an intersection is to merge, diverge, and cross traffic streams. The efficiency with which this function is performed depends largely on the geometric layout and traffic control in relation to prevailing traffic flow characteristics. Installation of traffic signals is usually preceded by manually conducted traffic studies. Once the signal is installed, continued attention is frequently limited to maintenance of equipment. But traffic conditions change over the years. Intersections that no longer meet current traffic requirements are bound to increase the cost of transportation in terms of delay and accidents.

This study was undertaken as part of research project EES-274, "Development of New Intersection Study Techniques," sponsored by the Ohio Department of Highways in cooperation with the U. S. Bureau of Public Roads. The objective was to develop a new intersection study technique. The specific aims were twofold:

1. To develop an accurate and efficient field observation method, utilizing time-lapse photography that is capable of obtaining simultaneous traffic data on all approach lanes; and
2. To develop a mathematical model of a signalized intersection, programmed for the IBM 7094 digital computer, which is flexible enough to permit the simulation of either pretimed, semiactuated or fully actuated traffic signals.

Paper sponsored by Committee on Traffic Control Devices and presented at the 48th Annual Meeting.

The new intersection study technique involves the combined application of the field observation method and the simulation model. Once the model has been adapted to a specific intersection by input data obtained from photographic data collection, the various facets of operating the intersection are studied by experimenting on the model. Traffic demand and traffic control variables are manipulated systematically and the interrelationship is analyzed. By testing alternative traffic control schemes and observing the results, the most desirable scheme can be identified.

DEVELOPMENT OF PHOTOGRAPHIC INSTRUMENTS

When photographic techniques are employed in traffic engineering studies, the position of the camera largely determines the quality and quantity of data. The field of view of a camera mounted on a tripod at street level would generally be inadequate for intersection studies. Better coverage can be obtained by mounting the camera on a hydraulic platform truck. A camera mounted on the roof of a tall building provides the best view if the building is close to the intersection and if other structures do not obstruct the view. Dependence on a conveniently located tall building, however, is a rather serious limitation.

In this study, the cameras were suspended by a steel cable over the center of the intersection. Two cameras were used: a 16-mm Bolex time-lapse camera installed in a housing rotated by remote control, and a 70-mm split-image camera.

Rotating Camera

The equipment consisted of a 16-mm Bolex time-lapse camera mounted on a rotating platform, suspended from a cable attached to the supporting poles of the traffic signal. Electrical power was drawn from the control box. The 10-mm lens has an angle of view of 58° × 38°, which is wide enough to bring both the stop line and the horizon into the field of vision. The platform also housed a television camera (Figs. 1 and 2). The operator, who controls the equipment with remote-controlled

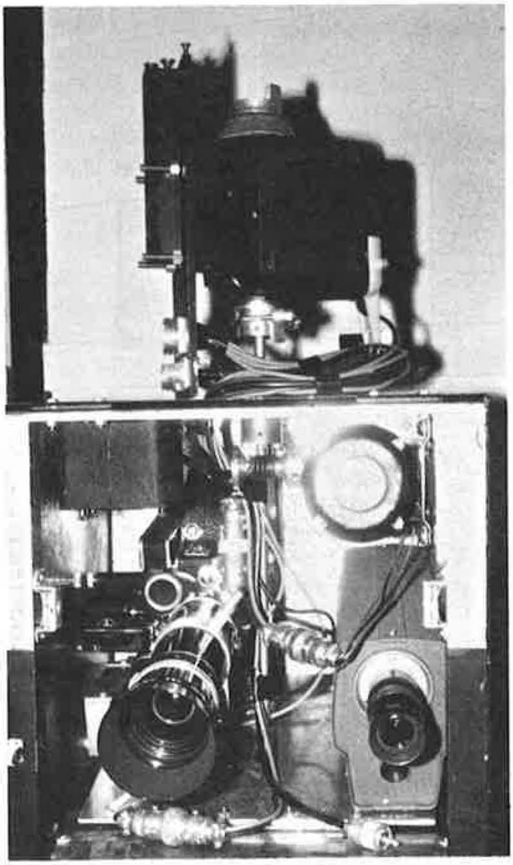

Figure 1. Rotating platform with the Bolex and the television cameras.

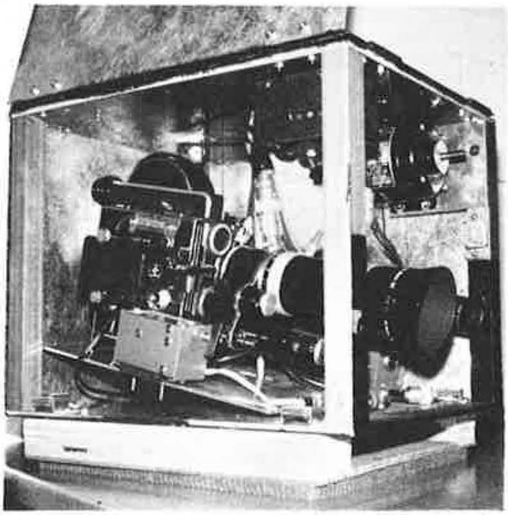

Figure 2. Remote-controlled rotating camera.

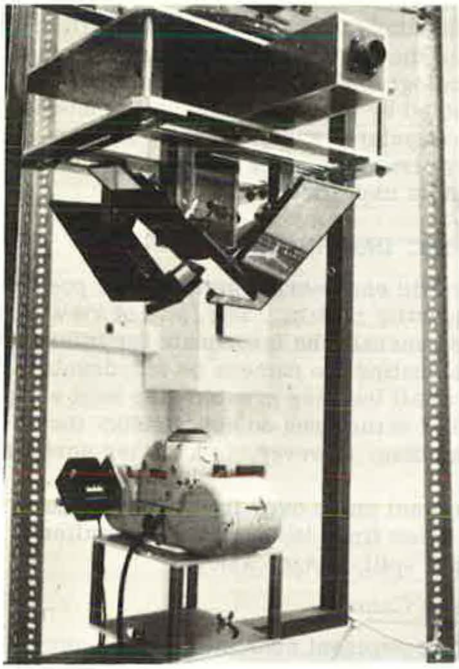

Figure 3. The 70-mm camera with mirror system.

Tenna-Rotor, can rotate the camera to the desired direction by viewing a closed-circuit television receiver. Both cameras can be tilted on a horizontal axis to insure the proper field of vision.

Split-Image Camera

The second instrument was designed to provide traffic data on all four approaches simultaneously. Based on a 70-mm Maurer P.2 camera, it was made available by the Wright-Patterson Air Force Base. The camera was mounted vertically on the bottom of a frame. It was aimed at a set of four mirrors installed above camera (Fig. 3). Each mirror was adjusted to bring one of the four approaches into view. Between the four mirrors, a data chamber contained a set of lights synchronized with the traffic signal lights. Each photograph, therefore, recorded the traffic condition on all approaches and the state of the traffic signal (Fig. 4).

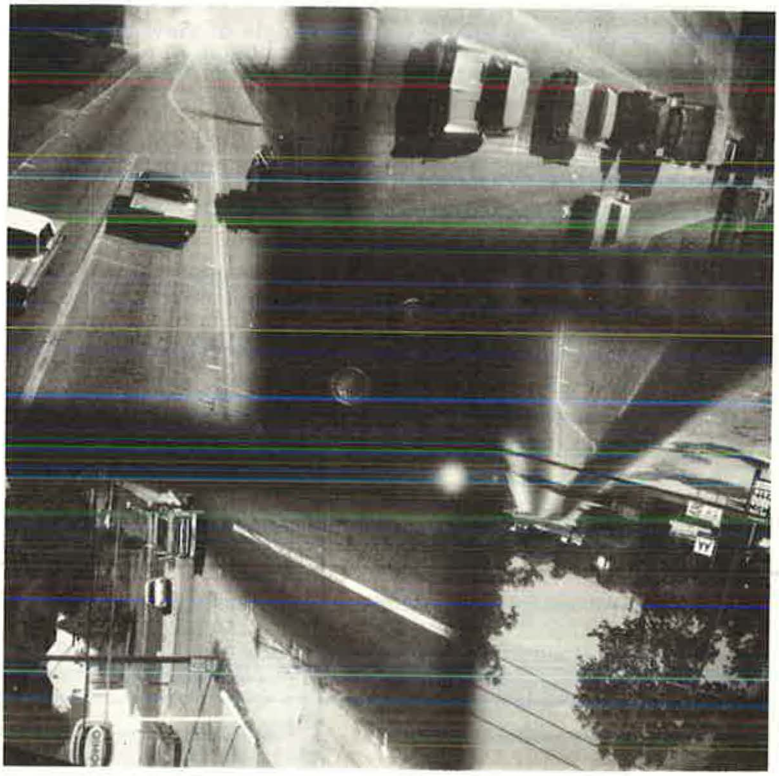

Figure 4. Photograph taken by split-image camera.

The camera was operated from a control console installed in the back of a station wagon parked near the intersection. A frame counter, within the console, recorded automatically the movement of the film through the camera. This camera can be used with 50- or 100-ft magazines. At one frame per second exposure rate, the 100-ft roll can record approximately nine minutes of traffic data.

Figure 5. Installation of the rotating camera.

Field Installation

A hydraulic lift truck was used to raise the equipment and personnel (Fig. 5). The truck was parked so that traffic could flow as freely as possible. Apparently, the camera did not attract attention during operation. Suspended between several traffic signal heads and directional signs, it was quite inconspicuous (Fig. 6).

Rear Projection Console

The Maurer P.2 camera uses 70-mm film. A motion picture projector capable of handling this film is expensive and not readily available. Therefore, a Beseler Slide King projector, originally designed to handle slides up to a $3\frac{1}{4}$- by 4-in. size, was converted to our specifications (Fig. 7). Two auxiliary reels powered by an electric motor were added to advance the film through the projector. The motor, controlled from the viewing console, can be operated either continuously at an approximate rate of 4 sec per frame or finer adjustments can be made by depressing a pushbutton. The image is projected on a glass plate covered by drafting paper and built into the top of a console (Fig. 8).

Before installation of the camera, white paint marks were sprayed at 25-ft intervals at both edges of all approach lanes. The marks were only 3 in. wide and 12 in. long to avoid any biasing effect on the traffic data. Most of the marks are visible on the photographs and provide a built-in reference scale. Missing marks were reconstructed by a technique based on the fundamental principle of projectivity (the theory of cross ratio). The same principle was utilized to develop from the 25-ft-interval marks a grid on which

Figure 6. The rotating camera in position.

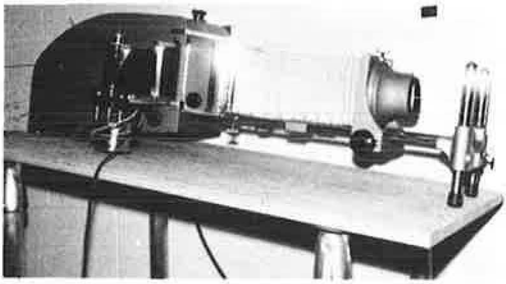

Figure 7. Revised Beseler projector.

every 5-ft line is shown. The grid system extended 200-ft on all approach lanes. A Kodak Analyzer was used to analyze the 16-mm film.

Advantages of the Photographic Technique

Application of the photographic technique to the investigation of complex traffic problems at intersections has numerous advantages over more conventional methods. It is especially valuable when personnel are limited. A two-man crew can install the camera and collect the required data. Data analysis can be performed by one person in the laboratory. Some information readily obtainable by photographic techniques is very difficult to collect by other techniques. The advantages of the photographic technique increase with the complexity of the data, because the film is a permanent record of all visible aspects of the traffic situation at an intersection. The film can be projected repeatedly at the convenience of a laboratory. Furthermore, it provides a 100 percent sample, and the method of analysis can be changed to meet the accuracy required for a specific purpose, It is most useful where the interrelationships of several factors are investigated because all factors are observed simultaneously.

