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5 Reports

Subject Areas

52 Road User Characteristics
53 Traffic Control and Operations
54 Traffic Flow
55 Traffic Measurements

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FOREWORD

The 5 authors of the papers in this RECORD report on some rather practical investigations of study techniques and also on measurements of the quality of service aspects of incidents and of control techniques. The information in the papers should be of interest to researchers and flow theorists as well as to a broad range of highway engineering specialists.

In the opening paper, Pahl points to a possibility for error to occur in the measurement of travel times by the use of pulse counts from a pulse generator because of end effects in the counting technique. Using a specially developed computer program to simulate data, the researcher made a statistical analysis to quantify errors and special effects introduced by this measurement technique on distributions of speeds, distributions of relative speeds, and space parameters computed from the data. The study indicates that the effects of the measurement technique can conditionally be ignored for speed distributions, but they cannot be made negligible for relative speed distributions.

Three methods for obtaining flow-concentration-speed relationships were selected for an experimental validation study by Gafarian, Lawrence, Munjal, and Pahl. Data grouped into short time slices, computation of a virtual concentration for each car, and classification of cars by their speeds were each compared with a standard based on isolation of periods of constant traffic flow. They found substantial agreement with methods based on short time slices, speed classes, and constant flow intervals, but not with the method based on virtual concentration. They conclude that the latter is not a valid method for obtaining flow-concentration-speed relationships. Three thoughtful discussions of this validation study extend and comment on the authors' findings.

Gordon reports on his study of the reactions of a number of drivers who were impeded by a slowly moving car (plant) on a single-lane road. Three different modes of response were observed, suggesting that the driver should be regarded as a strategist who continually adjusts his actions to fit his travel purpose and the conditions he faces.

Studying over 1,100 accidents and over 1,100 stalled vehicles on a $6\frac{1}{2}$-mile segment of the Gulf Freeway in Houston led Goolsby to the quantification of their impacts on freeway operations. A normal 3-lane peak directional flow of 5,560 vph was reduced to 2,750 vph with 1 lane blocked by accident and to 4,030 vph with an accident on the freeway shoulder. Other effects are reported, including total vehicle-hours of delay for various stalled vehicle or accident incidents.

In the final paper, diamond interchange signalization is analyzed by Munjal. The author identifies all the basic elements that comprise a phasing pattern, describes each by a set of parameters corresponding to signal controller adjustments, and then develops and synthesizes the various possible phasing patterns. Applicability of these phasing concepts to real-time control is also discussed.

CONTENTS

THE EFFECT OF DISCRETE TIME MEASUREMENT ON SPEED DATA

Juergen Pahl, University of California, Los Angeles

Time series measurements of vehicle speeds are often performed by measuring the travel time of each vehicle between 2 given points in units of pulse counts from a pulse generator of known frequency. The travel time is then measured in discrete equidistant values, but 2 discrete values are possible for each actual travel time because of end effects in the counting technique. Such a measurement technique effects a transformation of the continuous spectrum of actual vehicle speeds into a discrete spectrum of observed speed values. Because speed is inversely proportional to the measured time, the separation of the discrete speed values increases hyperbolically with increasing speeds. A computer program was developed to simulate such a measurement technique. The data simulated by this program were used for a statistical analysis to quantify the errors and special effects of such a measurement technique on selected distributions of speeds, distributions of relative speeds, and space parameters computed from the time series data. Among the results of this analysis are that the effects of the measurement technique can generally be ignored for speed distributions, if a certain set of intervals is used for classifying the speed values, but they cannot be made negligible for relative speed distributions.

•TIME SERIES measurements of vehicle speeds at a given highway location are often performed by measuring the travel time of each vehicle between 2 points that are a small, accurately known distance apart. The travel time is usually measured by electronically counting the number of pulses from a continuously running pulse generator of known frequency. The time is thus measured in discrete, equidistant values, but 2 such discrete values are possible for each actual travel time because of end effects in the pulse-counting technique. Because speed is inversely proportional to the measured time, the separation of the resulting discrete speed values increases hyperbolically for increasing speed values. Such a measurement technique effects a transformation of the continuous spectrum of actual vehicle speeds into a discrete spectrum of observed speeds with certain probabilities for the assignment of discrete values to each actual value.

Because the observed speed values are used in many inferences about traffic flow characteristics, it is important to know the effects of such measurement techniques on speed data. To quantify these effects, one would ideally like to compare known actual speed distributions for a highway with the corresponding speed distributions that are measured on the highway by this technique. In reality, however, only measured speed distributions are available. Unfortunately, it is not possible to reconstruct accurately the actual from the observed speed distributions; therefore, the use of real observed data is insufficient for such a study. To circumvent this problem, refuge was taken in the use of a simulation technique.

A computer program was developed to simulate the measurement technique, and a statistical analysis was performed to quantify the errors and special effects introduced by this measurement technique into speed data such as the distribution of speeds, the

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distribution of relative speeds, and space parameters computed from the time series data. Because not all the various implementations in practice of such a measurement technique could be anticipated, this study was performed by using the traffic analyzer of the Federal Highway Administration as a typical example. Even though the results from studying this example may not apply quantitatively to some other implementations of such a measurement technique, their qualitative features apply in any case.

DESCRIPTION OF THE MEASUREMENT TECHNIQUE

A typical setup for the measurement of vehicle speeds is the so-called speed trap that consists of 2 vehicle detectors positioned a distance L apart in the same lane. As a vehicle passes the first detector, it activates a counter that then starts to record the number of pulses from a continuously running generator of square wave pulses. When the vehicle passes the second detector, the counter is deactivated, and the accumulated number of pulses is recorded as the so-called speed code. The speed code is thus a measure of the travel time of a vehicle through the speed trap of length L. Any fraction of a square wave pulse that may be created at the time of the counter activation or deactivation is counted as a full pulse. As a consequence, the speed code observed for a given vehicle travel time varies as a function of the relative timing between the moment of counter activation activation and the sequence of square wave pulses.

This will be demonstrated for the case of the traffic analyzer, which generates square wave pulses with a time period of 0.01 sec and a duty cycle of 50 percent, i.e., the time length of each pulse equals one-half of the time period. If, for example, the travel time of a vehicle through the speed trap happens to equal exactly 2 times the time period of the square wave pulse, speed codes 2 and 3 are possible (Fig. 1). Assuming that the activation points occur at random, codes 2 and 3 are observed with probability of 0.5 each. If the travel time equals exactly 2.5 times the period of square wave pulses, the probability of observing a speed code 2 is 0, but it is 1 for observing a speed code of 3.

For any value of actual travel time, Figure 2 shows the probability of obtaining a particular speed code. Figure 2 applies, however, only to cases in which the arrival times of successive vehicles are independent of each other. For the case of dependent arrival times of successive vehicles, a dependency would be introduced into the speed codes for each vehicle, because the first pulse from each successive vehicle would exhibit a certain predictable timing with respect to the series of generated square wave pulses.

In real traffic, of course, time headways are dependent because of the interaction between vehicles that are traveling relatively close to each other in the same lane. Such a dependency is of the order of seconds (2). In order to obtain a dependency of observed speed codes for successive vehicles, however, the dependency between successive arrival times has to be of the order of 0.01 sec. A dependency in the order of seconds does not introduce any dependency

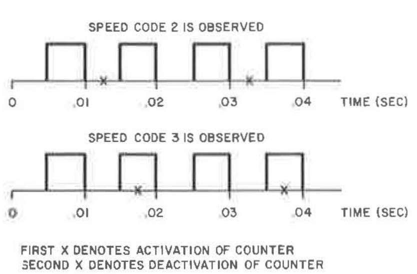

Figure 1. Demonstration example. (For a given car travel time of 0.20 sec through the speed trap, 2 different speed codes are possible depending on the relative timing between the square wave pulses and the pulses from the speed trap.)

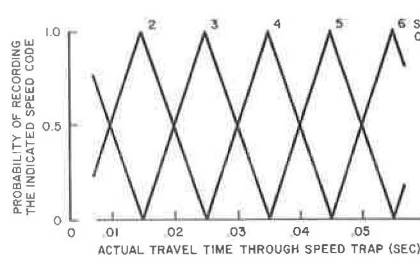

Figure 2. Probability function of observing the indicated speed codes.

in the digits of the order of 0.01 sec in the time headway. With respect to the square wave pulses generated in the traffic analyzer, arrival times can, therefore, be considered completely independent of each other. Then, Figure 2 shows the probability of observing a certain speed code for each vehicle on the highway.

Figure 2 shows that, for a recorded speed code U, the actual travel time of the vehicle through the speed trap must have been in the range from $[U - (3/2)]\ 0.01$ sec to $[U + (1/2)]\ 0.01$ sec. The midpoint of this interval is $[U - (1/2)]\ 0.01$ sec, and the corresponding vehicle speed is

$$v = \frac{L}{[U - (1/2)]\ 0.01\ \text{sec}} \tag{1}$$

The speeds computed from consecutive speed codes by Eq. 1 form a series of discrete values. The separation of these speed values increases hyperbolically with increasing values of speed.

To permit a study of the effects of this measurement technique on speed data, a computer program was developed that simulates this measurement technique. This program samples actual speeds from a specified speed probability density. For each actual speed value, the actual travel time through the speed trap is computed, and a speed code is found with the probabilities shown in Figure 1. Equation 1 is then used to compute the value of the observed speed. A detailed description of this computer program is presented in another report (1).

THE EFFECT OF THE MEASUREMENT TECHNIQUE ON SPEED DISTRIBUTIONS

In the following, the effect of the measurement technique on realistic distributions of vehicle speeds on a highway is studied. It has been shown for a highway with 2 lanes one-way (2) that the measured speed distributions can be approximated quite well by normal distributions and that the standard deviations of these distributions are small relative to the mean. The effect of the speed transformation was, therefore, studied for such normal speed distributions and subsequently for relative speed distributions and space parameters derived from these normal speed distributions.

Comparison of Actual and Observed Distributions

From the measured speed distributions (2) the distribution N(52.3; 22.0) for the fast lane and the case of light traffic has been selected for this study, where N(x;y) is a normal probability density of mean x and variance y. In order to study the effect of the measurement technique as a function of different variances, the distribution N(50.0; 49.0) was also selected for this study. This distribution approximates the speed distribution measured for vehicles in lanes 1 and 2.

From each of these normal distributions, 100,000 statistically independent speed values were sampled by the first part of the computer program developed for this analysis. These values were designated as actual speeds. The second part of this computer program then transformed these speeds into the observed speeds by simulating the effect of the traffic analyzer.

For each of these normal distributions, the actual speeds thus obtained were classified into speed intervals, and the actual speed distributions

$$s_i = m_i/MI_i \qquad i = 1, 2, 3, \ldots \tag{2}$$

were formed, where m_i is the number of actual speed values in the ith speed interval of length I_i, and M is the total number of generated speeds, which is 100,000 in all cases. Division by the interval length I_i normalizes the histogram area to 1. The observed speed distributions, s_i, i = 1, 2, 3, ..., were formed in analogy to Eq. 2.

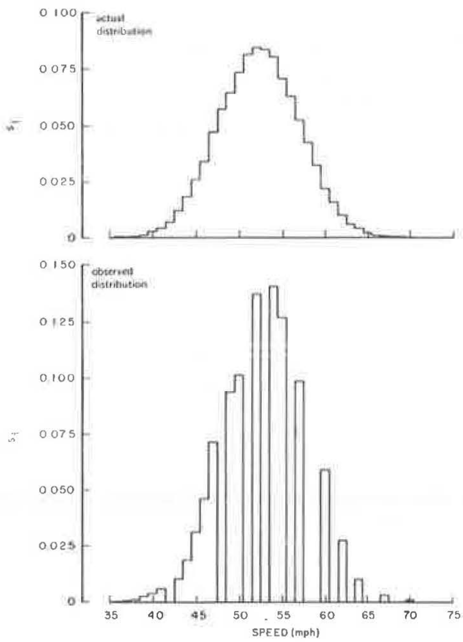

Figure 3. Speed distribution with speed intervals of constant width, N(52.3; 22.0), and speed trap length of 24 ft.

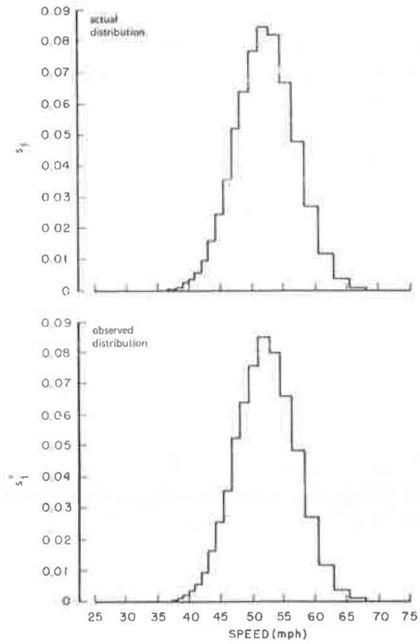

Figure 4. Speed distribution with speed intervals of varying width, N(52.3; 22.0), and speed trap length of 24 ft.

Figure 3 shows the resulting distributions for N(52. 3; 22.0), for the case of $I_i = 1$ mph for all i, and a speed trap length of 24 ft. This figure shows that many speed intervals that contain actual speed values do not contain any observed speed values, whereas others contain much more observed than actual speed values, and the dissimilarity between the actual and observed speed distributions grows with increasing speeds. This is due to the fact that the observed speed values are discrete and their separation increases hyperbolically with increasing speeds.

In order to account for the hyperbolically increasing separation between the discrete observed speed values, the same distribution is therefore presented classified into the variable intervals

$$\left[\frac{L}{U(0.01\text{ sec})} \times \frac{L}{(U-1)(0.01\text{ sec})}\right] \tag{3}$$

around the discrete speed values given by Eq. 1. The result is shown in Figure 4. In this case, the observed distribution simulates the actual distribution so much more accurately that it was decided to present all succeeding speed distributions with this variable interval size.

Figure 5 shows actual and observed distributions for N(50.0; 49.0) for the case of a speed trap length of 24 ft. The influence of the speed trap length on an observed speed distribution was studied by obtaining the observed speed distribution for a speed trap length of 30 ft for N(52.3; 22.0). The result is shown in Figure 6.

Visual comparison of the actual and observed distributions in Figure 4, 5, and 6 shows a good agreement. This was quantified by performing a chi-square test for each of these cases. The chi-square statistics are given by

$$\chi^2 = \sum_i \frac{(m_i - m_i')^2}{m_i} \tag{4}$$

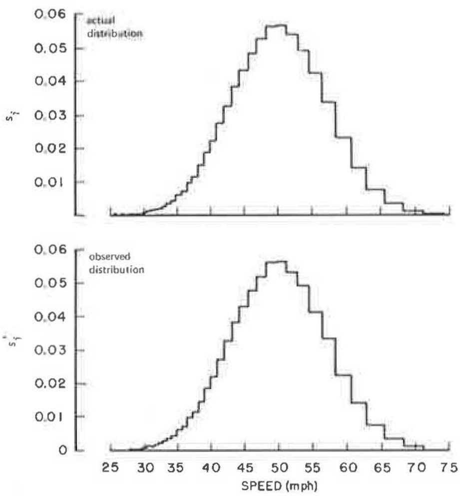

Figure 5. Speed distribution with speed intervals of varying width, N(50.0; 49.0), and speed trap length of 24 ft.

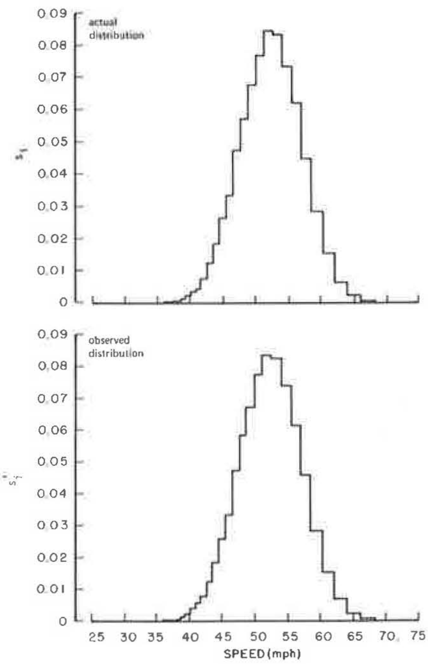

Figure 6. Speed distribution with speed intervals of varying width, N(52.3; 22.0), and speed trap length of 30 ft.

where m_i and m_i are the numbers of actual and observed speed values in the ith speed interval, and the summation is to be made over all speed intervals considered. In each speed interval used for this test, the number of vehicle pairs was well above six, so that the chi-square statistics apply. The number of degrees of freedom has been assumed as one less than the number of speed intervals used for this test. The results are given in Table 1.

With these results, the hypothesis that there is no effect of the measurement technique on the speed distributions in Figures 4, 5, and 6 cannot be rejected by chi-square tests taken at the 10 percent significance level (and thus cannot be rejected, of course, at the more usual 5 percent and 1 percent significance levels).

Mean and Variance of Actual and Observed Distributions

After this comparison of the shapes of actual and observed speed distributions, this section discusses the effect of the measurement technique on the mean and variance of the speed distributions. In order to include a speed distribution with a significantly different mean from the distributions studied in the foregoing, the speed distribution N(39.6; 21.6) for the fast lane in the case of fairly heavy flow (2) was added to the subsequent study. The the influence of both different mean with equal variance and different variance with equal mean could be evaluated.

TABLE 1

CHI-SQUARE TEST FOR DIFFERENCE BETWEEN ACTUAL AND OBSERVED SPEED DISTRIBUTIONS

Distribution	Trap Length (ft)	Speed Interval	Computed χ^2	Degrees of Freedom	Tabulated $\chi^2_{0.10}$
N(52.3;22.0)	24	Variable	13.1	19	27.2
N(50.0;49.0)	24	Variable	31.5	42	54.1
N(52.3;22.0)	30	Variable	20.0	26	35.6

TABLE 2

MEAN AND VARIANCE OF SPEED DISTRIBUTIONS

Distribution	Actual or Observed	Mean	Standard Deviation of Mean	Variance	Standard Deviation of Variance
N(52.3;22.0)	Actual	52.3590	0.216	21.94	1.35
	Observed	52.3686	0.220	22.45	1.40
N(39.6;21.6)	Actual	39.5788	0.214	21.57	1.33
	Observed	39.5819	0.215	21.73	1.35
N(50.0;49.0)	Actual	50.0434	0.322	48.88	3.02
	Observed	50.0517	0.327	49.31	3.08

The 100,000 speed values that were sampled from each assumed normal distribution using a 24-ft trap length were subdivided into 200 subsamples of 500 speed values each. The mean and the variance were computed for each subsample. These obtained values were then used to compute the mean of the entire sample, the variance of this mean, the mean variance, and the variance of this variance.

The values that resulted for each of the 3 normal distributions considered in this report are given in Table 2. To these results, the Wilcoxon Signed-Rank Test was applied to test the hypothesis that there is no difference between the means of the 3 actual and observed speed distributions. The alternative hypothesis is that the observed values are larger than the actual values. For the 3 assumed speed distributions, the actual values were subtracted from the observed values. The 3 resulting differences were ranked according to their absolute values, and the sign of the difference was attached to the ranks. The sum of the positive ranks, T_3, is computed. The probability $\Pr[T_3 \leq a]$, where a is the computed value of T_3, is then taken from the table (3). The hypothesis is rejected if $\Pr[T_3 \leq a] \geq 1 - \alpha$, where α is the significance level for the test.

This test was then applied also to the variances of the distributions. The results of this test are given in Table 3. They indicate that, for any chosen significance level, both means and variances of the observed speed distributions are larger than the corresponding values for the actual speed distributions. However, the differences in the means appear to be so small that they can generally be ignored for all practical purposes.

Space Parameters Computed From Time Series Data

The data from the traffic analyzer are time data; i.e., they are taken at a fixed location on the highway over a certain period of time. In contrast to such data are space data; i.e., data that are taken at a fixed moment of time over a certain stretch of highway. (Aerial photography is one method that yields space data.) Sometimes the mean and the variance of the speed distribution in space are of interest, but only time data are available. Then, use is often made of the relationship (4)

$$f_s(v) = (\bar{v}_s/v)\, f_t(v) \tag{5}$$

TABLE 3

WILCOXON SIGNED-RANK TEST OF RESULTS IN TABLE 2

Observed Minus Actual Mean	Signed Rank	Observed Minus Actual Variance	Signed Rank
+ 0.0096	+ 3	+ 0.51	+ 3
+ 0.0031	+ 1	+ 0.16	+ 1
+ 0.0083	+ 2	+ 0.43	+ 2
a = sum + = 6		a = sum + = 6	
$\Pr\{T_3 \leq 6\} = 1.000$		$\Pr\{T_3 \leq 6\} = 1.000$	

where $f_t(v)$ is the speed probability density in time, $f_s(v)$ is the speed probability density in space, and $\bar{v}_s$ is the space mean speed.

Integrating Eq. 5 over all speeds and solving for $\bar{v}_s$ yield the equation

$$\bar{v}_s = \frac{1}{\int_0^\infty \left\{\left[f_t(v)\right]/v\right\} dv} \tag{6}$$

By the use of this equation, the space mean speed can be computed as the inverse

TABLE 4

MEAN AND VARIANCE OF SPEED DISTRIBUTIONS IN SPACE

Distribution	Actual or Observed	Space Mean Speed	Standard Deviation of Space Mean Speed	Variance	Standard Deviation of Variance
N(52.3;22.0)	Actual	51.9349	0.218	22.03	1.38
	Observed	51.9353	0.222	22.50	1.42
N(39.6;21.6)	Actual	39.0207	0.219	21.77	1.38
	Observed	39.0202	0.220	21.91	1.39
N(50.0;49.0)	Actual	49.0309	0.334	49.64	3.18
	Observed	49.0316	0.337	50.01	3.21

of the harmonic mean of the speed probability density in time. Multiplying Eq. 5 by v^2 and integrating over all speeds yield the variance of the speed probability density in space. If the square of the space mean speed is subtracted from this variance, the variance about the mean

$$\sigma_s^2 = \overline{v}_s \overline{v}_t - \overline{v}_s^2 \tag{7}$$

is obtained, where $\overline{v}_t$ is the time mean speed. The use of this equation together with Eq. 6 then makes it possible to compute the variance of the speed probability density in space from time data.

For each of the speed distributions in time studied in the previous sections, the means and variances of the corresponding speed distribution in space were also computed by using Eqs. 6 and 7. Again, the data sample was subdivided into subsamples of 500 speed values each, and space mean speed and variance were computed for each subsample. These values were then used to compute the space mean speed for the entire sample, the standard deviation of this mean, the mean variance, and the standard deviation of this variance. The resulting values are given in Table 4.

The Wilcoxon Signed-Rank Test, described earlier, was applied to the differences between the actual and observed space mean speeds, as well as to the differences between the actual and observed variances. The test results are given in Table 5. These results indicate that the measurement technique does not effect a statistically significant change in the space mean speed but that it effects a statistically significant increase of the variance.

THE EFFECT OF THE MEASUREMENT TECHNIQUE ON RELATIVE SPEED DISTRIBUTIONS

In the following, the effect of the measurement technique on distributions of relative speeds is studied for the 3 selected underlying speed distributions of N(52.3; 22.0), N(39.6; 21.6), and N(50. 0; 49.0) and for the 2 speed trap lengths of 24 and 30 ft.

TABLE 5

WILCOXON SIGNED-RANK TEST OF RESULTS IN TABLE 4

Observed Minus Actual Space Mean Speed	Signed Rank	Observed Minus Actual Variance	Signed Rank
+ 0.0014	+ 2	+ 0.47	+ 3
- 0.0005	- 1	+ 0.14	+ 1
+ 0.0017	+ 3	+ 0.37	+ 2
a = sum + = 5		a = sum + = 6	
$Pr\{T_3 \leq 5\} = 0.875$		$Pr\{T_3 \leq 6\} = 1.000$	

Comparison of Actual and Observed Distributions

Relative speeds were computed as the differences between successive values of the sampled speeds,

$$(\Delta v)_j = v_j - v_{j-1}$$

$$j = 2, 3, \ldots, 100{,}000$$

From the computed relative speeds, the actual relative speed distribution

$$r_i = n_i/NI_i \qquad i = 1, 2, 3, \ldots \tag{8}$$

was formed, where n_i is the number of actual relative speeds in the ith relative speed interval of length I_i, and N is the total number of such values, which is 99,999 in this case. This procedure was then repeated for the observed speed values, and the observed relative speed distribution, r_i', was formed in analogy to Eq. 8.

Because the actual speed distributions are normal distributions, $N(\mu_V; \sigma_V^2)$, with mean μ_V and variance σ_V^2, the actual relative speed distribution is the normal distribution $N(0; 2\sigma_V^2)$.

Figure 7 shows the distributions for the relative speed intervals (-20, -19], ..., (-1, 0], (0,1], ..., (19, 20], where a parenthesis indicates an open end of the interval and a bracket a closed end. In this case, the resulting observed distribution is not symmetrical about zero because those observed relative speed values that are exactly equal to zero (this occurs when 2 adjacent observed speed values are equal) are classified into the interval (-1, 0]. This classification was discarded because the actual distribution is symmetrical about zero. In the following, only those classifications are used where the value zero is the midpoint of a relative speed interval.

For the 2 classifications (-20.5, -19.5], ..., (-0.5, 0.5], ..., (19.5, 20.5] and (-19, -17], ..., (-1, 1], ..., (17, 19]. Figures 8 through 13 show the relative speed distributions for N(52.3; 22.0), N(50.0; 49.0), and N(39.6; 21.6) and for a speed trap length of 24 ft. Figures 14 and 15 show the relative speed distributions for N(52.3; 22.0) resulting from the same classifications but for a speed trap length of 30 ft.

For each of these cases, a chi-square test, analogous to Eq. 4, was performed for the relative speed distributions. The results of this test are given in Table 6. These results indicate that actual and observed relative speed distributions are significantly different statistically in each case. It is apparent that a classification in 2-mph is always relatively preferable. The disagreement between actual and observed relative

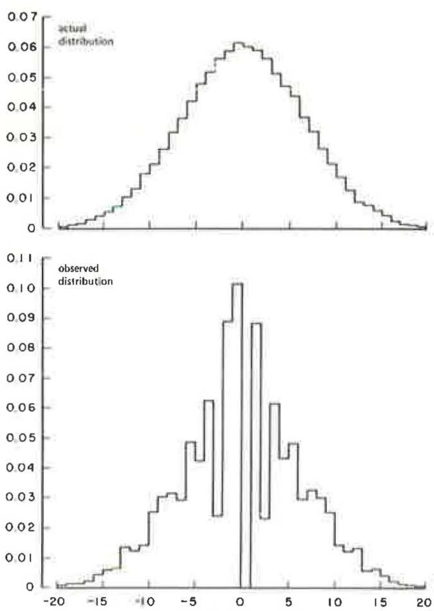

Figure 7. Relative speed distribution with 1-mph intervals (zero being the end point of one of them), N(52.3; 22.0), and speed trap length of 24 ft.

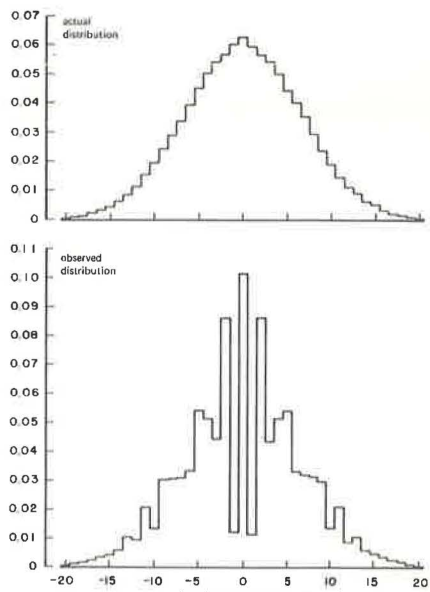

Figure 8. Relative speed distribution with 1-mph intervals (zero being the midpoint of one of them), N(52.3; 22.0), and speed trap length of 24 ft.