The development of the split-image camera provides a practicable method by which simultaneous traffic data can be obtained on all approaches to an intersection. The results of this study suggest that (a) simultaneous data (including traffic volumes, arriving times and stopped-time delay) are necessary for any meaningful investigation of intersection operation, and (b) time-lapse photography is the best method to obtain these data.

A DIGITAL SIMULATION MODEL OF SIGNALIZED INTERSECTIONS

Purpose

The objective of this phase of the study was to develop a model that could simulate the operation of a signalized intersection so accurately that the effect of traffic control could be predicted. In the model, this control can be effected by fixed-time, semi-actuated, or fully actuated traffic signals. The efficiency of control is measured in terms of delay. For each lane the number of stopped vehicles and the sum of the stopped-time delay are determined.

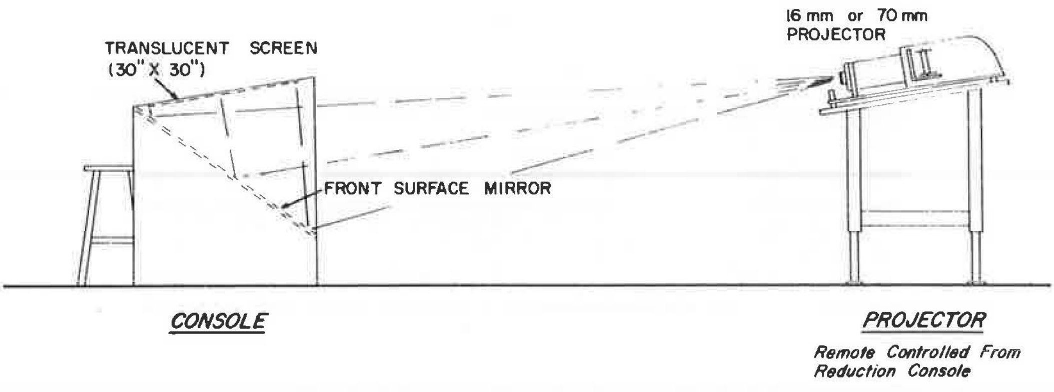

Figure 8. Setup of the rear projection console.

Structure

The model is a computer program which, when given a certain set of input parameters related to the intersection, manipulates these components so that they will act in a manner comparable to a real intersection.

The three components of an intersection are the physical properties of the intersection, the traffic control, and the traffic flow. The properties of the components are represented by three methods: (a) mathematical equations, (b) variables, and (c) statistical distributions. The time-scanning technique is used to update the system. At each time period, the following procedure is repeated:

1. The traffic signal is checked and reset if required;
2. The list of arriving times is checked for arriving vehicles and the list of vehicles in the system is updated if necessary;
3. The position and velocity are recomputed and a new acceleration rate is assigned for each vehicle as dictated by prevailing conditions;
4. If any vehicle has exited the system, the list of vehicles in the system is updated; and
5. Current time is checked against the time limit; if the time limit has been reached, the results of the simulation are printed out and the run is terminated.

The model simulates an intersection with two approach lanes from each direction. The minimum length of the simulated approach lanes is 200 ft. Beyond the stop line, the model considers only that portion of a traffic lane where a potential conflict exists between left-turning vehicles and straight-through or right-turning traffic.

The vehicles are represented by a set of double-subscripted variables. The subscripts identify the lane in which the vehicle is traveling and the order in which the vehicles arrive. Three of the variables are recomputed during each scanning interval: (a) the position, POS(I,J); (b) the velocity, V(I,J); and (c) the acceleration rate, A(I,J). The simulation procedure is summarized in Figure 9.

Description

The program deck is made up of 18 parts; the initialization, 16 subroutines, and the data deck.

Subroutine TGI—Traffic Generator—When arriving times are not read in as input, subroutine TGI is used to generate traffic by a random process. The hourly rate of flow, VOL(I) is read in for each lane. The corresponding headway distribution is approximated by the log-normal distribution. The probability density function is given as follows:

$$f(x) = \begin{cases} \dfrac{1}{x\sigma\sqrt{2\pi}}\, e^{-\dfrac{(\lg x - \mu)^2}{2\sigma^2}} & x > 0 \\ 0 & x \leq 0 \end{cases}$$

For the mean μ and the variance σ^2 the maximum liklihood estimates $\hat{\mu}$ and $\hat{\sigma}^2$ are substituted. From field data consisting of samples ranging from 339 vph to 1369 vph, $\hat{\mu}$ and $\hat{\sigma}^2$ values were computed. Regression equations were then developed using volume as the independent variable and the mean and variance as dependent variables:

$$\hat{\mu} = \text{YMEAN(I)} = 2.076 - 0.001 \text{ VOL(I)}$$
$$\hat{\sigma}^2 = \text{VARY(I)} = 0.5670 - 0.00006 \text{ VOL(I)}$$

The probability of a headway occurring between $x - \frac{1}{2}$ sec and $x + \frac{1}{2}$ sec is obtained by integrating the probability density function over the interval from $x - \frac{1}{2}$ to $x + \frac{1}{2}$ sec. The Monte Carlo method is then used to generate headways for traffic arriving on a given lane.

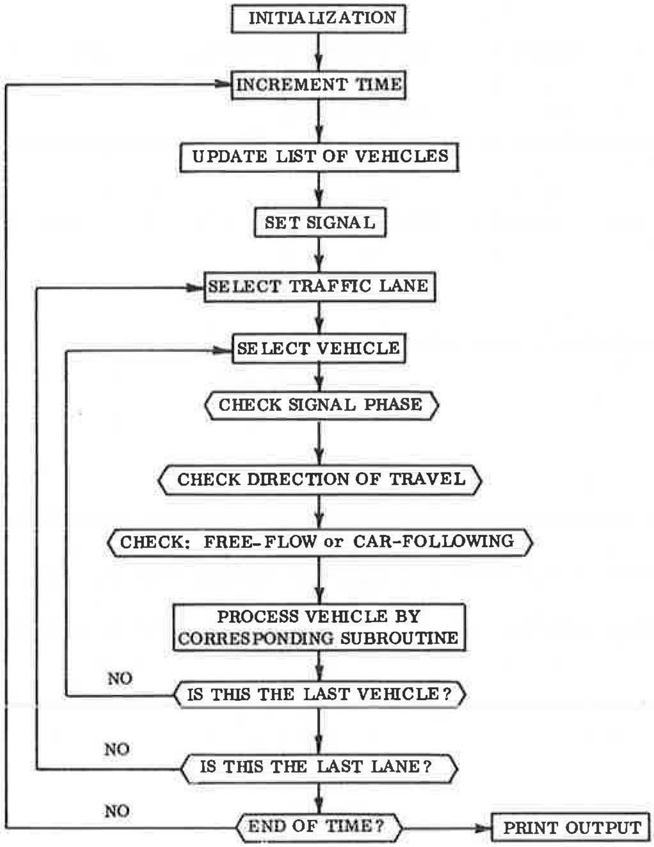

Figure 9. Summary of simulation process.

Subroutine SC2—Pretimed Signal—Subroutine SC2 is included in the program only when pretimed signal control is simulated. The corresponding input data include the length of the cycle as well as the starting time of all phases, measured from the begining of the cycle. During each time increment, the current time is compared with the starting times of the phases.

Subroutine SC3—Semiactuated Signal Controller—Subroutine SC3 replaces SC2 when semiactuated traffic control is simulated. It is used in conjunction with Subroutine SD5, which simulates the operation of the detectors.

Subroutine SC4—Fully Actuated Signal Controller—For fully actuated signals, both the minimum and the maximum green phases are specified for both directions. The vehicle detectors are simulated by Subroutine SD6.

Subroutine UD7—Update List of Arrived Vehicles—During each time increment, current time is compared with this list of arriving times. If it matches one of the items in the list, then the following steps are taken:

1. The position of the vehicle is defined as the entrance line. If there is a queue extending past the entrance line, the newly arrived vehicle is moved back as far as necessary.
2. For vehicles in the outside lane, the direction of travel is determined by the Monte Carlo method. All vehicles in the inside lane are to turn left.
3. The two variables, NVP(I) and KSUM(I), are incremented by one. The first variable indicates the number of vehicles present in the lane considered; the latter variable indicates the number of cars that have arrived.

Subroutine QU8—Queue Discharge—The simulation of a platoon starting up at the beginning of the green phase is based on the following assumptions: (a) a starting delay time separates the beginning of the acceleration of the first vehicle from the beginning of the green phase; (b) a constant response time separates the movement of any vehicle in the platoon from the movement of the preceding vehicle.

Subroutine FG9—Free Flow on Green; Straight-Through or Right-Turning Traffic—The velocity of a straight-through vehicle is compared with its desired velocity. If the difference exceeds a specified margin, a uniform acceleration rate is assigned to the vehicle. Right-turning vehicles are to assume the turning velocity when they reach the stop line. The acceleration rate is computed accordingly.

Subroutine CFG10—Car-Following on Green; Straight-Through or Right-Turning Traffic—Based on the current relative position of two subsequent vehicles, an acceleration rate is assigned to the following vehicle if there is a difference in the velocities. The minimum spacing to be maintained at a given speed is equal to the sum of the specified minimum spacing between stopped vehicles and the distance traveled during one second. If a right-turning vehicle is following a straight-through vehicle, the acceleration rate required to reduce the velocity to the turning velocity is also computed. The lower of the two rates will be applied.

Subroutine FG11—Free Flow on Green; Left Turns—When the left-turning vehicle approaches the intersection within a predetermined distance, gaps in the opposing traffic stream are investigated. All time lags or time gaps equal to or greater than 5 sec are accepted. If the gap is not acceptable, the turning vehicle decelerates to stop at the waiting point.

Subroutine CFG12—Car-Following on Green; Left Turns—This car-following Subroutine is based on the same principles as Subroutine CFG10.

Subroutine FA13—Free Flow on Amber; Straight Through and Right Turns—A vehicle traveling in the outside lane when the signal changes to amber will enter the intersection only if it cannot stop at a moderate deceleration rate.

Subroutine FA14—Free Flow on Amber; Left Turns—Delayed left-turning vehicles tend to take advantage of the caution phase and complete the turning movement. Consequently the stop-or-go decision is based on a different assumption from that in Subroutine FA13. It is assumed here that a "go" decision is made if the left-turning vehicle can enter the intersection on amber.

Subroutine TR15—Traffic Flow on Red—The first vehicle in each lane stops at the stop line. The following vehicles are governed by the car-following equation which provides a uniform spacing between stopped vehicles.

Subroutine WO16—Write Output—Subroutine WO16 specifies what is to be printed as output, depending on the objectives of a given simulation run. The type of traffic control is identified and the input parameters are also printed out. The tabulated results characterisiically include the specified traffic volume, the actual generated volume, the number of stopped vehicles, and the stopped delay per lanes.

TESTING THE NEW TECHNIQUE

Scope

The digital simulation model was designed to be employed in conjunction with the split-image camera in the study of signalized intersections. The technique was tested at the intersection of Ohio 3 and Ohio 161. The test achieved the following three objectives:

1. Simultaneous traffic data on all four approaches to the intersection were obtained by the split-image camera;
2. The simulation model was adapted to this intersection by thorough comparison of simulated and observed situations; and
3. The application of the technique to the investigation of various traffic-control elements in relation to their effect on the traffic flow was demonstrated.