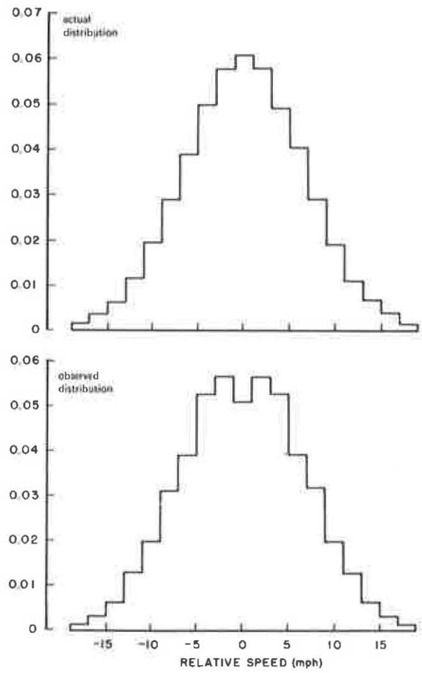

Figure 9. Relative speed distribution with 2-mph intervals, N(52.3; 22.0), and speed trap length of 24 ft.

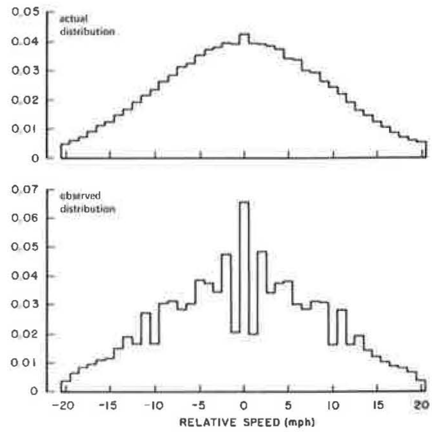

Figure 10. Relative speed distribution with 1-mph intervals, N(50.0; 49.0), and speed trap length of 24 ft.

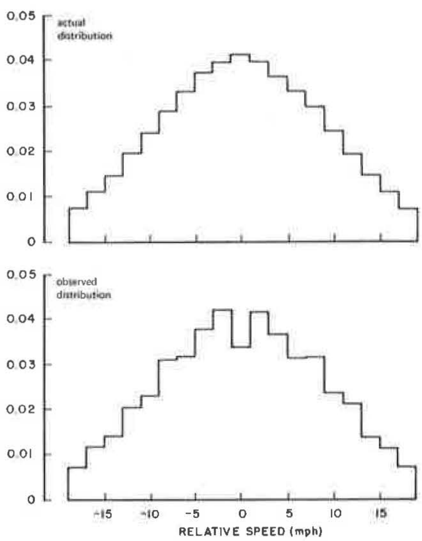

Figure 11. Relative speed distribution with 2-mph intervals, N(50.0; 49.0), and speed trap length of 24 ft.

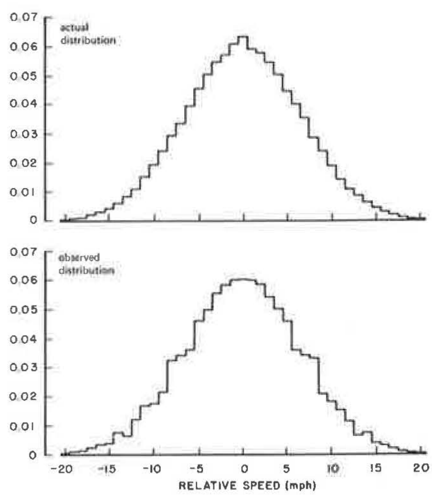

Figure 12. Relative speed distribution with 1-mph intervals, N(39.6; 21.6), and speed trap length of 24 ft.

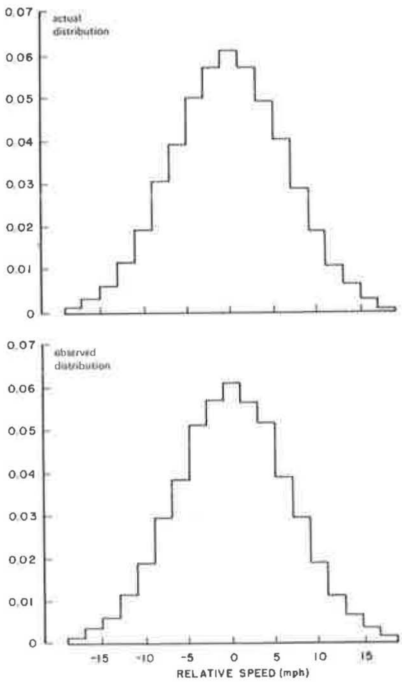

Figure 13. Relative speed distribution with 2-mph intervals, N(39.6; 21.6), and speed trap length of 24 ft.

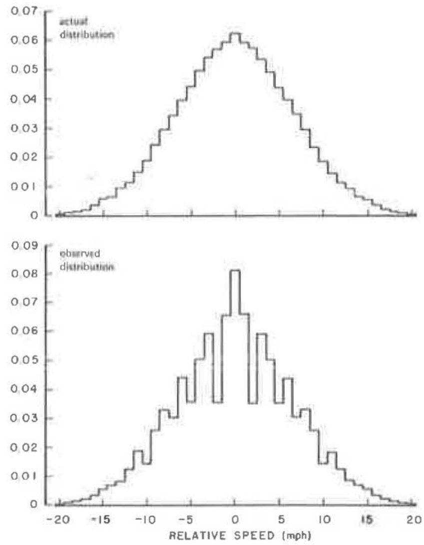

Figure 14. Relative speed distribution with 1-mph intervals, N(52.3; 22.0), and speed trap length of 30 ft.

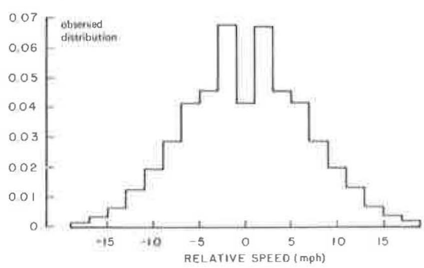

Figure 15. Relative speed distribution with 2-mph intervals, N(52.3; 22.0), and speed trap length of 30 ft.

speed distributions is worst in the case corresponding to Figure 7; this is one more reason to discard that kind of classification.

The change from a 24-ft to a 30-ft speed trap definitely reduces the disagreement between actual and observed distributions in the case of the 1-mph classification. Here the improvement by a 2-mph classification is relatively small.

Tentative results for a 3-mph classification have not yielded any substantial reduction in the disagreement between actual and observed distribution, and this was, therefore, not pursued further. Thus, it can be concluded that in the cases considered, no statistically significant similarity between actual and observed relative speed distributions can be achieved.

Mean and Variance of Actual and Observed Distributions

This section discusses the effect of the measurement technique on the mean and variance of these relative speed distributions. For this purpose, the same sample of 99,999 relative speeds utilized earlier was subdivided into 199 subsamples

TABLE 6

CHI-SQUARE TEST FOR DIFFERENCE BETWEEN ACTUAL AND OBSERVED RELATIVE SPEED DISTRIBUTIONS

Distribution	Corresponding Figure	Trap Length (ft)	Relative Speed Interval (mph)	Computed χ^2	Degrees of Freedom	Tabulated $\chi^2_{0.01}$
N(52.3;22.0)	7	24	1[a]	16,934	39	62.4
N(52.3;22.0)	8	24	1	15,354	40	63.7
N(52.3;22.0)	9	24	2	590	18	34.8
N(50.0;49.0)	10	24	1	4,743	40	63.7
N(50.0;49.0)	11	24	2	502	18	34.8
N(39.6;21.6)	12	24	1	958	40	63.7
N(39.6;21.6)	13	24	2	212	18	34.8
N(52.3;22.0)	14	30	1	1,682	40	63.7
N(52.3;22.0)	15	30	2	1,187	18	34.8

[a]Only classification in which central interval is not centered about zero.

TABLE 7

MEAN AND VARIANCE OF RELATIVE SPEED DISTRIBUTIONS

Generating Distribution	Actual or Observed	Mean	Standard Deviation of Mean	Variance	Standard Deviation of Variance
N(52.3;22.0)	Actual	-0.001128	0.0129	43.88	3.32
	Observed	-0.001160	0.0130	44.97	3.36
N(39.6;21.6)	Actual	-0.001118	0.0128	43.13	3.27
	Observed	-0.001141	0.0128	43.49	3.29
N(50.0;49.0)	Actual	-0.001684	0.0193	97.75	7.40
	Observed	-0.001643	0.0191	98.68	7.55

TABLE 8

WILCOXON SIGNED RANK TEST OF RESULTS IN TABLE 7

Observed Minus Actual Mean	Signed Rank	Observed Minus Actual Variance	Signed Rank
+0.000032	+ 2	+ 1.09	+ 3
+0.000023	+ 1	+ 0.36	+ 1
-0.000041	- 3	+ 0.93	+ 2
a = sum + = 3		a = sum + = 6	
$\Pr\{T_3 \leq 3\} = 0.625$		$\Pr\{T_3 \leq 6\} = 1.000$	

of 500 relative speeds each and 1 subsample of 499 relative speeds. In analogy to the earlier computations, mean and variance were computed for each of these 200 subsamples, and then the mean of the entire sample, the variance of this mean, the mean variance, and the variance of this variance were computed. The results are given in Table 7.

Again, the Wilcoxon Signed-Rank Test was applied to the differences between the actual and observed means, as well as to the differences between the actual and observed variances. The results of this test are given in Table 8. They indicate that there is no statistically significant difference between the actual and observed means of the relative speed distributions, whereas, the variance of the observed relative speed distribution appears to be statistically significantly larger than the actual variance.

CONCLUSIONS

The effect of the measurement technique caused by discrete time measurements on speed data was studied by the use of a computer program. This program sampled statistically independent speeds from normal speed distributions that approximated actually measured speed distributions quite well. Relative speed distributions were derived by computing differences between successive values of the sampled speeds. This way, the effect of car interaction that introduces a certain speed dependency has been ignored in the analysis of relative speed distributions. In reality, therefore, the effect of the measurement technique on relative speed distributions may deviate quantitatively from the results of this study, especially for heavy traffic flow. However, a speed dependency would not alter the qualitative features of the results obtained for the relative speed distributions in this analysis.

Differences in the actual and observed speed distributions due to the effect of the measurement technique can be made statistically insignificant, if the interval selected for classifying the speed values increases hyperbolically with increasing speeds. However, the measurement technique causes a statistically significant increase of the mean and variance of the speed distributions, whereby the change in mean is so small that it can generally be ignored for practical purposes. The space mean speed computed from the time series data appeared to remain unchanged in contrast to the corresponding variance, which increases significantly.

The specific way chosen for classifying relative speeds into intervals appears to be a prime factor in the quality of the measured relative speed distributions. Even though it was concluded that any classification makes actual and observed distribution significantly different statistically, a 2-mph classification appears to be preferable over a 1-mph classification. The value zero should be the midpoint, not the end point, of one of the relative speed intervals. The mean of the relative speed distributions is not significantly changed, whereas the variance is increased significantly.

In any study that uses speed data from time series measurements, special consideration should be given to the effect of discrete time measurements on the characteristics of the derived speed data. Results from such a consideration not only should influence the data analysis but also should be especially used to optimize the experimental setup prior to the acquisition of the data.

ACKNOWLEDGMENTS

The author would like to thank A. V. Gafarian for the clarifying discussions on mathematical topics of this study and T. Sands for his help with portions of the data reduction.

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AN EXPERIMENTAL VALIDATION OF VARIOUS METHODS FOR OBTAINING RELATIONSHIPS BETWEEN TRAFFIC FLOW, CONCENTRATION, AND SPEED ON MULTILANE HIGHWAYS

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Three methods frequently reported in traffic literature for obtaining flow-concentration-speed relationships were selected for an experimental validation study. In the first method, data are grouped into short successive time intervals for computations; in the second, cars are classified by computed virtual concentration; and in the third, cars are classified by their speeds. A standard method was developed against which these 3 methods were compared. The method used as a standard is based on the isolation of periods of constant traffic flow. The data used for the study were collected by the Federal Highway Administration's traffic analyzer and were taken at 12 unidirectional two-lane sites. Results show substantial agreement between the methods based on short time slices, constant flow intervals, and speed classes. However, the method based on virtual concentration disagrees with all of these in the high concentration range. Because the method based on virtual concentration yields flows much higher than any ever observed in practice, it should not be considered a valid method for obtaining flow-concentration-speed relationships.

•ALTHOUGH the idea of functional relationships between the quantities of flow, concentration, and speed is one widely used in traffic literature, no uniformity of opinion exists as to how the related values of flow, concentration, and speed should be derived from data taken at a fixed point in the roadway. This lack of uniformity is probably due in part to the fact that such data consist of measurements of the arrival times of cars and their speed from which flow and speed—but not concentration—can be directly derived. Concentration must be derived by some indirect method from the observed values of time headways and speeds. The necessity of deriving a value for the concentration from observed data has led to varying approaches in converting the raw data of headways and speeds to values on flow-versus-concentration and speed-versus-concentration curves. Differences in categorization of the raw data based on various theoretical assumptions about the nature of traffic flow, e.g., the theory advanced by Wardrop of independent streams of cars (1), have also led to divergent methods of calculating values for flow-concentration and speed-concentration curves.

In this study the results of 3 different methods commonly mentioned in traffic literature are compared: (a) The observed data are divided into small time intervals (on the order of a minute) and flow, space mean speed, and concentrations are calculated for each interval; (b) cars are classified by the computation of a virtual concentration for each car, and speed is calculated for each concentration class; and (c) cars are classified by speed, and an average virtual concentration is calculated for each class. All of these methods are applied to the same raw data taken from a multilane facility, and the resulting values are graphed.

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Whatever the backgrounds, 2 questions must be asked of these different methods in order to clarify this presently ambiguous problem of data reduction: Although different in methods of classifying raw data and deriving the related values of speed, concentration, and flow, do the methods give substantially the same results? If the methods do not give the same results, is there any reason for the rejection of one or more in favor of another?

In order to help us answer the first of these questions and to avoid a proliferation of graphical comparisons, we wished to develop a method that could be used as a standard against which we could compare other methods. The method developed was based on isolating from the observed data long time slices during which the flow remained constant (in the sense of the passage times of cars being indistinguishable from a stationary point process). The selection of these constant flow periods is fully described elsewhere (2). The rate of flow, space mean speed, and concentration for these time slices are then computed. To some extent this method represents an expansion of the method of short time slices, but differs in that (a) periods during which the flow of vehicles is changing rapidly are excluded, as they do not represent a steady-state condition; and (b) the estimates of flow, concentration, and space mean speed for the constant flow intervals are much more reliable than those for the short time interval because the number of cars included is substantially increased. The small amount of scatter in the estimates produced by the constant flow method makes it a useful standard for graphical comparisons.

The second of these questions dealing with the rejection of one method in favor of another is harder to answer. However, in the application of some methods, anomalies arise that indicate that the methods may not be valid; obviously, we prefer methods that do not have such anomalies.

The conclusions of this study are as follows:

1. There is substantial agreement among the methods based on short time slices, constant flow periods, and speed classes. The method based on virtual concentration disagrees with all of these in the high concentration range.
2. The method based on virtual concentration yields flows much higher than any ever observed in practice and should, therefore, not be considered a valid method.

THE VARIOUS METHODS STUDIED

The data available for this study were collected by the traffic analyzer and were obtained by measuring the times and speed at which cars passed a fixed point on the highway. These data were recorded for each lane. The passage times were recorded in units of $1/10{,}000$ hour, that is, 0.36 sec. The speeds were determined by measuring the time τ each car traveled a speed trap distance L. This time was measured in units of 0.01 sec. In this study, the value of L was 30 ft for the lighter traffic flow and 15 ft for the heavy traffic flow conditions.

From these data, values of flow, concentration, and speed were computed by 3 basically different methods published in the literature, and plots of flow versus concentration and average speed versus concentration were prepared. For an examination of the resulting relationships in graphical form, a standard was developed against which the relationships would be compared. In this study, a method based on constant flow periods, fully described elsewhere (2), was considered as the standard. This method and the 3 methods to be validated are described in detail in the following.

The Method Used as Standard

For the purposes of this study, traffic flow can be regarded as a statistical process generating passage times and associated values of speed in each lane. Average flow and average speed are 2 parameters describing some of the properties of this process. Estimates of such parameters will fluctuate with time. These fluctuations can be due to one or two causes, depending on the properties of the generating process. The first cause, the purely statistical fluctuations around the mean, is always present. The second cause, fluctuations of the mean, adds to the fluctuations of the first one only if the flow is not stationary in time. A good statistical estimate of parameters describing the process is obtained by selecting periods of flow that are stationary in time, i.e., periods in which the passage times of cars can not be distinguished from a stationary point process.

For each period thus obtained, the parameters flow

$$q = N/T \tag{1}$$

space mean speed (harmonic mean of observed speeds)

$$\bar{v} = \frac{N}{\sum_{i=1}^{N} 1/v_i} \tag{2}$$

and concentration

$$k = q/\bar{v} \tag{3}$$

were computed, where T is the time length of the constant flow period, N is the number of cars passing the point of measurement in this time period, and v_i is the measured speed value for each car.

These parameters were computed for each lane separately, as well as for both lanes superimposed, whereby the superimposed case was obtained by superimposing the statistical processes measured separately for each lane.

The Methods To Be Validated

$\frac{1}{50}$ Hour Groupings—The entire time period of data acquisition was divided into successive time intervals of the length of $\frac{1}{50}$ hour each (3). This was a more convenient length than 1 min, which is most frequently used, because passage times of cars were measured in units of $\frac{1}{10,000}$ hour for each time interval. The quantities

$$q = 50\,N$$

in cars/hour,

$$\bar{v} = \frac{N}{\sum_{i=1}^{N} 1/v_i}$$

in miles/hour, and

$$k = q/\bar{v}$$

in cars/mile were computed, where N is the number of cars passing the point of measurement in the particular time interval under consideration ($\frac{1}{50}$ hour). These computations were performed for each lane separately, as well as for both lanes superimposed.

Virtual Concentration—A value of virtual concentration (3) was computed for each car as the inverse of its space headway. Because only time headway and speed of each car were measured, the space headway in this case had to be estimated. It was assumed that the speed of the n - 1 car, v_{n-1}, remained constant for a time period t_n, which is equal to the time headway of the nth car after it had passed the point of measurement. The space headway was then computed as the product of v_{n-1} and t_n. This yields the virtual concentration in cars/mile

$$k_n = 1/(v_{n-1}t_n) \tag{4}$$

for the nth car. The space headways for all cars were classified into intervals of 5 ft. For each such interval, the quantities

$$k = \frac{N}{\sum_n v_{n-1} t_n} \tag{5}$$

$$\bar{v} = \frac{N}{\sum_n 1/v_n} \tag{6}$$

$$q = k\bar{v} \tag{7}$$

were computed, where N is the number of cars in the concentration interval under consideration. These computations were performed for each lane separately. In this case, computation could not be made for both lanes superimposed because then the spacings obtained between cars do not correspond to their real spacings. As a matter of fact, the time headway—and thus the spacing—can be zero for some cars, leading to an infinite value of virtual concentration for these cars. Instead, both lanes were added together; i.e., the computations were not altered but, disregarding lane number, all cars of both lanes were classified into the concentration intervals.

Classification by Speeds—Cars were classified by their speeds (3, 4) into the speed intervals determined by increments of 0.01 sec in time through the speed trap (5). For each such speed interval, a value of concentration was computed in analogy to Eq. 5 with the only difference being that here N is the number of cars in the speed interval under consideration. With the computed values of concentration for each speed interval, corresponding values of flow were computed by using Eq. 7, where $\bar{v}$ in this case is the midpoint of the speed interval. In analogy to the computations in the method of virtual concentrations, these computations were performed for each lane separately, as well as for both lanes added together.

COMPARISON OF THE METHODS

The methods described in the previous section were implemented for data taken at a two-lane unidirectional half of a highway in Virginia. The data were acquired by measuring the passage times and speeds of cars in each lane from 3:50 to 6:30 p.m. The resulting experimental values of flow concentration and speed given by the method based on constant flow periods are given in Table 1.

Figures 1 through 18 show the speed-concentration and flow-concentration relationships resulting from the use of the methods of $1/50$ hour groupings, virtual concentrations, and classification by speeds (classes containing less than 5 cars were not plotted). The

TABLE 1
CONSTANT FLOW PERIODS

Flow (cars/hour)			Concentration (cars/mile)			Speed (mph)		
Lane 1	Lane 2	Lanes 1 and 2 Superimposed	Lane 1	Lane 2	Lanes 1 and 2 Superimposed	Lane 1	Lane 2	Lanes 1 and 2 Superimposed
916	1,120	2,035	16.5	16.8	33.3	55.5	66.7	61.2
1,308	2,003	3,311	26.0	35.4	61.3	50.4	56.6	54.0
1,441	1,613	3,053	58.9	73.0	131.9	24.5	22.1	23.2
1,544	1,779	3,323	63.2	64.7	127.9	24.4	27.5	26.0
883	1,169	2,052	15.8	17.7	33.4	56.1	66.2	61.4

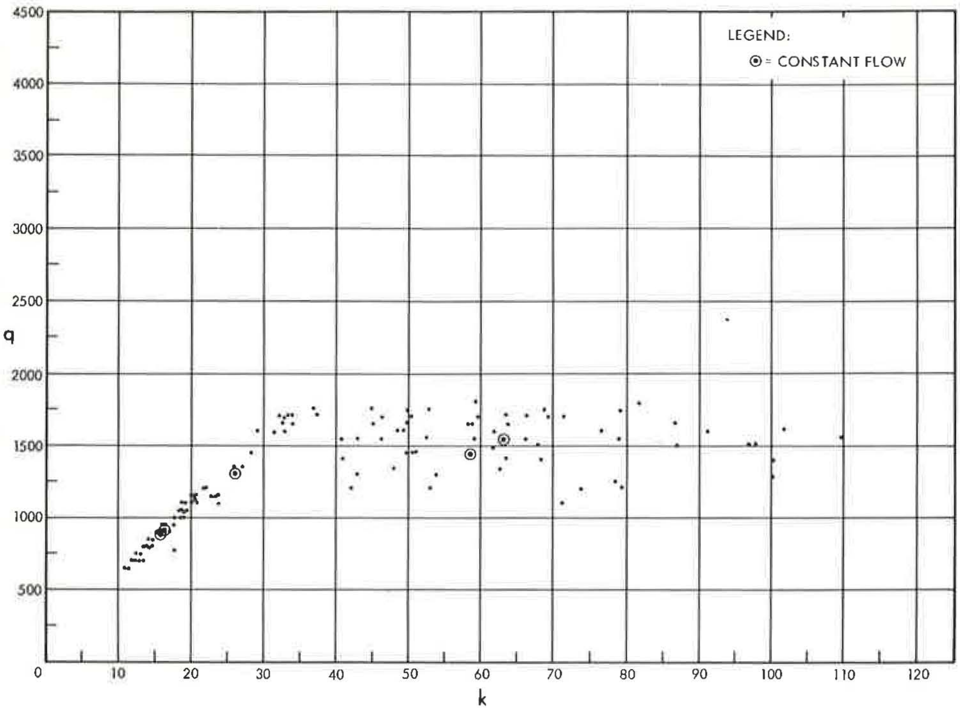

Figure 1. Lane 1, q - k plot, $\frac{1}{60}$ hour groupings.

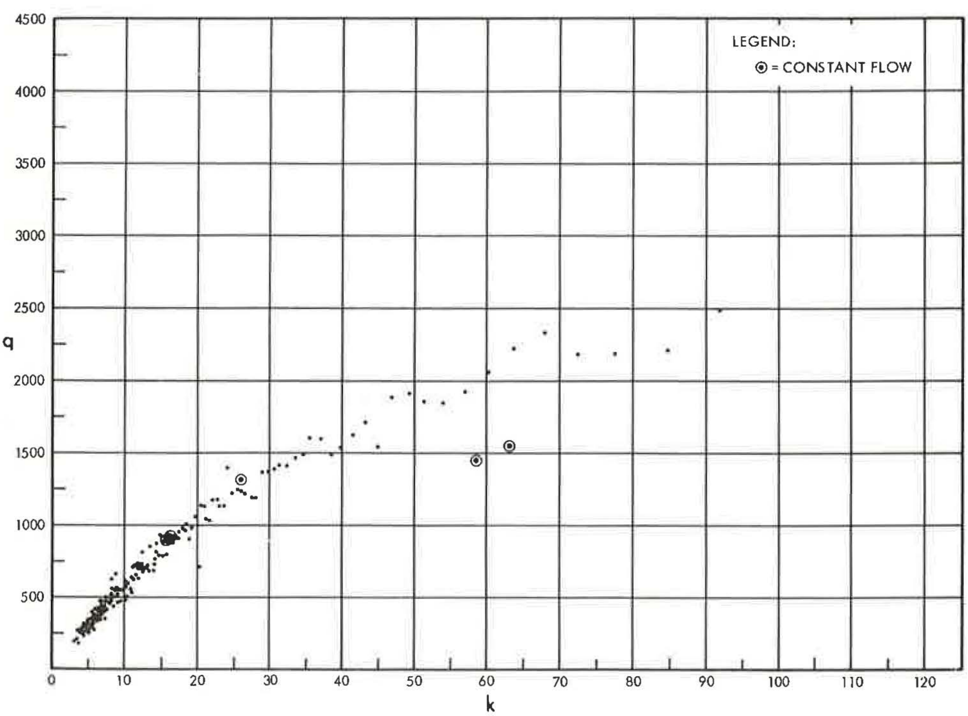

Figure 2. Lane 1, q - k plot, cars classified by virtual concentration.

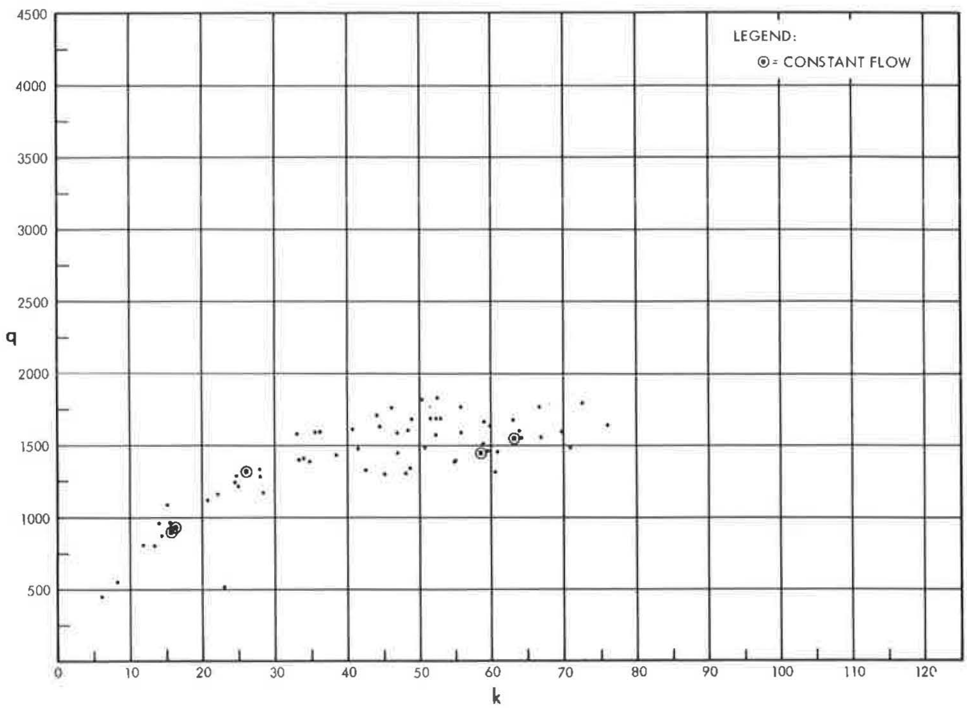

Figure 3. Lane 1, q - k plot, cars classified by speed.