Results

Approximately 16 minutes of traffic data were obtained on two 100-ft rolls of 70-mm film and used in the preparation of the traffic input for the simulation runs. The input consisted of the arriving times, average approach speeds, average turning speeds, and turning volumes. The intersection was controlled by a semiactuated traffic signal. Traffic control input included the minimum and maximum green phases on Ohio 3; the minimum green phase, the unit extension time, and the location of the detectors on Ohio 161; and the yellow phase and the all-red period for both highways. The results of the simulation runs were the subject of intensive comparative analysis (Table 1). The simulated results represent the arithmetic mean of three simulation runs. The agreement between the observed and the simulated conditions is quite close. Comparing the 2940-sec stopped delay time observed during the total 16-min study period with the 2879 sec obtained through simulation, the difference is only 2.1 percent. The difference between the simulated and the observed average stopped delays per stopped vehicles is 1 sec. The agreement is even better when stopped delays for all vehicles are compared: observed and simulated delays are equal on Ohio 3; the difference is 0.4 sec on Ohio 161; and the difference, considering the whole intersection, is 0.3 sec.

Once the model was adapted to the study site, it was used to evaluate the effect of various changes in traffic control. Timing of the semiactuated signal was systematically revised, and the resulting changes in delay were observed. It was found, for example, that under conditions represented by the traffic data, total stopped-time delays could be reduced significantly by reducing the minimum green phase on Ohio 3 and increasing the vehicle extension time on Ohio 161. The reduction of the minimum green phase to 25 sec on Ohio 3 results in a 10 percent reduction in the simulated stopped delay at the intersection. The reduction in delay effected by a 1 sec increase in the unit extension time is 8.7 percent. When both changes are introduced simultaneously, the total stopped-time delay is reduced by 17.5 percent, or nearly the sum of the reductions achieved separately by each change in the signal timing. The detailed results of the improvement (referred to as improved semiactuated signal) in the timing of the signal are given in Table 2.

Considering that the minor roadway carried higher traffic flow during the study period than the roadway favored by the signal, it seems that the semiactuated signal is not well suited to the control of this intersection. Although it was shown that changes in the signal timing could result in significant reduction of delay, other types of traffic signals were expected to perform better. The efficiency of the semiactuated signal control was compared with the efficiency of the fully actuated and the pretimed signal control (Table 2). The fully actuated signal is timed to have 12-sec minimum green

TABLE 1

SIMULATION COMPARED WITH DATA—COMBINED FILMS D3A AND D3D
(16 minutes and 7 seconds)

Roadway	Ohio 3	Ohio 161	Combined
No. of vehicles during study period	110	140	250
No. of stopped vehicles:			
Data	39	94	133
Simulated	34	104	138
Stopped delay time, sec:			
Data	501	2439	2940
Simulated	503	2376	2879
Difference, percent	0.4	2.6	2.1
Stopped delay per stopped vehicles, sec:			
Data	12.8	25.9	22.1
Simulated	14.8	22.8	20.8
Difference, sec	2.0	3.1	1.3
Stopped delay per all vehicles, sec:			
Data	4.6	17.4	11.8
Simulated	4.6	17.0	11.5
Difference, sec	0	0.4	0.3

TABLE 2

COMPARISON OF THREE TYPES OF TRAFFIC CONTROLS
(Data: Film D3D—521 sec)

Traffic Control	Ohio 3	Ohio 161	Combined
No. of vehicles	64	89	153
No. of stopped vehicles:			
Semiactuated:			
Present	24	57	81
Improved	27	51	78
Pretimed:			
50/50 split	29	52	81
42/58 split	35	41	76
Fully actuated	35	54	89
Stopped delay time, sec:			
Semiactuated:			
Present	409	1321	1730
Improved	508	920	1428
Pretimed:			
50/50 split	501	893	1394
42/58 split	608	772	1380
Fully actuated	388	609	997
Stopped delay time per stopped vehicles, sec:			
Semiactuated:			
Present	17.0	23.2	21.4
Improved	18.8	18.0	18.3
Pretimed:			
50/50 split	17.3	17.2	17.2
42/58 split	17.4	18.8	18.2
Fully actuated	11.1	11.3	11.2
Stopped delay time per all vehicles, sec:			
Semiactuated:			
Present	6.4	14.9	11.3
Improved	7.9	10.3	9.3
Pretimed:			
50/50 split	7.8	10.0	9.1
42/58 split	9.5	8.7	9.0
Fully actuated	6.1	6.8	6.5

and 40-sec maximum green phase on both highways. The vehicle detectors are located 110-ft from the stop line. The pretimed signal is tested with two designs; with 50/50 split of the green time and with 42/58 split. The latter is based on the distribution of the traffic volumes on the two highways. The cycle is 60 sec in both cases. The delay obtained by the pretimed signals is much lower than that obtained by the semiactuated signal at the present, and slightly lower than that obtained by the improved timing of the semiactuated signal. The total delay is not reduced significantly by changing the split from 50/50 to 42/58. However, the number of stopped cars is reduced and is more evenly distributed on the two roadways. It can be concluded that under the condition represented by the traffic data on film D3D, the pretimed signal is more efficient than the semiactuated signal. A further reduction in delay is observed when the intersection is controlled by a fully actuated signal. The average stopped-time delay per vehicles is reduced from 11.3 to 6.5 sec. This represents a very significant reduction of 42.5 percent. The trend in Ohio is to phase out the semiactuated signals in favor of the fully actuated signals. The proportion of expected saving in stopped-time delay and the corresponding saving in transportation cost provide strong support for this policy.

CONCLUSIONS AND FUTURE PLANS

The study procedures are summarized in the form of a block diagram in Figure 10. The major achievements can be related to four phases of the study.

1. A technique of obtaining simultaneous traffic data on all approaches of an intersection by a split-image camera, suspended over the center of an intersection, was successfully developed and tested.

2. An inexpensive rear-projection console was built and tested in conjunction with a modified slide projector in the analysis of the 70-mm film.

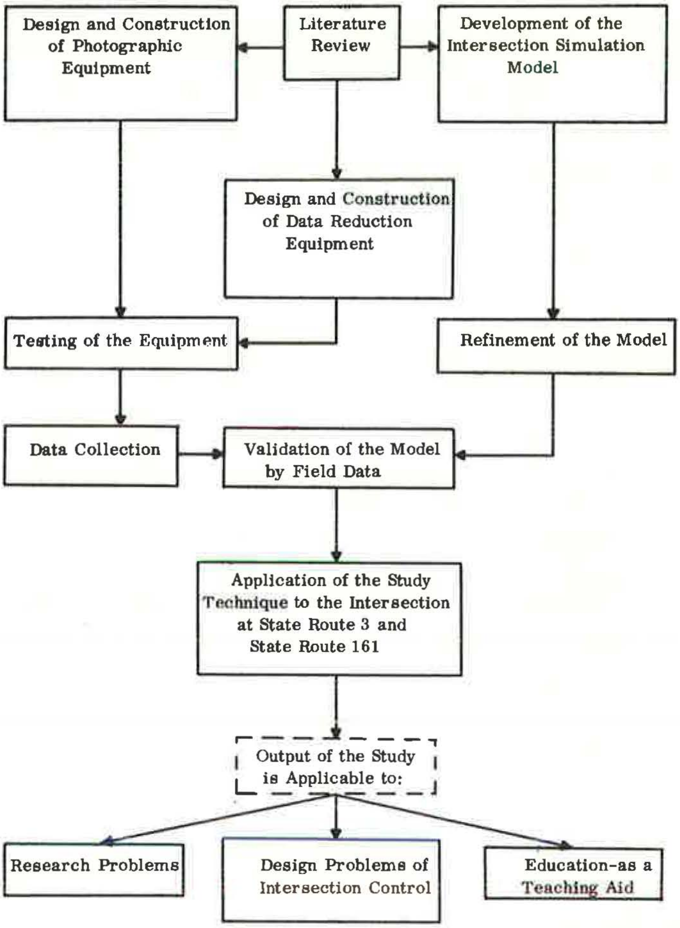

Figure 10. Schematic summary of the research.

3. A versatile digital simulation model of a signalized intersection was developed and tested. The results of the simulation compared well with field observations.
4. The efficiency of the intersection operation was successfully related to alternative schemes of traffic control by simulation.

The limitations of the technique are the following:

1. The split-image camera unit is based on a relatively expensive and rare camera;
2. The capacity of the camera is limited to approximately nine minutes at one frame per second; and
3. Vehicle movement within the center of the intersection cannot be recorded.

Future plans call for the elimination of these limitations. A new unit will be designed and built, based on a 35-mm camera. The capacity of the magazine should be much higher than that of the 70-mm camera. The center of the intersection will be brought into the view of the camera. The improved data collection method and the simulation model will be used in a study aimed at the investigation of the effect of left-turning storage lanes on the efficiency of intersection operation.

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2. Nemeth, Zoltan Anthony. The Development of a New Intersection Study Technique. Unpublished Ph. D. dissertation, Ohio State Univ., 1968.

Discussion

ROBERT L. MORRIS, Texas Instruments Incorporated—While there are many advantages to the photographic intersection study technique described, it seems that methods using conventional vehicle detectors could prove useful. One alternative system would consist of vehicle detectors in each of the approaching lanes of the intersection combined with a recorder that would have as inputs the state of the traffic signal, the vehicle detector outputs, and the time of day. Each lane would have a detector at the stop line and at least one detector some distance back from the stop line. The intersection should be chosen so that the spacing between detectors would bot be constrained by minor entrances to the roadway or proximity to adjacent intersections. If space allows, additional detectors could be installed further away from the intersection so that characteristics of the traffic flow could be determined before reaching the direct influence of the intersection signal. The recorder storage medium would be of a type that is readily machine-transferable to computer storage.

The recorded vehicle count from the detectors could be the total for a set time period or for each phase interval of the intersection signal. A small, general purpose computer would be programmed for processing the recorded data to determine the efficiency of the intersection control system. Using available vehicle detectors, the exact number of vehicles stopped for each phase of the intersection controller cannot be obtained as it can with photographic methods. However, an approximation to the number of stopped vehicles could be obtained by computer calculation, and other quantities such as average speed, density and volume of the traffic flow for each lane could also be obtained. It is possible that this measurement technique could show that maximizing the volume while minimizing density could be a meaningful and readily measured criterion for the efficiency of an intersection control system.

The cost of installing permanent types of vehicle detectors could not normally be justified for an intersection survey. There could be some cases, however, where additional permanently installed detectors could be used in more complex controllers planned for the future. For the usual case, temporary vehicle detectors would be used to supplement those existing detectors. A version of the pneumatic tube-type detector could be useful; for multilane roads a transition to rigid tubing can be made for crossing adjacent lanes in which detection is undesired. Another possibility is the development of a "tape-on loop" for a temperory RF loop detector or a thin-film or Hall effect magnetometer-type detector that could be temporarily attached to the road surface. Vehicle detectors of the type that produce a single indication per vehicle would be preferred to the axle-counting types.

Advantages of using vehicle detectors for intersection study include simpler installation, simpler operation (reloading the camera would mean traffic interruption, a special vehicle, and additional personnel), and less complex equipment from the view point of the usual signal installation maintenance organization. The survey time capacity of the photographic technique is presently nine minutes; this could be improved somewhat, but could not approach a continuous 24-hr surveillance on a practical basis, whereas 24-hr surveillance could easily be accomplished by the use of a recorder suitable for storing the vehicle detector output and signal data. Unsuspected conditions could be discovered using a 24-hr surveillance, whereas with a limited-time capacity, a judgment must be made on what time periods are meaningful. Transfer of the data from the recorder to processor would be done by machine, as opposed to the tedious process of human examination of the photographic data. An intersection control study based on data from vehicle detectors could give additional insight into the use of these devices in traffic control systems.