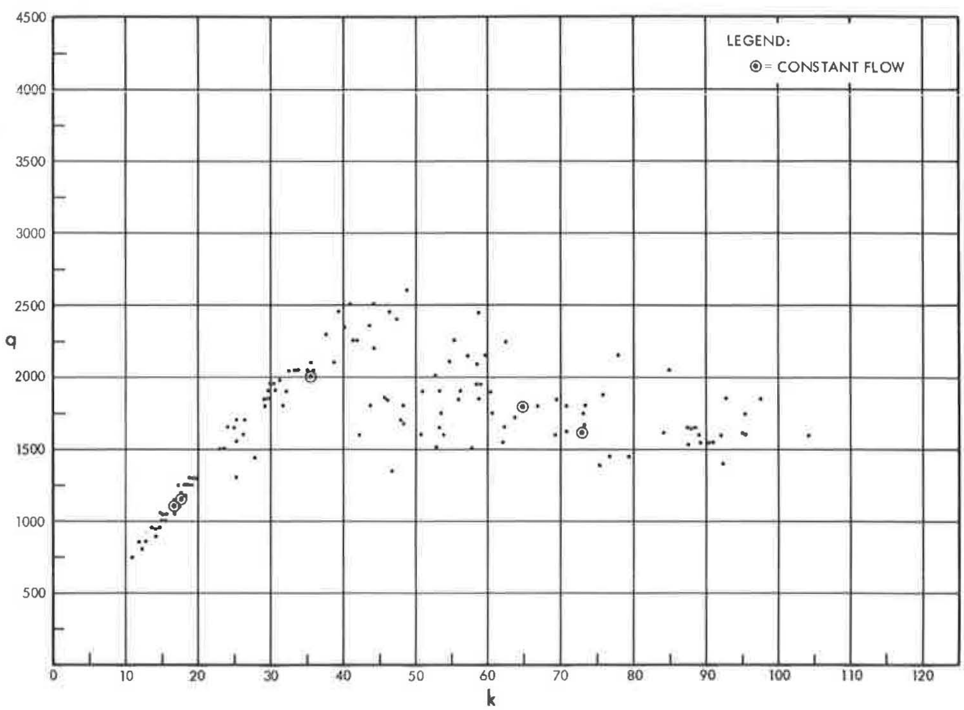

Figure 4. Lane 2, q - k plot, $\frac{1}{50}$ hour groupings.

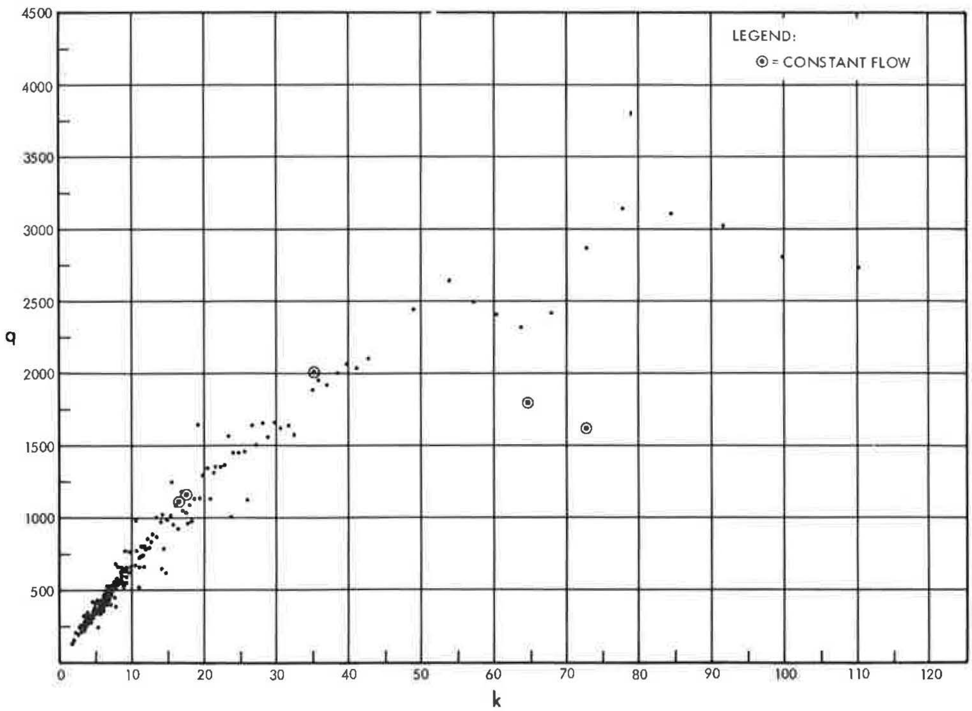

Figure 5. Lane 2, q - k plot, cars classified by virtual concentration.

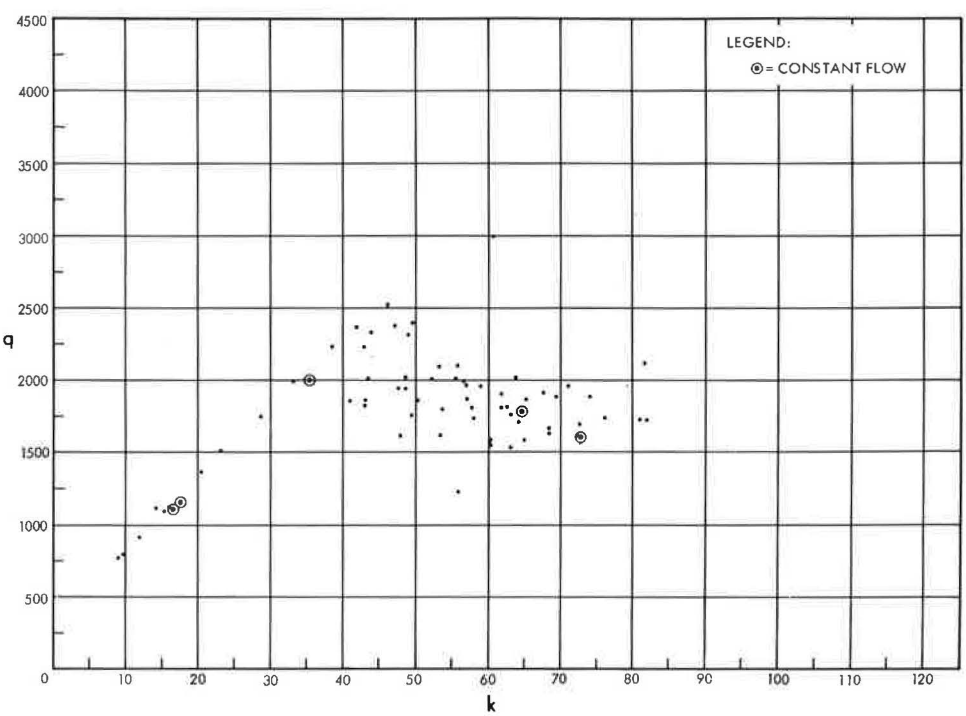

Figure 6. Lane 2, q - k plot, cars classified by speed.

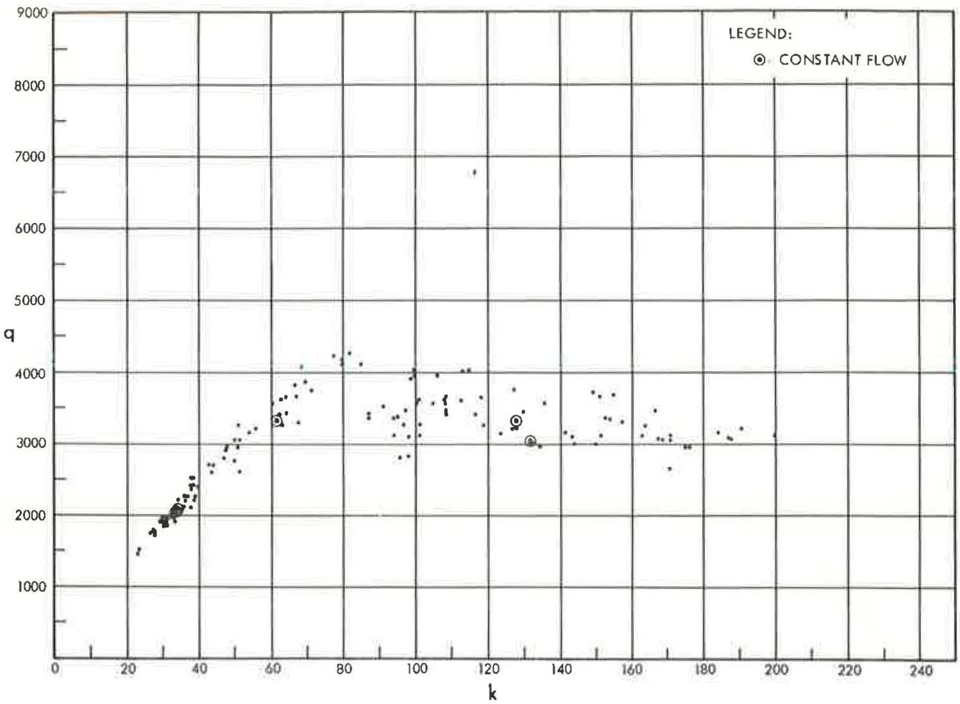

Figure 7. Lanes 1 and 2 superimposed, q - k plot, $\frac{1}{60}$ hour groupings.

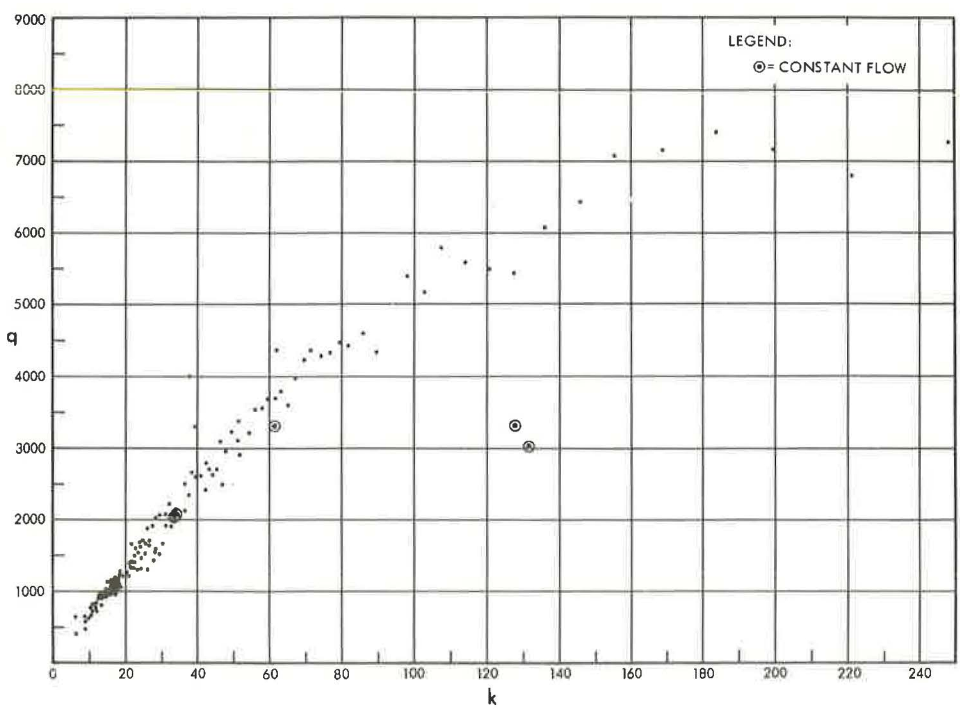

Figure 8. Lanes 1 and 2 added together, q - k plot, cars classified by virtual concentration.

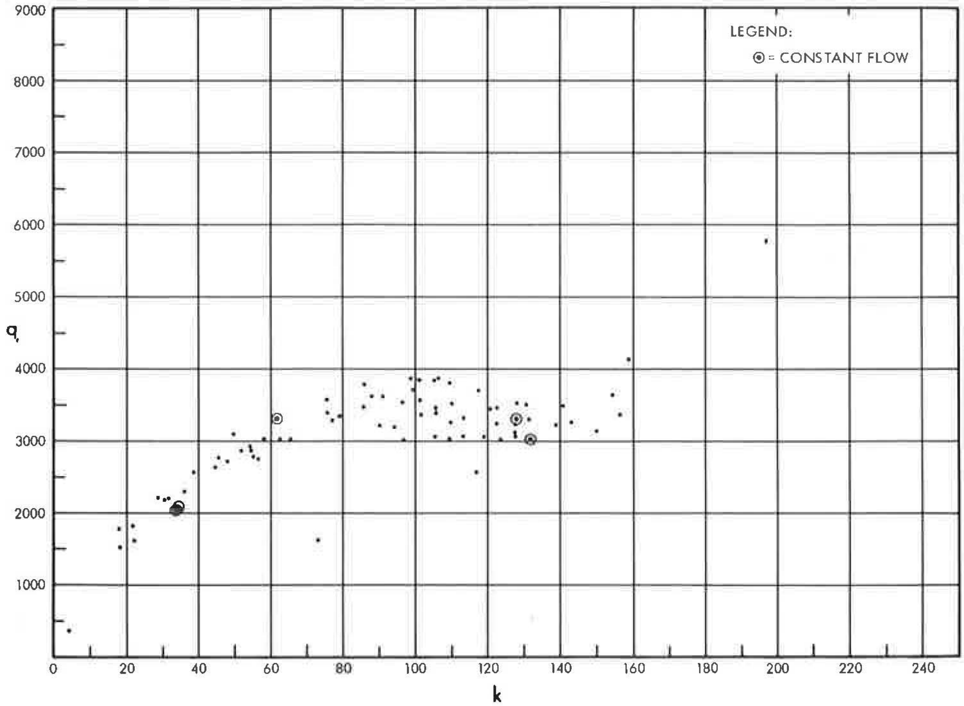

Figure 9. Lanes 1 and 2 added together, q - k plot, cars classified by speed.

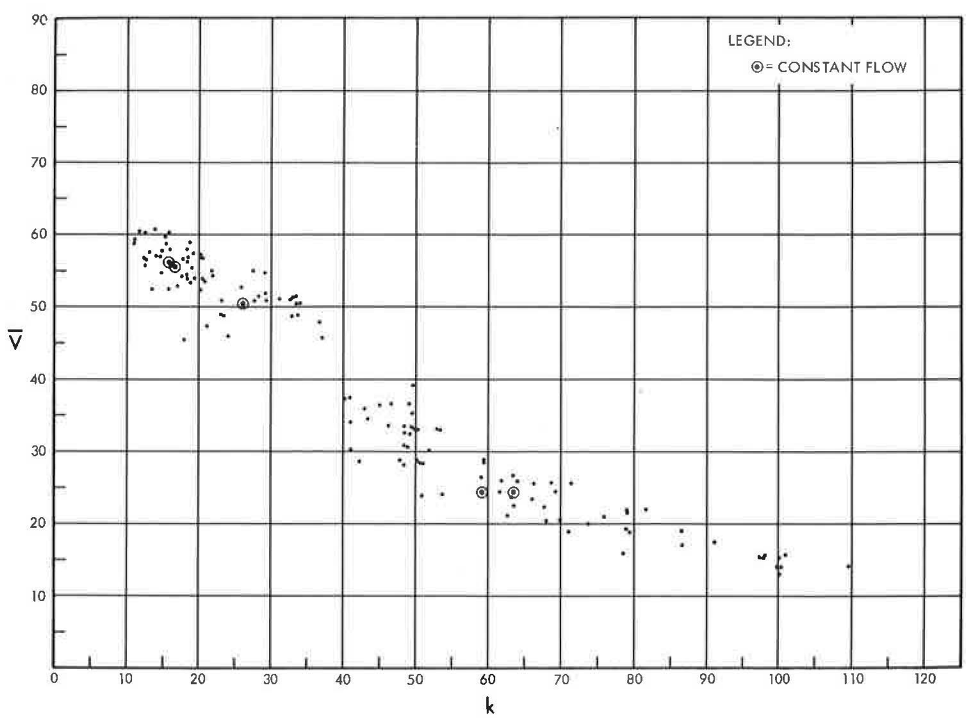

Figure 10. Lane 1, $\overline{v}$ - k plot, $^1/_{60}$ hour groupings.

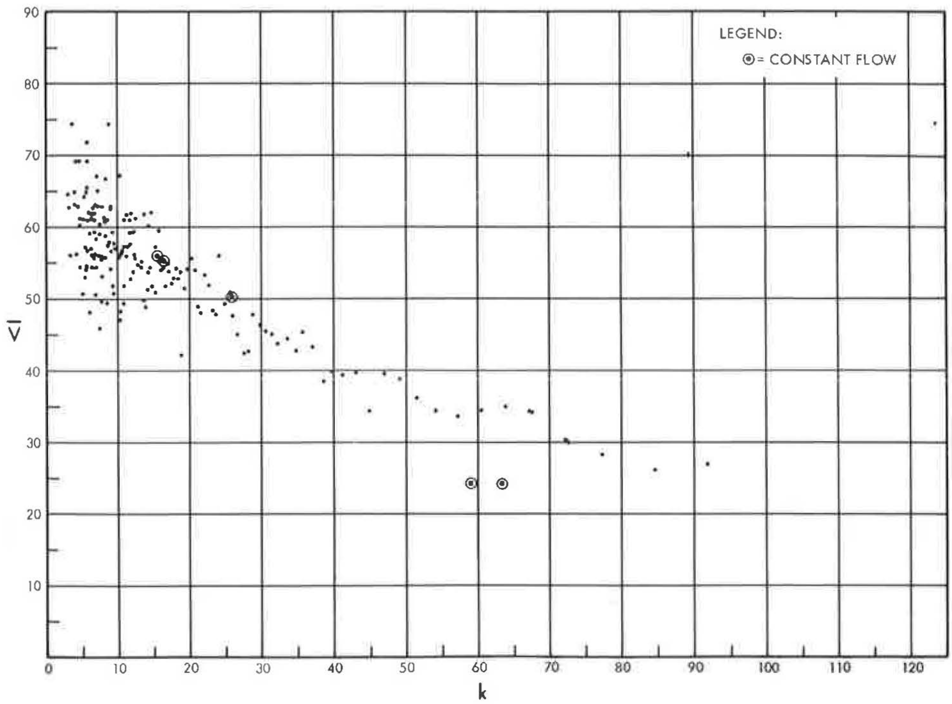

Figure 11. Lane 1, $\bar{v}$ - k plot, cars classified by virtual concentration.

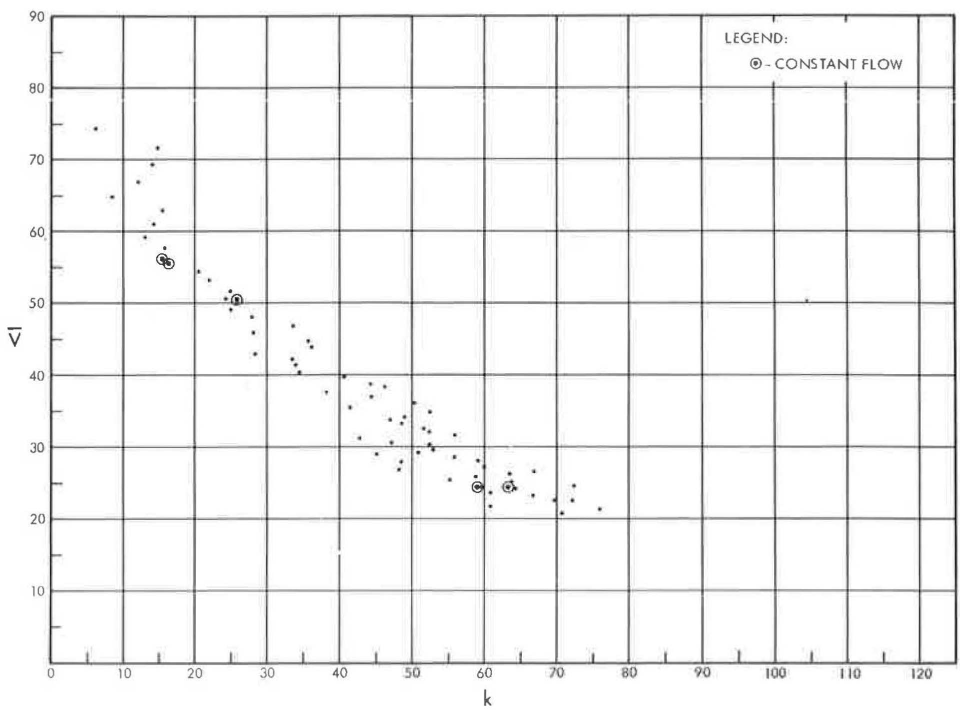

Figure 12. Lane 1, $\bar{v}$ - k plot, cars classified by speed.

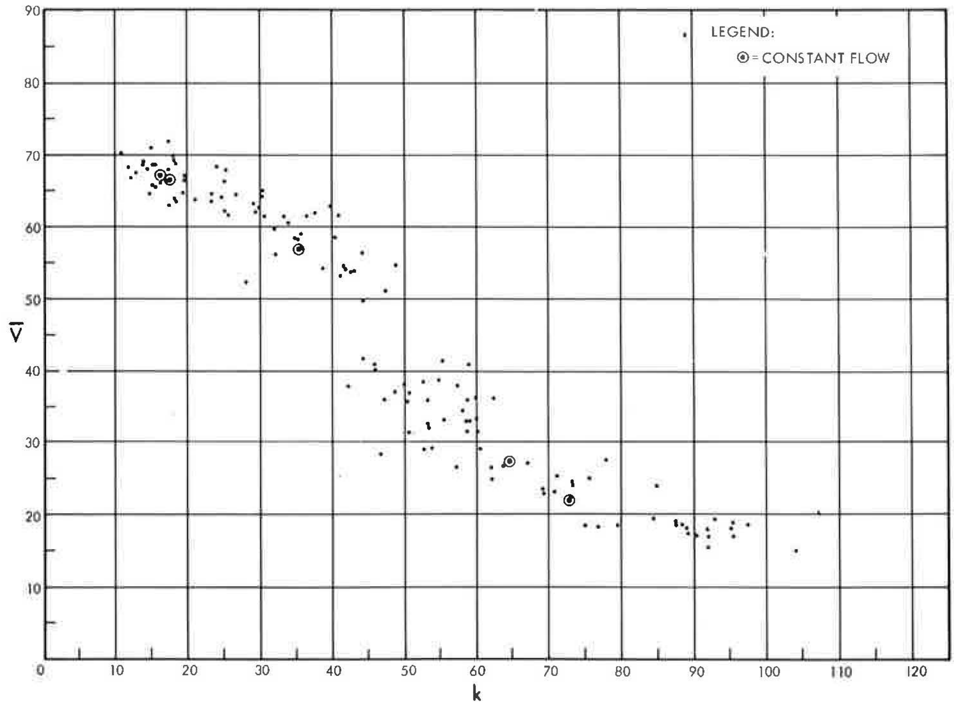

Figure 13. Lane 2, $\bar{v}$ - k plot, $^1/_{50}$ hour groupings.

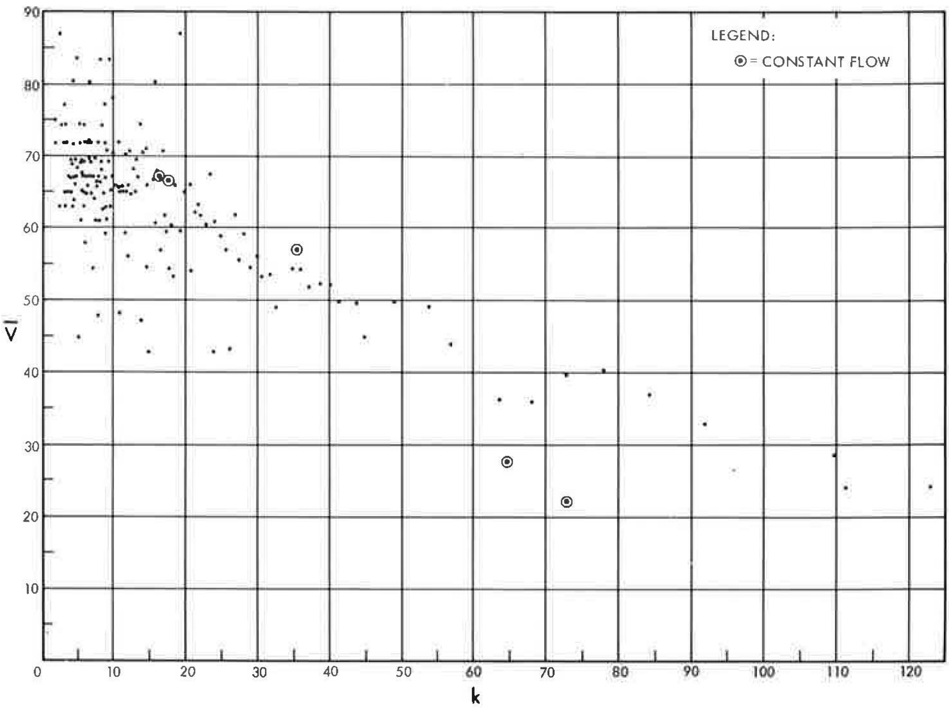

Figure 14. Lane 2, $\bar{v}$ - k plot, cars classified by virtual concentration.

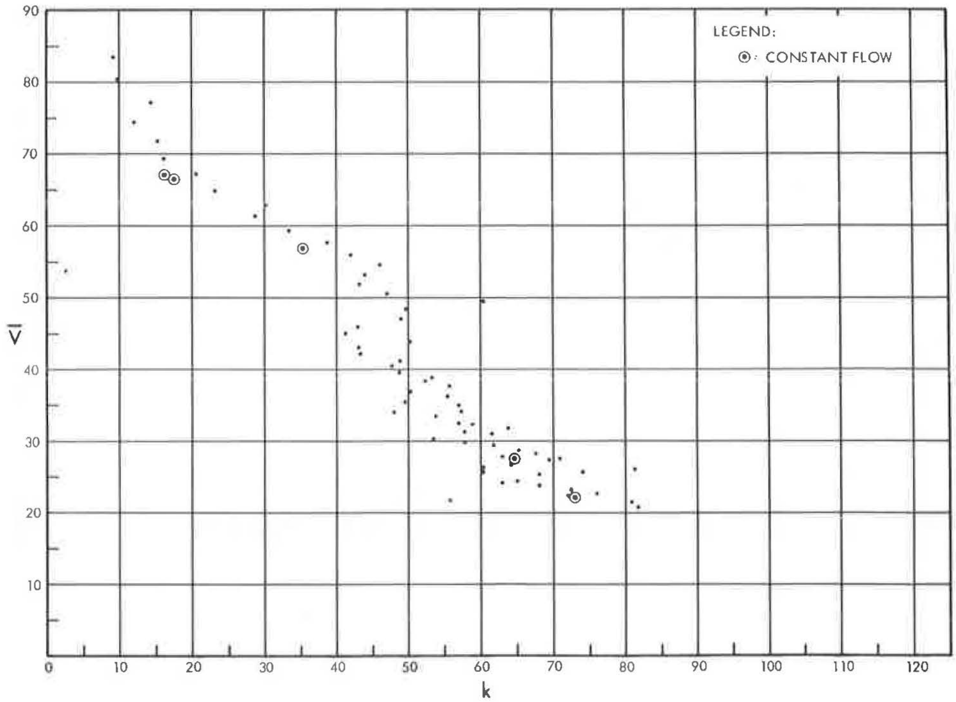

Figure 15. Lane 2, $\bar{v}$ - k plot, cars classified by speed.

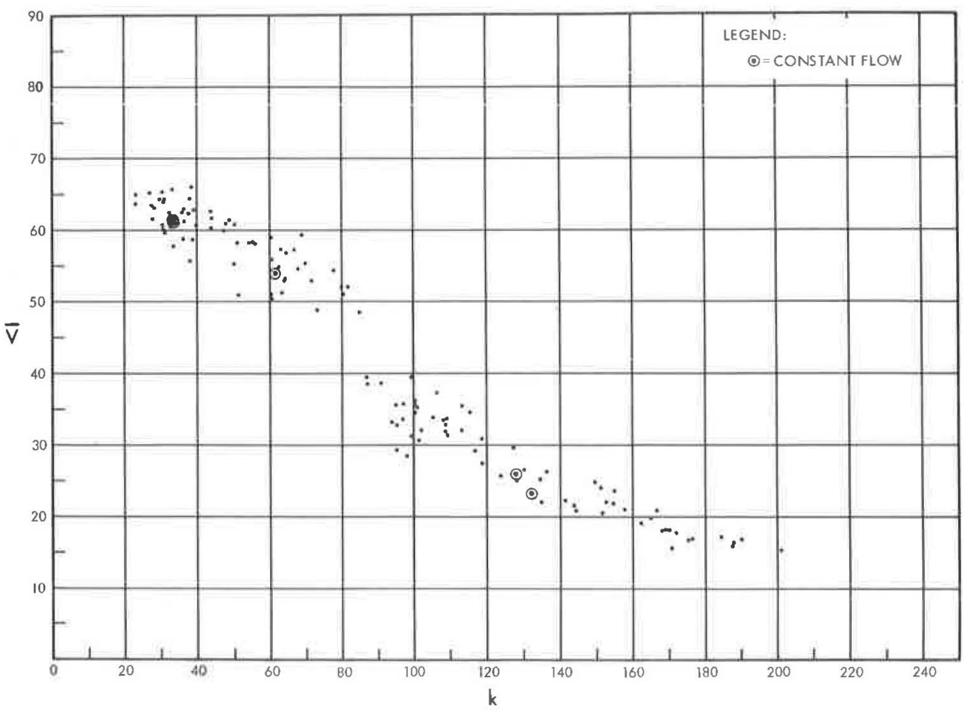

Figure 16. Lanes 1 and 2 superimposed, $\bar{v}$ - k plot, $1/50$ hour groupings.