Optimal Selection of a Progression for a Two-Way Arterial Street

JOHN B. KREER and WAYNE D. PANYAN,
Michigan State University

A procedure is demonstrated for selecting a progression on a two-way street which minimizes total steady-state vehicle transit time while taking into account constraints by driver car-following behavior.

•IT is common practice to coordinate the timing of the traffic signals on urban streets so as to achieve progressions. When this is done, it is possible to design the progression on arterial streets so that vehicles traveling at the design speed will encounter at the most one red light. The parameters of the progression, such as cycle length, splits, and offsets, can be determined graphically from a time-space diagram (Fig. 1) by trial and error. From such a diagram it is also possible to determine the bandwidth of the progression, i.e., the time headway between the first and last cars of the platoon (traveling at the design speed) that can pass through the green signal at a given intersection without encountering red signals at subsequent intersections.

The progression design is not unique for any given street in that there are many combinations of progression speeds and bandwidths that can be realized. If the signal timing is implemented by fixed timers at local controllers and if the street carries traffic in only one direction, perhaps the best choice is an average speed that will accommodate medium density traffic. However, when progression is applied to two-way streets, an increase in the bandwidth in one direction is usually accompanied by a decrease in bandwidth in the other direction. Several papers have been published on the trade-offs that might be made and on techniques for selecting bandwidths that are optimum in the sense that (a) both bandwidths are equal and maximum or (b) one is maximum subject to the restriction that the other does not drop below a preassigned minimum. Very little, if any, work has been reported on the design of progressions to meet given flow demands on the arterial street and its cross streets.

This paper develops a rational basis for realizing the optimal progression on two-way streets as a function of the prevailing flow demands. It is both economically and technically feasible to automatically change the timing of traffic signals, by means of a central digital controller, in accordance with the measurable demands at any given time. Initial efforts in this direction have already been undertaken in Toronto, San Jose, and Wichita Falls (1, 2, 3). These early efforts indicate that significant gains can be obtained by adjusting the timing of traffic signals to the prevailing demand in an on-line manner. Several cost studies have shown that even small improvements in the flow of traffic can result in large economic benefits (3, 4). The specific benefit depends largely on the criteria used by the central digital controller in deciding which stream of traffic should receive the green signal at any given instant of time.

In addition to matching the operation of the network to the demands on the system, the control algorithm must also minimize the effects of random disturbances in the system. The control algorithm presented in this paper assumes that changes in demands on a system are sufficiently slow so that optimization of the control action can be achieved technically by considering a steady-state model of the traffic network and

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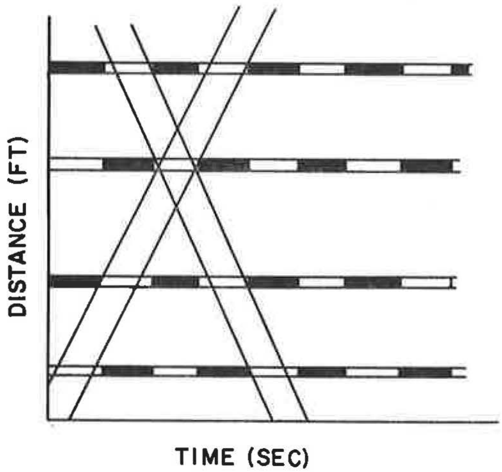

Figure 1. Typical time-space diagram.

that, as a practical expedient, control in the face of random disturbances can be achieved by short-term localized modifications of the progressions.

The demand on an arterial is measured in terms of the number of vehicles per unit of time arriving at the entry points to the system. For the purposes of the discussion, let the points of entry be the two ends of the street. If the flow rate in each direction on the arterial is exactly equal to the demand, then of course no queuing occurs and the progression is said to be operating satisfactorily.

The first question of concern is to determine the flow rate that a given progression will sustain. The classical time-space diagram analysis gives only the bandwidth or time headway between the first and last vehicles in the platoon. The number of vehicles per unit of time that can be served by the street depends not only on the bandwidth of the progression but also on the density of the vehicles in the platoon. The relationship between these quantities can be obtained from the theory of traffic flow.

Definitions for the symbols used in the following discussion are:

Symbols	Definition
B_i	bandwidth—inbound direction
B_o	bandwidth—outbound direction
B	maximum equal bandwidth
G	minimum green time
y	flow rate
y_s	continuous stream flow rate
y_{sm}	maximum continuous stream flow rate
y_{di}	demand—inbound direction
y_{do}	demand—outbound direction
x	lane occupancy
x_j	lane occupancy resulting in jam
v	velocity
v_i	progression speed—inbound direction
v_o	progression speed—outbound direction
v_f	free speed
v_e	progression speed equal in both directions
F	performance index
μ	Lagrange multiplier

Various experimental and theoretical investigations (5, 6, 7) of the flow in a single lane of traffic have established that flow rate is related to the lane occupancy by a parabolic relationship of the form shown in Figure 2. The corresponding speed of the traffic stream varies with lane occupancy (Fig. 3). The behavior indicated in Figure 3 has been shown by car-following models (7) to be the result of the drivers in the traffic stream reducing their speed when confronted with smaller intervehicular spacing (in-

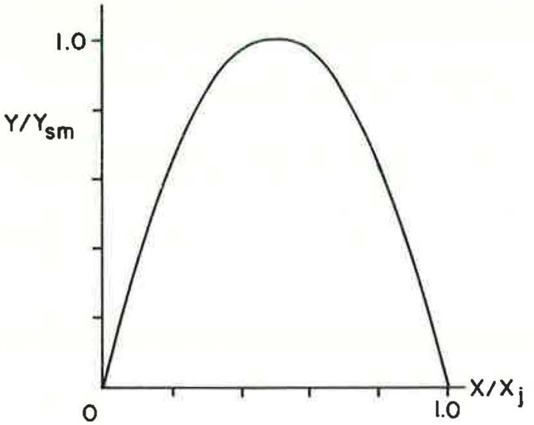

Figure 2. Normalized flow rate (y/y_{sm}) as a function of normalized lane occupancy (x/x_j) for a continuous traffic stream.

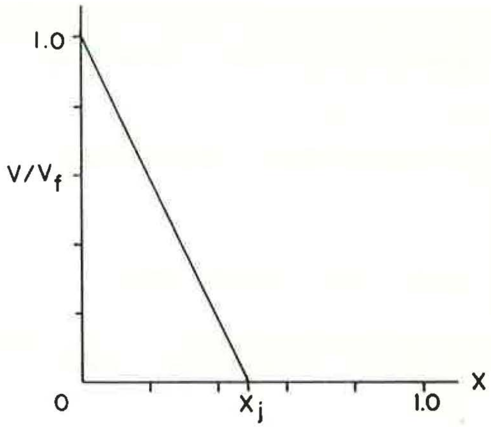

Figure 3. Normalized speed (v/v_f) as a function of lane occupancy (x) for a continuous traffic stream.

creased lane occupancy). Inasmuch as the same basic phenomenon takes place in a platoon as in a continuous stream of vehicles, the maximum flow rate that can be expected without breakdown of the progression is given by the product of the continuous stream flow-occupancy characteristic and the ratio of bandwidth to cycle length. For example, if the continuous stream flow-occupancy characteristic of Figure 2 is given by $y_s = G(x)$ where y_s is the continuous stream flow rate and x is the lane occupancy, the flow-occupancy characteristic for a progression system is given by $y = BG(x)$ where B equals bandwidth per cycle. Thus, the flow-occupancy characteristics in each of the two directions on the arterial are different if the ratios of bandwidth to cycle length are different in the two directions; that is, if $B_i \neq B_o$.

In the analysis which follows, Greenshield's traffic model (6)

$$y = 4y_{sm}\left(\frac{x}{x_j}\right)\left(1 - \frac{x}{x_j}\right) \tag{1}$$

$$v = v_f\left(1 - \frac{x}{x_j}\right) \tag{2}$$

is used, where y_{sm} is the peak flow rate, x_j is the jam lane occupancy, and v_f is the free speed. Comparable results can be obtained using other models which have been proposed.

In a continuous steady-state traffic stream, the vehicles adjust their speeds so that the flow rate just satisfies the demand, provided the capacity of the street is not exceeded. Up to the point of saturation, increased demand is accommodated by higher lane occupancy and a lower individual vehicle speed. If the demand exceeds the capacity of the street, congestion very quickly results upstream. In an analogous fashion, the service flow rate of a two-way progression system should be adjusted to the demand. More specifically, the service flow rate in each direction is given by the product of the bandwidth-to-cycle ratio and the continuous stream flow rate that would prevail on that street at the progression speed. When this service flow rate is just matched to demand in both directions, the bandwidths can be expected to be filled with vehicles. Setting the progression speed higher than the value for which demand and service rate are balanced invites queuing and congestion, while setting it lower implies a lower quality of service than necessary.

If x is eliminated from Eqs. 1 and 2, the result is

$$y = 4y_{sm}\left(\frac{v}{v_f}\right)\left(1 - \frac{v}{v_f}\right) \tag{3}$$

Applying the principle that bandwidths and progression speeds should be selected so that service rates equal demand requires

$$y_{di} = 4B_i y_{sm}\left(\frac{v_i}{v_f}\right)\left(1 - \frac{v_i}{v_f}\right) \tag{4}$$

and

$$y_{do} = 4B_o y_{sm}\left(\frac{v_o}{v_f}\right)\left(1 - \frac{v_o}{v_f}\right) \tag{5}$$

where y_{di} and y_{do} are the inbound and outbound demands and v_i and v_o are the progression speeds.

A reasonable method of apportioning the potential service between the traffic demands in the two directions is to select the progression which minimizes total vehicle hours of travel time in the system. A measure of total vehicle hours of travel time is

$$F = \frac{y_{di}}{v_i} + \frac{y_{do}}{v_o} \tag{6}$$

Note that F is directly proportional to total vehicle hours under steady-state operation, since y is a measure of the number of vehicles served in any interval of time, and the individual trip time is proportional to the reciprocal of the steady-state velocity. F must be minimized while taking into account the constraints of realizability of the bandwidths and progression speeds selected.

Before proceeding with the optimization process, a few remarks about progression speeds are necessary. It is possible to vary the progression speeds within the constraint

$$\frac{1}{v_i} + \frac{1}{v_o} = \frac{2}{v_e} \tag{7}$$

without changing the bandwidths, where v_e is the progression speed that would prevail for these bandwidths if v_i and v_o are equal. A change in the offset of each signal will force drivers in one direction to reduce their transit time between intersections by the same amount that drivers in the other direction must increase their transit time if bandwidths are to be preserved. Since distance between any pair of intersections remains constant, the sum of the reciprocals of progression speeds must remain constant. Inspection of Eq. 7 indicates that allowing v_e to increase without disturbing bandwidths results in higher permissible values of v_i and v_o.

If Eqs. 4 and 5 are substituted into Eq. 6, the result is

$$F = \frac{4B_i y_{sm}}{v_f}\left(1 - \frac{v_i}{v_f}\right) + \frac{4B_o y_{sm}}{v_f}\left(1 - \frac{v_o}{v_f}\right) \tag{8}$$

If F is minimized by the method of Lagrange multipliers subject to the constraint of Eq. 7, the optimum will occur when

$$\frac{\partial}{\partial v_i}\left(F + \mu\left[\frac{1}{v_i} + \frac{1}{v_o} - \frac{2}{v_e}\right]\right) = 0 \tag{9}$$

and

$$\frac{\partial}{\partial v_o}\left(F + \mu\left[\frac{1}{v_i} + \frac{1}{v_o} - \frac{2}{v_e}\right]\right) = 0 \tag{10}$$

where μ is the Lagrange multiplier.

The simultaneous solution of Eqs. 9 and 10 after substitution of Eq. 8 results in

$$\left(\frac{v_i}{v_o}\right)^2 = \frac{B_o}{B_i} \tag{11}$$

Taking the ratio of Eqs. 4 and 5 results in

$$\frac{y_{di}}{y_{do}} = \frac{B_i}{B_o}\frac{v_i}{v_o}\frac{(v_f - v_i)}{(v_f - v_o)} \tag{12}$$

Substituting Eq. 11 into Eq. 12 yields

$$\frac{y_{di}}{y_{do}} = \frac{v_o}{v_i}\frac{(v_f - v_i)}{(v_f - v_o)} = \frac{\left(\frac{v_f}{v_i} - 1\right)}{\left(\frac{v_f}{v_o} - 1\right)} \tag{13}$$

Simultaneous solution of Eqs. 7 and 13 results in

$$\frac{v_i}{v_f} = \frac{\left(1 + \frac{y_{do}}{y_{di}}\right)}{\frac{y_{do}}{y_{di}} + \frac{2v_f}{v_e} - 1} \tag{14}$$

$$\frac{v_o}{v_f} = \frac{\left(1 + \frac{y_{di}}{y_{do}}\right)}{\frac{y_{di}}{y_{do}} + \frac{2v_f}{v_e} - 1} \tag{15}$$

Thus, for a given level of demand, the progression speeds will be determined by the largest value of v_e for which the values of B_i and B_o, satisfying Eqs. 4 and 5, can be realized.

It has been shown (8) that the progression obtained for maximum equal bandwidths with equal progression speeds can be modified to allow B_i to be increased up to min $[2B, G]$, where B is the maximum equal bandwidth for the given progression speed and G is the minimum green. With B_i set in this range, the corresponding value for B_o is

$$B_o = \max\left[0, 2B - B_i\right]$$

Solving Eq. 4 for B_i and eliminating v_i using Eq. 14 results in

$$B_i = \frac{y_{di}}{4y_{sm}} \frac{\left[\frac{y_{do}}{y_{di}} + \frac{2v_f}{v_e} - 1\right]^2}{\left[1 + \frac{y_{di}}{y_{do}}\right]\left[\frac{2v_f}{v_e} - 2\right]} \tag{16}$$

A similar procedure using Eqs. 5 and 15 yields

$$B_o = \frac{y_{do}}{4y_{sm}} \frac{\left[\frac{y_{di}}{y_{do}} + \frac{2v_f}{v_e} - 1\right]^2}{\left[1 + \frac{y_{di}}{y_{do}}\right]\left[\frac{2v_f}{v_e} - 2\right]} \tag{17}$$

By using the method of Little et al (8), one can determine values of 2B as a function of v_e for the arterial geometry and specified traffic signal splits. The optimum value

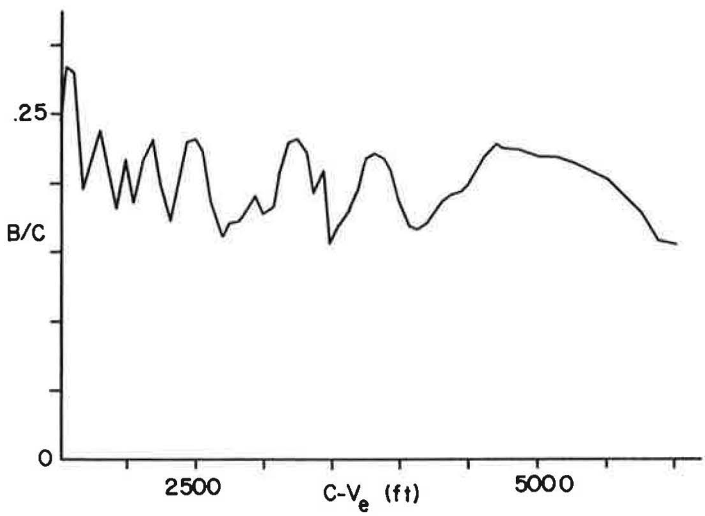

Figure 4. Realizable equal bandwidth as a function of progression speed for a section of Euclid Avenue in Cleveland, Ohio.

TABLE 1

OPTIMAL PROGRESSION CHARACTERISTICS FOR 10 INTERSECTION SECTIONS WITH VARIOUS DEMAND CONDITIONS

$\frac{y_{di}}{y_{sm}}$	$\frac{y_{do}}{y_{sm}}$	$\frac{v_e}{v_f}$	B_i	B_o	$\frac{v_i}{v_f}$	$\frac{v_o}{v_f}$
0.23	0.23	0.5	0.23	0.23	0.5	0.5
0.23	0.115	0.75	0.27	0.19	0.69	0.82
0.23	0.0767	0.75	0.26	0.15	0.67	0.86
0.23	0.0575	0.8	0.28	0.17	0.71	0.91
0.23	0.046	0.8	0.27	0.16	0.71	0.92

of v_e is the largest value of v_e for which this procedure indicates that 2B is greater than the sum of the values for B_i and B_o which are computed using Eqs. 16 and 17.

A plot of B as a function of the product of cycle length and v_e is shown in Figure 4 for a section of Euclid Avenue in Cleveland, Ohio. It can be seen that there is little bandwidth advantage obtained by using low values of v_e. Some examples of progression speeds and bandwidths which would be selected by the procedure outlined in this paper are given in Table 1.

Existing traffic responsive systems adjust the progression by modifying cycle length only. Changing cycle length reduces or increases the progression speeds in both directions by the same factor. This unduly penalizes the traffic in the low demand directions. The results (Table 1) indicate it is possible to give better service to the heavy demand direction in terms of a higher progression speed by taking account of the light demand.

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Accident Exposure and Intersection Safety for At-Grade, Unsignalized Intersections

VASANT H. SURTI, Department of Civil Engineering and Mechanics,
The Catholic University of America

The purpose of this study is to provide a method that will enable traffic and safety engineers to compare certain types of intersections relative to traffic safety. The method is based on determining an accident exposure index and could be utilized to identify the intersections that are prone to accidents. Four priority-type, unsignalized, at-grade intersections of varying geometrics were investigated. A fairly good correlation was found between the number of accidents and the accident exposure index. The index was calculated on the basis of the merging of the two traffic streams, and hence the single-vehicle accidents cannot be assumed to correlate with the index.

•BECAUSE urban transportation systems are planned with emphasis on auto travel, it becomes increasingly important to ensure the optimum operation of the system. However, as a chain is only as strong as its weakest link, so the overall operation of a highway system is highly dependent on the operations in critical sections. Intersections in a street system and interchanges in a freeway system can be regarded as the weak links. One very important component of the system is the at-grade intersection. It is important that the traffic engineer know which control is best for a given intersection condition. However, only meager information is available concerning controls below the level of traffic signals.

The purpose of this study is to enable traffic and safety engineers to compare certain types of unsignalized intersections relative to traffic safety. This method is based on determining an accident exposure index for selected intersections to identify those that are prone to accidents, especially during the daylight hours. The intersections investigated in this study are the priority type—the type of intersection where one of the intersecting streets is given a definite priority over other streets. The nonpriority or minor street for such an intersection is controlled by either a "stop" or "yield" sign. In this study, all minor street approaches are controlled by a "stop" sign.

Each driver would like to proceed as he pleases through a street network from his origin to his destination. Because his path crosses that of other vehicles at intersections in the system, however, it is desirable to minimize the chances that the potential intersection of vehicle paths will result in collisions.

STUDY SITES AND DATA COLLECTION PROCEDURE

Site Selection

After consultations with personnel of the Safety Section of the District of Columbia Department of Highways and Traffic, four intersections were selected for investigation. They were the intersections of 7th Street and Michigan Avenue N.E. (Fig. 1), 11th and P Streets N.W. (Fig. 2), 9th and K Streets N.W. (Fig. 3), and 12th and C Streets N.E.

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Figure 1. Intersection of 7th Street and Michigan Avenue N.E.

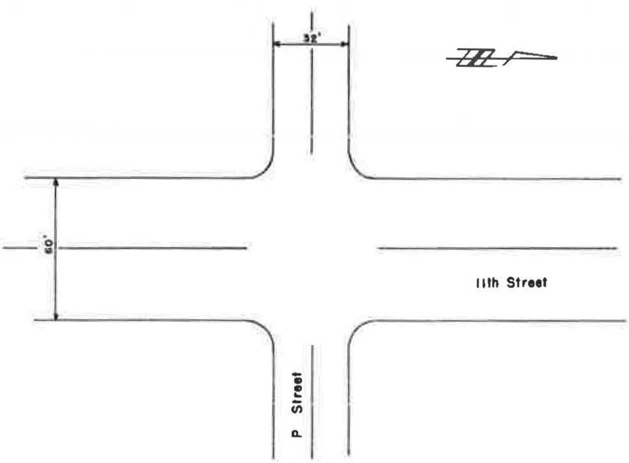

Figure 2. Intersection of 11th and P Streets N.W.

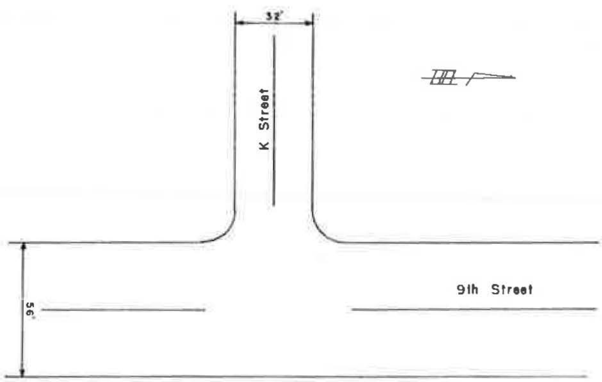

Figure 3. Intersection of 9th and K Streets N.W.

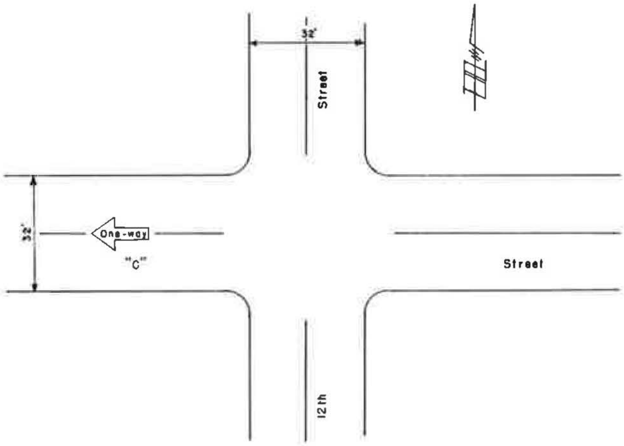

Figure 4. Intersection of 12th and C Streets N.E.

(Fig. 4). All were unsignalized, at-grade, priority-type intersections. A brief description of each intersection location is given in Table 1.

Data Collection for Peak-Hour Demand

One-hour traffic counts on a 5-minute basis were obtained for all approaches on each intersection study site during the afternoon peak period. Three recorders were found to be adequate for this purpose. Traffic counts for each approach were stratified into left turn, through, and right turn maneuvers.