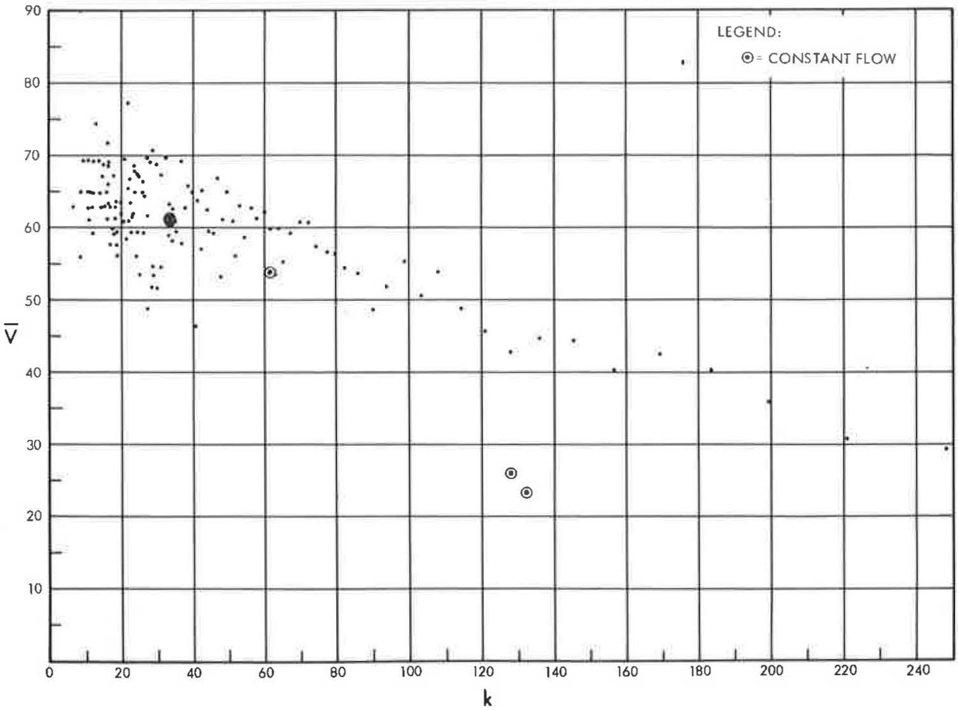

Figure 17. Lanes 1 and 2 added together, $\overline{v}$ - k plot, cars classified by virtual concentration.

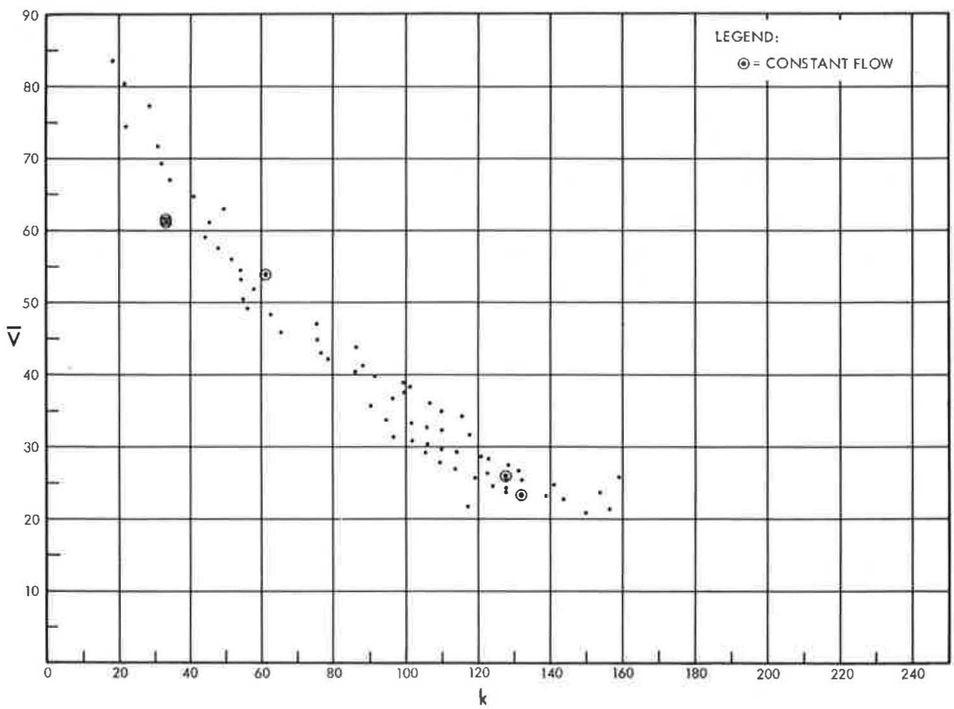

Figure 18. Lanes 1 and 2 added together, $\overline{v}$ - k plot, cars classified by speed.

TABLE 2
COMPARISON OF METHODS

Plot	Lane	Concentration Range	Methods to Be Validated		
			$\frac{1}{50}$ Hour Groupings	Virtual Concentration	Speed
$\bar{v}$ - k	1	Low	Agreement	Agreement	Agreement
		High	Agreement	Too high	Agreement
	2	Low	Agreement	Agreement	Agreement
		High	Agreement	Too high	Agreement
	1 and 2 superimposed or added together	Low	Agreement	Agreement	Slightly too steep slope
		High	Agreement	Too high	Agreement
q - k	1	Low	Agreement	Agreement	Agreement
		High	Agreement	Too high	Agreement
	2	Low	Agreement	Agreement	Agreement
		High	Agreement	Too high	Agreement
	1 and 2 superimposed or added together	Low	Agreement	Agreement	Slightly too steep slope
		High	Agreement	Too high	Agreement

points resulting from the constant flow method are superimposed on each graph. Table 2 gives the results from the comparison of the 3 methods to be validated with the method of constant flow periods.

The method of $\frac{1}{50}$ hour groupings gives satisfactory agreement with the standard method. For low concentrations, the virtual concentration method tends to agree with the standard method; for the higher concentration, however, this method yields very high values of flow and speeds relative to the standard. We note that the values of flow are higher than ever reported for flows in a two-lane facility. The method of classification by speeds tends to agree with the standard at the higher concentrations and at low concentrations for the single-lane case. At the low concentrations for both lanes taken together, however, this method results in slightly too steep a slope of the $\bar{v}$ - k curve, and a slightly low slope of the q - k relationships.

The fact that the method of classification by speeds agrees with the constant flow method for the lanes taken by themselves and disagrees somewhat with the constant flow method for both lanes taken together leads us to believe that the speed classification method should perhaps not be extended to the derivation of q - and $\bar{v}$ - k relationships for both lanes taken together.

Analogous procedures were repeated for 11 other two-lane, one-way sites. The concentration range of the data for these sites was smaller in each case than for the site presented. However, within these smaller concentration ranges the comparison of all 3 methods with the standard resulted in the same conclusions as obtained for this site. Therefore, only this site was selected for the purpose of demonstrating the results of this study.

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Discussion

JOSEPH A. WATTLEWORTH, Civil Engineering Department, University of Florida, Gainesville—The authors have presented the results of quite an interesting study. These results will be important to many people who are involved in research on traffic stream flow or in real-time surveillance and control of traffic flow. Frequently some of the measurements that are used by traffic engineers have a somewhat mysterious air to them and are not completely understood. Scientific analyses of these measurements, such as the analyses conducted by the authors, are welcome and important.

The authors have demonstrated that the methods used to obtain data for a particular variable are quite important as well as the variable itself. In the reported study, one of the three data collection procedures that was used (the virtual concentration technique) was rejected because of the inconsistency of data obtained by this technique with the data obtained by the other methods.

Three methods of obtaining flow, speed, and concentration data were compared to a standard method. The standard method involved the use of data for periods in which the flow rate remained relatively constant. Flow was measured directly and speed-trap travel times were used to calculate the space mean speed for the period. Concentration was then calculated from the basic stream flow equation, $q = kv$, where q = flow rate, k = concentration, and v = space mean speed.

In the first method, these same data were collected for time periods of $\frac{1}{50}$ hour, and values for q, k, and v were calculated for each time period. In the second method, the virtual concentration of each vehicle was calculated from the measurement of travel time in the speed trap. Space mean speed for the time period was also calculated from the travel time measurements. Flow rate was calculated from concentration and space mean speed. The periods of analysis were periods of constant concentration. The third method involved the classification of data into groups in which the speeds were essentially constant. In this method, concentration for each period was determined from the travel time samples and the midpoint of the speed interval was used as the speed. Flow rate was then calculated from these values.

The authors found that the results of the first method correlated most closely to the standard method. This, perhaps, should not be surprising because of the similarity of the data collection techniques. One would expect more variation in the test method data because of the smaller time periods used. For this method the optimal density agrees well with values of 40 to 60 vehicles/lane-mile obtained in previous studies (6, 7). This method depends on speed measurements in which the speed trap travel time has an accuracy of ±0.01 sec. For a speed of 60 mph, the travel time over the 30-ft trap is about 0.33 sec. The speed measurements, then, have an accuracy of ±3 percent.

The virtual concentration method yielded flow values that were unreasonably high (2,500 to 3,000 vehicles/hour). This possibly is due to some extremely short time periods included in the analyses. The report did not discuss this possibility. In addition, this method is heavily dependent on headway measurements. Arrival times of each of the vehicles constituting a particular headway have a measurement error that falls in the range from -0.36 sec ($-\frac{1}{10,000}$ hour) to zero. Therefore, the headway has an accuracy of ±0.36 sec. For a headway of 2.0 sec, the accuracy of its measurement is ±18 percent. Because flow rates are based on the inverse of the measured headways, the measured flow rates would tend to be biased somewhat on the high side. These measurement errors would not appear to account entirely for the extremely high flow rates that were obtained in the virtual concentration method.

Some of these same comments apply to the third method, that in which the time slices were based on constant speed values. Flow rate in this method is also determined by headway measurements.

The conclusions of the authors would appear to be valid. The method of constant time slices is the most accurate of the 3 methods tested.

The authors have made an important contribution to the understanding of the measurements of some of the basic variables of traffic stream flow.

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RICHARD ROTHERY, General Motors Research Laboratories, Warren, Michigan—Three methods that have been used for calculating estimates of speed, concentration, and flow on individual lanes of a multilane highway by previous investigators are compared to a fourth technique. Because this latter technique is used as a standard, it is of value to focus attention on it here.

The procedure that the authors have used is well known in the field of time series analysis (8). In the analysis of a time series of events (in this case, vehicle arrival times), a primary objective is to determine the existence of trends or deviations from a constant mean flow that may be present in the data. One particularly effective technique that reflects this gross property is a graphical presentation of the data where the total number of vehicles that have passed an observation point at or before a time, t, is plotted against t. Examples of this graphical technique in analyzing traffic flow may be found in the literature (2, 9, 10).

The data presented and analyzed by the authors are of particular interest because they use this approach to establish periods during which the mean flow may be regarded as constant. Five such constant flow periods were obtained from the data and these 5 estimates of flow together with the corresponding estimates of concentration form the skeleton of a flow-concentration "curve." Possibly the most interesting facet of this paper is that the estimates of flow and concentration using the method of constant time intervals or constant speed classifications agree with this curve. Such agreement implies that flow and concentration fluctuations follow the same relationship around the mean.

The method denoted as using virtual concentrations deserves special comment. Virtual concentration is a misnomer because the data in this case are classified into groups according to their spatial headway in intervals of 5 ft. That this technique leads to speed estimates that are biased has been known. The bias is inherent in the technique. It results directly from dispersion in the distribution of spacings for a given speed even if such dispersion were to be symmetrical and speed-independent. The effect is to bias speed estimates toward higher values at high concentrations and lower values at low concentrations. The inadequacy of this approach has also been pointed out by Edie and Foote (3).

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SIDNEY WEINER, Federal Highway Administration—This paper presents a rather simple, although useful, technique in the exploratory validation of seemingly diverse methods for obtaining joint measurements on freeway characteristics, namely, flow, concentration, and speed. This is accomplished by first establishing a standard method that isolates periods of constant traffic flow. Thus, if q can be considered to be constant, the observed concomitant variables, speed and concentration, can then be regarded as properly identified.

Actually, as is indicated by Breiman and Lawrence (2), one desires as reliable a measurement as possible for flow rate. If the measurements were based on a short period, then fewer samples are involved leading to a considerable variation in the calculated quantity whether it be flow, speed, or concentration. To reduce such "short-scale" variations, the standard method first isolates those time periods during which traffic flow data exhibit a stationary behavior. This then permits one to form an estimate of q based on a longer stretch of the stationary flow than otherwise possible. The statistical variation of such an estimate is thereby reduced, and our reliance on this figure is accordingly increased. Concomitant measurements on such a flow rate give us a useful representation of the overall behavior of the freeway. It is noted that this method yields only 5 data points to be used as bench marks as given in Table 1 and plotted on each of the 18 figures. It is remarkable that only these few points result from 160 min of observation.

In the next step, the authors plot the set of points obtained by each of the frequently employed methods on separate graphs and visually compare each set with the standard set. This comparison is clearly given in Table 2 indicating that the method of categorizing traffic flow data by virtual concentration for each car is invalid because it yields very high values of flow and speed at higher concentrations. For measurements taken on individual lanes, the 2 other methods, one based on $\frac{1}{50}$ hour groupings and the other based on speed interval classification, were both found to be in agreement with those points presented by the constant flow or standard method.

On the face of it, one gets the clear impression that the 3 methods, the constant flow, the categorization by $\frac{1}{50}$ hour interval, and the categorization by speed interval, exhibit qualitative agreement. However, it is implicit that there is certainly a distinction among the underlying models. If we examine the q - k curves, for example, the constant flow method considers the flow to be approximately constant for certain ranges of the concentration, while those based on the other 2 methods consider the concentration to be approximately constant for a relatively small time interval or small speed interval. The assumptions underlying the other 2 models are not in disagreement with the assumption underlying the standard model—they are merely less stringent. It may be desirable to develop a theoretical analysis to examine these methods so as to determine to what extent any method may underestimate or overestimate the underlying relationships or to establish appropriate criteria in their selection.