SAFETY ANALYSIS

A collision between two moving vehicles can occur only when both vehicles attempt to occupy the same space at the same time. Thus, highway accidents can occur only under four conditions: head-on collision, rear-end collision, sideswiping, and crossing each other's travel path. Highway design can minimize or entirely eliminate all of the conditions under which such accidents occur. The degree of traffic congestion and the degree of hazard are determinant design factors, subject to considerations of construction and right-of-way cost, physical topography, and developments.

This investigation is primarily concerned with the identification of "accident prone" at-grade, unsignalized, priority-type intersections.

The term "accident prone", which has received wide use, has acquired many different definitions. Some traffic engineers prefer an absolute number of accidents occurring at a given location per year as a criterion, whereas others take into account traffic volumes with the resulting use of an index value such as accidents per million vehicles

TABLE 1

SUMMARY OF STUDY INTERSECTIONS

Intersection	Type	Major Street	Minor Street	Location Within Metropolitan Area
7th Street and Michigan Avenue N.E.	4-legged, two-way with two-way	Michigan Avenue	7th Street	Outlying business district
11th and P Streets N.W.	4-legged, two-way with two-way	11th Street	P Street	Outlying business district
9th and K Streets N.W.	T-type, two-way with two-way	9th Street	K Street	Fringe area
12th and C Streets N.E.	4-legged, C Street one-way, 12th Street two-way	C Street	12th Street	Outlying business district

per year at intersections or accidents per hundred million vehicle-miles per year on sections of open highway. There is a need for a standard criterion for an accident prone location and, perhaps more important, a method by which it can be determined.

Accident Exposure Index for a Four-Legged, At-Grade Intersection

In studying traffic movements at highway intersections, the conventional method of portraying the direction and density of vehicle traffic is by means of vectors in which an arrow indicates the direction of the flow and a variable bandwidth indicates the traffic density. The maximum number of traffic vectors at an intersection is equal to the product of the number of legs entering the intersection, n, times the same number minus one, n - 1; that is, n(n - 1). For example, a four-legged, unsignalized, at-grade intersection has 12 directional arrows and 24 potential collision points where two vehicles have a probability of colliding (Fig. 5). The term "potential collision point" is defined as the point of intersection of two different vehicle travel paths. A vehicle going from west to east has the possibility of colliding with any vehicle that is traveling through any of the potential collision points in its path during the time interval that this vehicle passes each of these points. During this critical time interval, this vehicle has the possibility of colliding with as many vehicles as cross potential collision points 1, 12, 11, 10, 19, and 20. The accident exposure for a car going from north to west will be equal to the number of vehicles passing potential collision point 24 during the same time interval. For several vehicles going from north to west, the accident exposure would equal the sum of the products of the number of cars going from north to west and the number of cars passing potential collision point 24 for a given time interval.

Earlier, Surti (1) developed a procedure to determine an exposure index using average daily traffic (ADT) volumes. Because the distribution of the traffic volumes for an

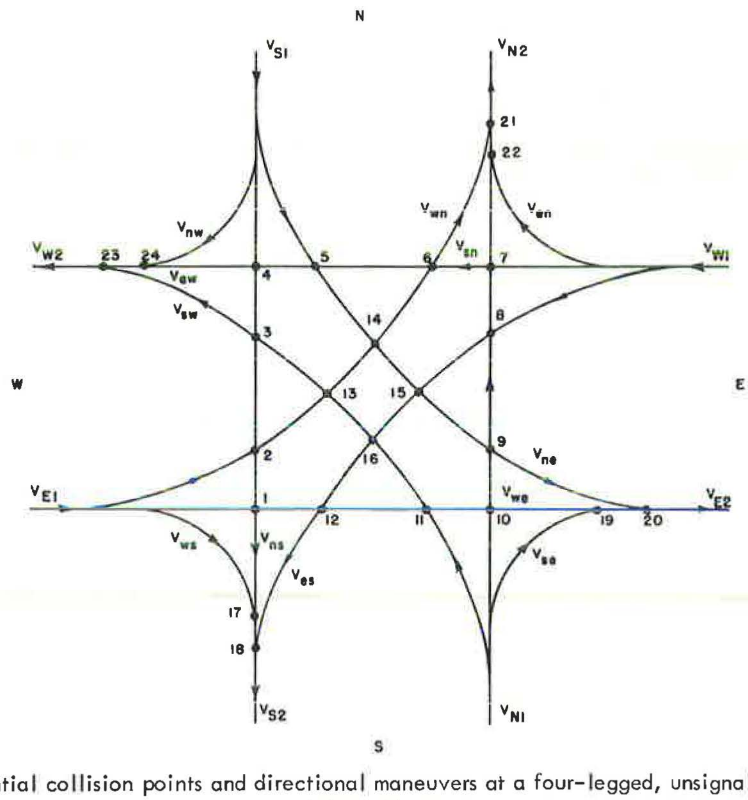

Figure 5. Potential collision points and directional maneuvers at a four-legged, unsignalized, at-grade intersection.

entire day is not uniform and more than half of the total trips are made during the peak periods, it would be more appropriate to use peak-hour volume to obtain the value for exposure index for a given intersection. Peak periods vary with the size of a city and the location of an intersection. For large metropolitan areas, the peak periods are generally considered from 7:00 to 9:30 a.m. for the inbound morning rush period and 4:00 to 6:30 p.m. for the outbound traffic. In this study, a method is developed for the determination of an accident exposure index for the priority-type, unsignalized, at-grade intersections, and an attempt is made to determine if there is any correlation between the accident exposure index and the number of accidents during the peak period.

In Figure 5, the peak-hour volumes (PHV) are shown as V_{ij}. The first character of the subscript represents initial direction and the second character represents final direction. The points at which the diverging maneuvers occur are not considered as potential collision points. First the exposure index for each of the potential collision points and then a general equation for the exposure index of the entire intersection are determined.

Exposure E_i at point 1 for a vehicle traveling from west to east for a given time interval i equals the number of vehicles southbound from the north direction during the same time interval i.

The exposure E_p for the entire peak period can be expressed as

$$E_p = \frac{PHV_{ns}\ (i)}{18{,}000}$$

where 18,000 is the number of seconds in one peak period (5 hours).

From this it follows that for the entire year

$$E_a = \frac{PHV_{ns}\ (i)\ 260}{18{,}000}$$

where 260 is the number of weekdays during 1 year.

The selection of the critical time period, i, could be arbitrary because it has only a relative value. To simplify calculations, about three-fourths of a second is assumed as a critical time. Substituting for i = 0.6923 second in the above expression, a simplified expression of the following form is obtained:

$$I = \frac{1}{100}\ PHV_{ns}$$

where I is the accident exposure index for each car traveling from west to east at potential collision point 1.

The total accident exposure index at the potential collision point 1 would be equal to

$$I_1 = \frac{1}{100}\ (PHV_{ns} \times PHV_{we})$$

or

$$100\ I_1 = PHV_{ns} \times PHV_{we}$$

It is assumed that the flows of traffic in the opposite directions are equal and there is a balanced traffic movement. At first this assumption seems unreasonable because there is unbalanced directional traffic volume during the morning and afternoon peak periods. However, when we consider the total time of the morning and afternoon peak periods, the total traffic in each direction is generally balanced out.

Let V_{E1}, V_{W1}, V_{N1}, and V_{S1} be the respective eastbound, westbound, northbound, and southbound peak-hour traffic volumes before crossing the intersection. Also let

V_{E2}, V_{W2}, V_{N2}, and V_{S2} be the respective eastbound, westbound, northbound, and southbound peak-hour traffic volumes after crossing the intersection. The peak-hour volume V_{E1} consists of vehicles on the eastbound approach leg that intend to make either through, left, or right turn movements. We can write this in the form of an equality expression. Thus we have the following expressions for V_{E1}, V_{W1}, V_{N1}, and V_{S1}:

Before crossing the intersection

$$V_{E1} = V_{wn} + V_{we} + V_{ws}$$

$$V_{W1} = V_{en} + V_{ew} + V_{es}$$

$$V_{N1} = V_{se} + V_{sn} + V_{sw}$$

$$V_{S1} = V_{nw} + V_{ne} + V_{ns}$$

The peak-hour traffic volume V_{E2} is made up of traffic approaching from north, west, and south after they have crossed the intersection. We can write the equality expressions for V_{E2}, V_{W2}, V_{N2}, and V_{S2} in a manner similar to the expressions for before crossing the intersection:

After crossing the intersection

$$V_{E2} = V_{ne} + V_{we} + V_{se}$$

$$V_{W2} = V_{nw} + V_{ew} + V_{sw}$$

$$V_{N2} = V_{en} + V_{wn} + V_{sn}$$

$$V_{S2} = V_{es} + V_{ws} + V_{ns}$$

Because we have assumed a balanced traffic movement and equal traffic volumes in the opposite directions, we have

$$V_{E1} = V_{W2}$$

$$V_{E2} = V_{W1}$$

$$V_{S1} = V_{N2}$$

$$V_{S2} = V_{N1}$$

and

$$V_{ns} = V_{sn} = a$$
$$V_{ew} = V_{we} = b$$
$$V_{es} = V_{se} = c$$

$$V_{wn} = V_{nw} = d$$
$$V_{sw} = V_{ws} = e$$
$$V_{ne} = V_{en} = f$$

The values of 100 times the accident exposure index of each of the 24 potential collision points are given in Table 2.

The exposure index for the entire intersection would equal the sum of the indexes of each of the 24 collision points:

$$100\ I_T = ab + ad + ae + ab + bf + bd + ab + ac + af + ab + be + bc + de + df + cf + ce + ae + ac + bc + bf + ad + af + be + db$$

$$= 4\ ab + 2\ ad + 2\ ae + 2\ bf + 2\ bd + 2\ ac + 2\ af + 2\ be + 2\ bc + de + df + ce + cf$$

Rearranging and adding and subtracting 2 ab in the above expression gives

$$100\ I_T = 2\ ab + 2\ ad + 2\ ae + 2\ ab + bf + 2\ bd + 2\ ab + 2\ ac + 2\ af - 2\ ab + 2\ be + 2\ bc + de + df + ce + cf$$
$$= 2\ a\ (b + d + 3) + 2\ b\ (a + f + d) + 2\ a\ (b + c + f) + 2\ b(c + e - a) + (c + d)\ (e + f)$$

Adding and subtracting 2 a to the fourth term in parentheses and by substitution gives

$$100\ I_T = 2\ a\ (V_{E1}) + 2\ b\ (V_{S1}) + 2\ a\ (V_{W1}) + 2\ b\ (c + e + a - 2\ a) + (V_{es} + V_{wn})(V_{sw} + V_{ne})$$
$$= 2\ a\ (V_{E1} + V_{W1}) + 2\ b\ (V_{N1} + V_{S1} - 2\ V_{ns}) + (V_{es} + V_{wn})(V_{sw} + V_{ne})$$

$$100\ I_T = 2\ V_{ns}\ (V_{E1} + V_{ns}) + 2\ V_{ew}\ (V_{N1} + V_{S1} - 2\ V_{ns}) + (V_{es} + V_{wn})(V_{sw} + V_{ne})$$

In the above equation the appropriate volumes in the opposite direction could be used (e.g., $V_{ns} = V_{sn}$, $V_{ne} = V_{en}$).

TABLE 2

ACCIDENT EXPOSURE INDEX OF EACH OF THE 24 POTENTIAL COLLISION POINTS FOR A FOUR-LEGGED, AT-GRADE INTERSECTION

Potential Collision Point Number	100 I	100 I for Balanced Movement	Potential Collision Point Number	100 I	100 I for Balanced Movement
1	$V_{we} \times V_{ns}$	a × b	13	$V_{wn} \times V_{sw}$	d × e
2	$V_{wn} \times V_{ns}$	d × a	14	$V_{ne} \times V_{wn}$	f × d
3	$V_{sw} \times V_{ns}$	e × a	15	$V_{es} \times V_{ne}$	c × f
4	$V_{ew} \times V_{ns}$	b × a	16	$V_{es} \times V_{sw}$	c × e
5	$V_{ew} \times V_{ne}$	b × f	17	$V_{ns} \times V_{ws}$	a × e
6	$V_{wn} \times V_{ew}$	b × b	18	$V_{ns} \times V_{es}$	a × c
7	$V_{ew} \times V_{sn}$	b × a	19	$V_{we} \times V_{se}$	b × c
8	$V_{sn} \times V_{es}$	a × c	20	$V_{we} \times V_{ne}$	b × f
9	$V_{sn} \times V_{ne}$	a × f	21	$V_{sn} \times V_{wn}$	a × d
10	$V_{sn} \times V_{we}$	a × b	22	$V_{sn} \times V_{en}$	a × f
11	$V_{we} \times V_{sw}$	b × e	23	$V_{ew} \times V_{sw}$	b × e
12	$V_{we} \times V_{es}$	b × c	24	$V_{ew} \times V_{nw}$	b × d

TABLE 3

PEAK-HOUR APPROACH AND DIRECTIONAL TRAFFIC VOLUMES FOR THE 7TH STREET AND MICHIGAN AVENUE N.E. INTERSECTION

Approach Volume	Directional Volume
$V_{E1} = 965$	$V_{ns} = 19$
$V_{W1} = 485$	$V_{ew} = 400$
$V_{N1} = 179$	$V_{es} = 50$
$V_{S1} = 101$	$V_{wn} = 8$
	$V_{sw} = 6$
	$V_{ne} = 62$

TABLE 4

PEAK-HOUR APPROACH AND DIRECTIONAL TRAFFIC VOLUMES FOR THE 11TH AND P STREETS N.W. INTERSECTION

Approach Volume	Directional Volume
$V_{E1} = 126$	$V_{ns} = 489$
$V_{W1} = 131$	$V_{ew} = 74$
$V_{N1} = 800$	$V_{es} = 14$
$V_{S1} = 539$	$V_{wn} = 2$
	$V_{sw} = 36$
	$V_{ne} = 40$

Research in this field has been undertaken by May (2), Grossman (3), and Breuning and Bone (4). May presented a statistical method of determining an accident prone location by comparing accident rates with the traffic volumes for a series of intersections. Using this information, he computed a regression line. The accident rates are defined as accident rates per million vehicles per year at intersections. An accident prone location can be assumed to exist if its rate is greater than one standard deviation above the regression line.

Grossman developed a numerical index for the accident exposure index. He employed 24-hour traffic counts and used the sums of the crossing movements.

Breuning and Bone also used average daily traffic volumes and developed a procedure for determining the accident exposure index for freeway interchanges. This index is a dimensionless quantity, and has a built-in weighted effect for all directional maneuvers. It not only can identify the intersections that are accident prone, but also can prove useful in comparison of intersections before and after improvements. A similar approach can be used to determine exposure indexes for T-type, one-way with one-way, one-way with two-way, and other types of intersections.

Index for 7th Street and Michigan Avenue Intersection—Michigan Avenue and 7th Street N.E. form essentially a four-legged intersection, because the traffic generated by the Bunker Hill Road approach is insignificant compared to the remaining four approaches. The peak-hour volume for Bunker Hill Road was less than 10 vehicles and hence it can be ignored. We can use the formula developed for the four-legged intersection to determine the peak-period accident exposure index value for this intersection. Table 3 gives the necessary peak-hour traffic volumes.

The accident exposure index expression for a four-legged intersection as developed earlier is as follows:

$$100\ I_T = 2\ V_{ns}\ (V_{E1} + V_{W1}) + 2\ V_{ew}\ (V_{N1} + V_{S1} - V_{ns}) + (V_{es} + V_{wn})(V_{sw} + V_{ne})$$

By substitution of the appropriate values from Table 3, we have

$$100\ I_T = 2\,(19)(965 + 485) + 2\,(400)\ [179 + 101 - (2 \times 19)] + (50 + 8)(6 + 42)$$

$$100\ I_T = 38\,(1450) + 800\,(280 - 38) + (58)(68)$$

$$100\ I_T = 55,100 + 193,600 + 3,944$$

$$I_T = \frac{252,644}{100}$$

$$I_T = 2526.44$$

The peak-period accident exposure index for 7th Street and Michigan Avenue N.E. is thus 2526.44.

Index for 11th and P Streets Intersection—The intersection of 11th and P Streets N.W. also falls in the category of a four-legged intersection and we can make use of the same expression to determine the peak-period accident exposure index. The traffic volumes necessary to compute the peak-period accident exposure index are given in Table 4. Again using the expression for the peak-period accident exposure index we have

$$100\ I_T = 2\ V_{ns}\ (V_{E1} + V_{W1}) + 2\ V_{ew}\ (V_{N1} + V_{S1} - 2\ V_{ns}) + (V_{es} + V_{wn})(V_{sw} + V_{ne})$$

Substituting the appropriate traffic volumes from Table 4, we have

$$100\ I_T = 2\ (489)(126 + 131) + 2\,(74)\ [800 + 539 - 2\ (489)] + (14 + 2)(36 + 40)$$

$$100\ I_T = 2\ (489)(257) + 2\ (74)(361) + 16\ (76)$$

$$100\ I_T = 251{,}346 + 53{,}428 + 1{,}216$$

$$100\ I_T = 305{,}990$$

$$I_T = 3059.90$$

The peak period accident exposure index for the 11th and P Streets N.W. intersection is thus 3059.90.

Accident Exposure Index for a T-Type, Unsignalized, At-Grade Intersection

The potential collision points and directional maneuvers for a typical "T" intersection are shown in Figure 6. It has six potential collision points and six directional

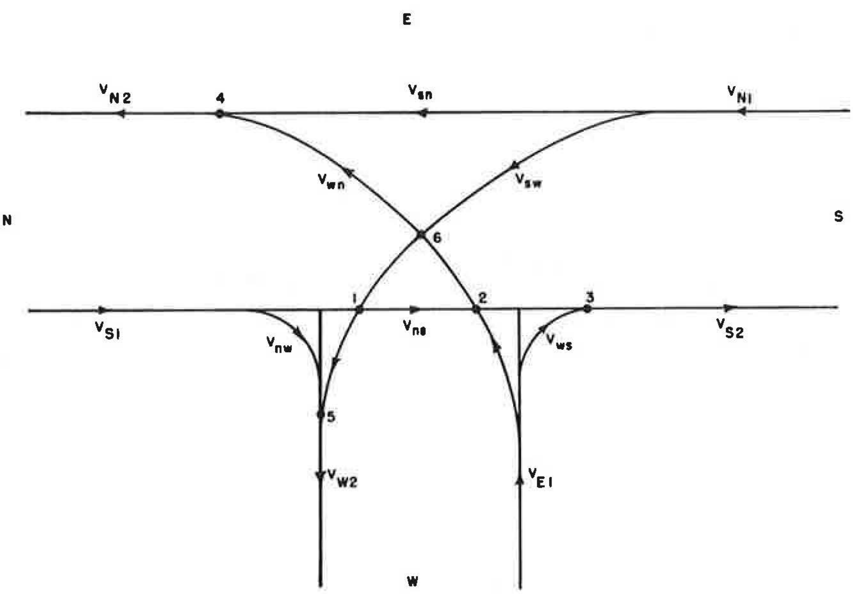

Figure 6. Potential collision points and directional maneuvers at a T-type, unsignalized, at-grade intersection.

maneuvers. Again we use the same terminology developed in the determination of peak-period accident exposure index for the four-legged intersection. The directional peak-hour traffic volumes are V_{ns}, V_{sn}, V_{nw}, V_{ws}, V_{sw}, and V_{wn}. Peak-hour approach traffic volumes before crossing the intersection are V_{E1}, V_{N1}, and V_{S1}; after crossing the intersection are V_{S2}, V_{N2}, and V_{W2}.

The total peak-period accident exposure index at potential collision point 1 is

TABLE 5

PEAK-PERIOD ACCIDENT EXPOSURE INDEX OF EACH OF THE SIX POTENTIAL COLLISION POINTS FOR A T-TYPE, UNSIGNALIZED, AT-GRADE INTERSECTION

Potential Collision Point Number	100 I	100 I for Balanced Movement
1	$V_{sw} \times V_{ns}$	a × b
2	$V_{ns} \times V_{wn}$	b × c
3	$V_{ws} \times V_{ns}$	a × b
4	$V_{sn} \times V_{wn}$	b × c
5	$V_{sw} \times V_{nw}$	a × c
6	$V_{sw} \times V_{wn}$	a × c

$$I_1 = \frac{1}{100} (V_{ns} \times V_{sw})$$

or

$$100\ I_1 = V_{ns} \times V_{sw}$$

Again assuming equal traffic flows in opposite directions and balanced traffic movement, we have $V_{E1} = V_{W2}$, $V_{S1} = V_{N2}$, $V_{N1} = V_{S2}$, $V_{sw} = V_{ws}$, $V_{ns} = V_{sn}$, and $V_{wn} = V_{nw}$. The expression for peak-hour approach traffic volumes are as follows:

Before crossing the intersection

$$V_{E1} = V_{ws} + V_{wn}$$

$$V_{S1} = V_{nw} + V_{ns}$$

$$V_{N1} = V_{sw} + V_{sn}$$

After crossing the intersection

$$V_{N2} = V_{sn} + V_{wn}$$

$$V_{W2} = V_{nw} + V_{sw}$$

$$V_{S2} = V_{ws} + V_{ns}$$

Let $V_{sw} = V_{ws} = a$; $V_{ns} = V_{sn} = b$; and $V_{wn} = V_{nw} = c$.

The peak-period accident exposure index for the entire intersection is equal to the sum of the indexes of each of the six potential collision points. The index for each of the six potential collision points is given in Table 5.

The total peak-period accident exposure index for the intersection can now be written as

$$\begin{aligned} 100\ I_T &= ab + bc + ab + bc + ac + ac \\ &= 2\ ab + 2\ bc + 2\ ac \\ &= 2\ a\ (b + c) + 2\ bc \\ &= 2\ a\ (V_{S1}) + 2\ bc \\ 100\ I_T &= 2\ V_{ws}\ (V_{S1}) + 2\ V_{ns}\ V_{nw} \\ 50\ I_T &= V_{ws}\ V_{S1} + V_{ns}\ V_{nw} \end{aligned}$$

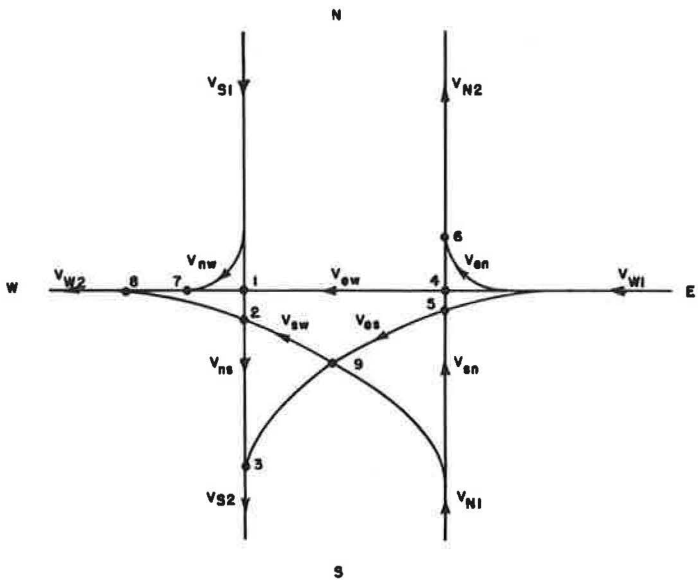

Figure 7. Potential collision points and directional maneuvers at a four-legged, one-way with two-way, unsignalized, at-grade intersection.

Index for 9th and K Streets Intersection—The intersection at 9th Street and K Street N.W. is a T-type, unsignalized, at-grade intersection and the expression just derived can be used to determine the peak-period accident exposure index:

$$50\ I_T = V_{ws}\ V_{S1} + V_{ns}\ V_{nw}$$

Utilizing the required peak-hour approach and directional traffic volumes from the data for this intersection, we have $V_{S1} = 970$, $V_{ws} = 255$, $V_{ns} = 710$, and $V_{nw} = 260$. By substitution in the expression, we have

$$\begin{aligned} 50\ I_T &= V_{ws}\ V_{S1} + V_{ns}\ V_{nw} \\ &= 255\,(970) + 710\,(260) \\ &= 247{,}350 + 184{,}600 \\ 50\ I_T &= 431{,}950 \\ I_T &= 8639.0 \end{aligned}$$

The peak-period accident exposure index for the 9th and K Streets intersection is thus 8639.0.

Accident Exposure Index for a Four-Legged, One-Way With Two-Way, Unsignalized, At-Grade Intersection

The potential collision points and directional maneuvers for a typical one-way with two-way intersection are shown in Figure 7. It has nine potential collision points and seven directional maneuvers. It is interesting to note that just by having one street one-way for a four-legged intersection, the number of potential collision points is reduced from 24 to 9.

The intersection shown in Figure 7 is one-way in the westbound direction and two-way in the north-south direction. The peak-hour directional traffic volumes are V_{ns},

V_{sn}, V_{ew}, V_{en}, V_{nw}, and V_{es}. Peak-hour approach traffic volumes before crossing the intersection are V_{W1}, V_{N1}, and V_{S1}; after crossing the intersection they are V_{W2}, V_{N2}, and V_{S2}.

The total peak-period accident exposure index for this intersection at potential collision point 1 is

$$I_1 = \frac{1}{100} (V_{ew} \times V_{ns})$$

or

$$100\ I_T = V_{ew} \times V_{ns}$$

TABLE 6

PEAK-PERIOD ACCIDENT EXPOSURE INDEX OF EACH OF THE NINE POTENTIAL COLLISION POINTS FOR A FOUR-LEGGED, ONE-WAY WITH TWO-WAY, UNSIGNALIZED, AT-GRADE INTERSECTION

Potential Collision Point Number	100 I	Potential Collision Point Number	100 I
1	$V_{ew} \times V_{ns}$	6	$V_{en} \times V_{sn}$
2	$V_{sw} \times V_{ns}$	7	$V_{nw} \times V_{ew}$
3	$V_{es} \times V_{ns}$	8	$V_{sw} \times V_{ew}$
4	$V_{ew} \times V_{sn}$	9	$V_{sw} \times V_{es}$
5	$V_{ew} \times V_{sn}$		

Assuming, as before, equal traffic flows in the opposite directions and balanced traffic movement, we have $V_{N1} = V_{S2}$, $V_{S1} = V_{N2}$, and $V_{sn} = V_{ns}$.

The expressions for peak-hour approach traffic volumes are as follows:

Before crossing the intersection

$$V_{N1} = V_{en} + V_{ew} + V_{es}$$

$$V_{N1} = V_{sn} + V_{sw}$$

$$V_{S1} = V_{nw} + V_{ns}$$

After crossing the intersection

$$V_{W2} = V_{nw} + V_{sw} + V_{ew}$$

$$V_{N2} = V_{en} + V_{sn}$$

$$V_{S2} = V_{es} + V_{ns}$$

The peak-period accident exposure index for each of the nine potential collision points is given in Table 6.

The total peak-period accident exposure index for the intersection is equal to the sum of the indexes of each of the nine potential collision points:

$$\begin{aligned} 100\ I_T = {} & (V_{ew} V_{ns}) + (V_{sw} V_{ns}) + (V_{es} V_{ns}) \\ & + (V_{ew} V_{sn}) + (V_{es} V_{sn}) + (V_{en} V_{sn}) \\ & + (V_{nw} V_{ew}) + (V_{sw} V_{ew}) + (V_{sw} V_{es}) \end{aligned}$$

Because we have assumed $V_{ns} = V_{sn}$

$$\begin{aligned} 100\ I_T = {} & V_{ns} (V_{ew} + V_{es} + V_{en}) + V_{ew} (V_{ns} + V_{nw}) + V_{ns} (V_{sw} + V_{es}) \\ & + V_{sw} (V_{ew} + V_{es}) \end{aligned}$$

$$100\ I_T = V_{ns} (V_{W1}) + V_{ew} (V_{N1}) + V_{ns} (V_{sw} + V_{es}) + V_{sw} (V_{ew} + V_{es})$$

TABLE 7

A SUMMARY OF ACCIDENTS AND PEAK-PERIOD ACCIDENT EXPOSURE INDEXES FOR THE SELECTED INTERSECTIONS

Intersection	Type of Intersection	Peak-Period Exposure Index	Number of Accidents, Daylight Hours, 1966-1967
7th Street and Michigan Avenue N.E.	Four-legged, two-way with two-way, minor street approaches offset by about 100 ft	2526.44	3
11th and P Streets N.W.	Four-legged, two-way with two-way	3059.90	10
9th and K Streets N.W.	T-type, all approaches two-way	8639.00	25
12th and C Streets N.E.	Four-legged, C Street one-way westbound, 12th Street two-way	1344.67	2

Index for 12th and C Streets Intersection—The intersection at 12th Street and C Street N.E. falls in the category of a four-legged, one-way with two-way, unsignalized, at-grade type of intersection. Using the expression for the peak-period accident exposure index just derived, we have

$$100\ I_T = V_{ns}\ (V_{W1}) + V_{ew}\ (V_{N1}) + V_{ns}\ (V_{sw} + V_{es}) + V_{sw}\ (V_{ew} + V_{es})$$

Using the peak-hour approach and directional volumes for this intersection, we have $V_{N1} = 85$, $V_{W1} = 810$, $V_{ns} = 69$, $V_{ew} = 739$, $V_{sw} = 18$, and $V_{es} = 14$. By substitution,

$$\begin{aligned} 100\ I_T &= 69\ (810) + 739(85) + 69\ (18 + 14) + 18\ (739 + 14) \\ &= 55{,}890 + 62{,}815 + 2{,}208 + 13{,}554 \\ &= 134{,}467 \\ I_T &= 1344.67 \end{aligned}$$

The peak-period accident exposure index for the 12th Street and C Street N.E. intersection is thus 1344.67.

EVALUATION

The accident information for the selected intersections was obtained from the files of the District of Columbia Department of Highways and Traffic. Only accidents during daylight hours (6:00 a.m. to 6:00 p.m.) for a 2-year period (1966-1967) on the selected intersections were considered.

Table 7 gives the accident information and the peak-period accident exposure index for the four selected intersections. It is observed that there is a fairly good correlation between the number of accidents during the daylight hours and the peak-period accident exposure index. Based on this exposure index, accident rates for intersections become easily comparable and allow a direct analysis of the safety of each intersection design.

It must be remembered, however, that the exposure index is calculated on the basis of accidents caused by the merging of the two traffic streams. Single-vehicle accidents cannot be assumed proportional to the product of the two traffic streams. There are also complications of driver behavior under various traffic and emotional circumstances. Accidents of this type certainly will not correlate with the exposure index.

The peak-period accident exposure index is only a relative value and is free of any units, similar to the Reynold's number in hydraulics, a measure of turbulence in the flow of a fluid. Similarly, the exposure index may be said to measure traffic friction at an intersection.

CONCLUSIONS

This investigation has shown that the peak-period accident exposure index is a reliable means of evaluating the "accident proneness" of an intersection. A fairly good correlation was found between the derived exposure index and the accident experience of the selected intersections during daylight hours.

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Discussion

C. BLASE McCARTHY, Tulane University—While there is certainly a need for better techniques of measuring the accident potential or exposure of at-grade intersections, the supporting data in the subject paper do not appear to justify the conclusions that the proposed accident exposure index is a reliable means of evaluating the "accident proneness" of an intersection.

The suggested measure can be faulted on two major counts. One, the index is advanced as a measure of the total accident potential of an intersection without differentiating between classes of accidents or types of vehicle conflicts. Thus the proposed index does not seem to offer any significant advantage over existing techniques. A more useful index would be one that measures the hazard of various types of vehicle conflicts independently, such as the crossing conflict or the merging conflict. An index of this form would allow more detailed analysis of the intersection hazard and would allow remedial measures to be directed to the more dangerous maneuvers. For example, the elimination of left turns might reduce one type of conflict, but the corresponding increase in straight-through traffic might increase the total intersection accident potential if straight-through traffic is basically more hazardous than turning traffic.

Another objection to the format of the proposed index is with regard to the utilization of afternoon peak-hour traffic counts as a volume measure. The assumption that the morning peak-hour traffic balances the p.m. peak is absurd when applied to intersections containing one-way streets. Study Site No. 4, the intersection of 12th and C Streets N.E., is an example of this situation.

With regard to the data presented to substantiate the use of this exposure index, the following comments are in order.

At Site No. 1, the offset of 7th Street from the entrance to Catholic University created two additional conflict points between the left turn traffic from these cross streets.

At Site No. 2, 11th Street is 60 ft in width, which indicates two moving lanes in each direction. For multilane streets, it would seem reasonable to use lane volumes inasmuch as there are more potential conflict locations with smaller numbers of potential conflicts at each location. For example, a right turn from a minor street will merge only with the outside lane of the major street.

At Site No. 3, 9th Street is 56 ft in width and probably has four moving lanes.

A major objection to the study is that all four study sites are of distinctly different geometric characteristics. Personal studies of much larger samples of geometrically similar intersections have shown wide variations in accident exposure even for similar traffic flows. The suggested correlation cannot be accepted in view of the exceedingly small sample size and variations in sample characteristics.

VASANT H. SURTI, Closure—Prof. McCarthy's discussion is quite beneficial to the clarification and usefulness of the method proposed by the author.

It is emphasized that the accident exposure index developed in the study is not meant to be a reliable means of evaluating the "accident proneness" of an intersection, but rather an initial step or approach toward finding a better accident determinant, because the simple fact is that accidents are often caused by the unpredictable driver.

The method proposed by McCarthy is hypothetical at this time, and the author would like to encourage him to conduct a research study.

The index is not intended to be a measure of the total accident potential of an intersection, but rather only of certain types of accidents (involving merging or turning movements) that can be directly attributable to a traffic control deficiency (e.g., the accident exposure index does not include single-vehicle, pedestrian, nighttime, and rear-end accidents).

With regard to the use of afternoon volume counts as a volume measure, the first Surti study ADT figures were used and the correlation between afternoon peak volumes and off-peak volumes was found to be acceptable. Because peak volumes represent the most critical conditions, they were used.

Time and funds permitted a sample of only four intersections; however, the results of research conducted by the New Jersey Department of Transportation (6) are worth noting. This study analyzed several intersections relative to safety using multiple regression and the accident exposure index techniques. The results were encouraging in favor of employing the accident exposure index technique as a relative measure of intersection safety.

Reference

6. Reilly, E. F. Intersection Design. New Jersey Department of Transportation. A paper presented at the 47th Annual Meeting of the Highway Research Board, January 1968.