One final remark may be added. This study was based on traffic analyzer data measuring traffic variables past a point for a given time period. At present we are obtaining spatial data involving one-half mile or more of traffic. We, thus, can obtain direct estimates of the concentration and better estimates of the other parameters. With such improved data, more valid estimating procedures can be provided for these traffic parameters.

A. V. GAFARIAN, R. L. LAWRENCE, P. K. MUNJAL, and J. PAHL, Closure—The authors would like to thank Rothery, Wattleworth, and Weiner for their discussions and suggestions concerning this paper.

In their discussions both Rothery and Weiner focus on the standard method that uses data from periods of constant traffic flow. We note that the technique used to isolate these periods represents a statistical analog of the graphical technique discussed by Rothery. This nonstandard statistical testing procedure that was developed by Breiman and Lawrence (2) consists of a sequential test of the hypothesis that there is no change in average flow and an estimation of where the change in flow occurred when the hypothesis is rejected. The advantages of this procedure over the original graphical technique are that it could be used easily on large amounts of data, it was reproducible, and the constant flow intervals chosen did not tend to exclude occasional large random fluctuations in flow.

As Rothery mentions, the inadequacy of the virtual concentration method has been pointed out by Edie and Foote in their experimental studies of traffic flow in tunnels. However, their study concerned a single lane with no passing situation, and it was felt by the authors that a test of this method on a multilane facility would be of interest. It is clear that, if the distribution of spacings were speed independent, a bias would result from this method; however, if such independence does not hold, it is not readily apparent that the method is biased.

Wattleworth's error analysis is appreciated and forms a useful addition to the paper as presented. We feel there are 2 points that should be clarified: (a) The error figures presented in the analysis represent the maximum possible error, and in general, the measurement error is considerably smaller; and (b) for the most part we are concerned with mean values of the measured variables, and the errors in the estimations of these means are, of course, substantially smaller than the errors of the individual measurements.

Weiner's comment on the desirability of developing a theoretical analysis of the accepted models to determine potential biases is well taken. Such analysis, in combination with spatial data now being obtained by the FHWA, may lead to establishing the relative accuracy of the accepted methods.

THE DRIVER IN SINGLE LANE TRAFFIC

Donald A. Gordon, Traffic Systems Division, Federal Highway Administration

A study was made of the reactions of drivers who were impeded by an experimental car moving at constant slow speed on a single-lane road. The situation was arranged so that the 20 subject drivers were not aware that they were being observed. The photographic records indicated the distances of the drivers behind the experimental car over the 3-mile course. Three modes of driver response were noted: (a) avoidance, where drivers moved backward out of the influence of the experimental car, 10 percent of the drivers; (b) car-following, where drivers stayed close to the experimental car and did not execute large backward or forward movements, 30 percent of the drivers; and (c) a combination of avoidance and car-following, 60 percent of the drivers. Drivers' lead distance patterns did not conform to Herman's car-following equation. The equation may apply better to the situation where a driver reacts to disturbances introduced by the car in front. Drivers showed an indifference threshold. They accepted a range of positions behind the experimental car and reacted only when lead distance exceeded certain limits. A statistically significant resemblance was found in the driving patterns of operators who stayed close to the plant. This and previous studies suggest that the driver should be regarded as a strategist who continually adjusts his actions to fit his travel purpose and the road conditions he faces. He is not a stereotyped reacting element or "black box" to be simply described by a fixed equation.

•ALTHOUGH many studies have been made of traffic flow (1, 2, 4), the research has not generally been related to the actions of the individual driver. Yet, the driver is the active element in the vehicle-highway-driver system. To a considerable extent, traffic flow reflects his behavior and psychology.

The aim of this study is to describe driver behavior in single-lane traffic. In this situation, the actions of the driver affect subsequent drivers in the traffic stream and, hence, are of considerable interest to the traffic engineer. An experimental vehicle or plant was deliberately introduced into the traffic stream and driven at a speed slower than that of the traffic stream itself. Impeded drivers behind the plant reacted to it, and their responses of slowing, matching pace, and so on were photographed and later analyzed.

In this study the plant moved at constant speed. All changes in velocity between the plant and the observed vehicle were made by the rear driver. Driver responses would be expected to be more clearly revealed in this simplified setting than would have been the case if the lead driver had also altered speed. It was also considered important for methodological reasons that drivers not be aware that they were under surveillance. The author knows of no form of experimental instructions that would ensure that a driver, knowing that he is being observed, would drive as he normally does.

BACKGROUND OF THE PROBLEM

The best known theory of drivers' reactions on single-lane roads is associated with the work of Herman and his co-workers (7). On the basis of experimental observations

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on a test track and in New York City tunnels, Herman derived the following equation to describe what has been called "car-following":

$$[(d^2x)/dt^2]_{n+1} = \alpha_0 \left[\frac{(dx/dt)_n - (dx/dt)_{n+1}}{x_n - x_{n+1}} \right] \tag{1}$$

where

$[(d^2x)/dt^2]_{n+1}$ = acceleration of the following car;
α_0 = constant related to speed;
$(dx/dt)_n - (dx/dt)_{n+1}$ = difference in speed between cars; and
$x_n - x_{n+1}$ = headway distance between cars.

(Herman included a term to account for man-machine lag. This term is omitted in Eq. 1 because the lead car was driven at constant speed.) The equation states that the acceleration of the following car is directly proportional to the difference in speed between the 2 cars and inversely proportional to the distance between the vehicles.

If Herman's equation is integrated, the following relationship is obtained:

$$(dx/dt)_{n+1} = \alpha_0 \log \left[\frac{x_n(t) - x_{n+1}(t)}{L} \right] \tag{2}$$

where

L = effective length of the following car.

The variables in this equation, velocity of the following car and distance between cars, are convertible in steady-state traffic to concentration and flow, the master variables of traffic flow. The implication is that Herman has found a basis for traffic flow in the reactions of the individual following driver.

To a psychologist, the term car-following seems to imply that the rear driver has the intention of following the car in front. This interpretation is implicit in experimental studies where the driver has been instructed to "follow that car" or "follow the lead car at what you consider to be a minimum safe distance at all times" (1). This motivational interpretation of car-following is not valid. In freeway studies, where the driver had freedom of movement, it has been shown that his intention is not to follow but to move ahead toward his destination (5). Genuine car-following might be said to occur in funeral processions, in situations where the driver follows a proceeding car in fog, or in situations where the driver is being guided by someone in front to an unfamiliar address. But even in these cases, it is doubtful that the driver follows at minimum safe distance.

A more operational definition of car-following is given by Eq. 1. The equation states that acceleration of the rear driver is directly proportional to the difference in velocity between the vehicles and inversely proportional to lead distance. The validity of this formulation may be tested by comparison with actual driver reactions. Such a comparison is made in a later section of this study.

That the driver is influenced by factors other than the actions of the car ahead is suggested by several experimental studies. Forbes found that the lag between the second pair of cars in a 3-car queue was much shorter than that between the first pair (3). The rear driver was responding to the first vehicle in the queue rather than to the actions of the vehicle directly ahead. This finding has been confirmed by Michaels and Solomon (10). Forbes also showed that the driver slowed on right curves where visibility is limited, on downgrades, and under conditions of low illumination and limited visibility.

EQUIPMENT AND PROCEDURE

Photographic Procedure

Two 16-mm cameras bolted to the frame in the rear of the vehicle were used to photograph the experimental scene. One camera was aimed at the road behind the plant;

the other was pointed over the shoulder of the driver toward the speedometer. The cameras were activated by solenoids that tripped every 0.47 sec. Analysis of the speedometer photographs showed that plant speed did not vary by more than ±2 mph, hence speedometer photographs were omitted on the last 10 runs.

Experimental Vehicle

The plant was a gray, 1965 Dodge sedan, equipped with a speed governor. The tripod and cameras were positioned in the rear of the vehicle in place of the rear cushions. The experimenters sat in the front and faced forward, away from the subject driver being photographed.

Courses

The experimental course was a long 2-lane, high-traffic road section in the Langley, Virginia, area. A bridge road in the neighborhood was also considered. It was not used because it was too short and complex. A long tunnel might have made an acceptable course; but there is none in the Washington area. The selected course (course A) runs 2.7 miles along Old Dominion Road between Chain Bridge Road and Williamsburg Boulevard (Fig. 1). It begins and ends with a traffic light. A second course (course B) was provided by the opposite lane. As indicated on the map, the courses had intersecting roads and curves but did not have stop lights or stop signs. Despite the inhomogeneities of the course, it was usually possible to maintain a fixed pace along it.

Procedure

At the start of a run, the plant car was parked on the shoulder of the road on the course side of the traffic light. When the signal changed from red to green, the plant

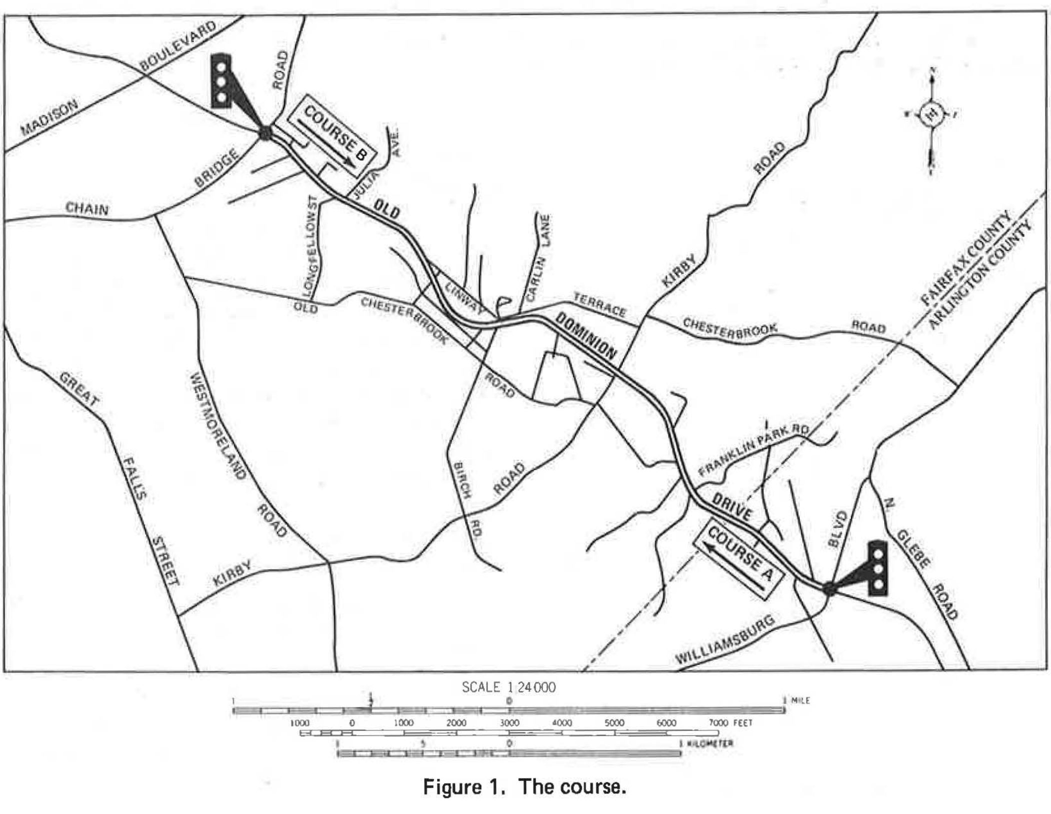

Figure 1. The course.

driver swung into the road and assumed scheduled pace ahead of traffic. A speed of 30 mph was maintainted on the lane from Williamsburg Boulevard to Chain Bridge Road; a 35-mph pace was maintained in the other direction. At slower speeds traffic queued behind the plant, and the driver behind tended to take risks to pass. At faster speeds, the rear driver was left behind.

Ten runs were made on each course. If the photographed car passed, turned off the course, or moved back out of sight, the run was aborted. Another run was then attempted, starting at the beginning of the course.

Analysis of Data

Lead distance was calculated from perspective changes recorded on the films. As the vehicle approached the plant, its angle increased inversely with distance. The relationship is expressed in the formula

$$d = k/\beta \qquad (3)$$

where

- d = distance from focal plane of camera to the front of observed vehicle;
- β = angle of some particular feature on front of vehicle, such as between headlight centers, convenient to measure on its projected screen image; and
- k = constant that depends on size of vehicular feature, focal lengths of camera and projector lenses, and distance from projector to screen.

The approximation to the tangent function in Eq. 3 is justified because the angles of interest are almost all less than 6 deg. The formula was applied by projecting a photograph where distance d from the camera to the vehicle was known. From known d and β, k could be determined. Once k was known, the distance associated with any β could be found. A correction of 7 ft was subtracted to correct for the camera to rear bumper distance of the plant car. The precision of these measurements is a function of the vehicle feature measured, illumination, and camera-vehicle distance. It is believed sufficient to support the study findings.

TABLE 1

VEHICLES OBSERVED IN QUEUE BEHIND SUBJECT VEHICLE

Course	Vehicle	Part of Course Completed		
		One-Third	Two-Thirds	All
A	A-3	3	5	5
	A-5	3	5	6
	B-4	4	2	3
	C-1	2	1	3
	C-2	3	4	4
	C-3	3	1	2
	D-1	3	4	4
	D-3	5	6	7
	D-4	3	2	5
	F-2	2	3	7
	Mean	3.1	3.2	4.6
B	F-1	2	2	2
	F-3	1	0	0
	G-1	1	2	4
	G-2	2	2	1
	G-3	4	4	2
	H-1	2	2	3
	H-2	2	2	1
	H-3	2	3	4
	I-1	1	1	1
	I-2	2	1	1
	Mean	1.9	1.9	1.9

RESULTS

Traffic Environment of Observed Vehicles

Cars tended to line up behind the slowly moving plant. The number of cars seen behind the subject vehicles is given in Table 1. It may be seen that all observed vehicles except F-3 had one or more cars behind. Evidently, the drivers' traffic environment consisted of the plant in front and vehicles behind. It will be noted that longer queues occurred on course A where the plant maintained a faster pace.

Drivers' Reactions in Single-Lane Traffic

When impeded in front, drivers show 3 distinct types of reactions, as evidenced by the records:

1. Avoidance—The driver dropped back out of the influence of the plant vehicle (one driver on each course showed this type of reaction);

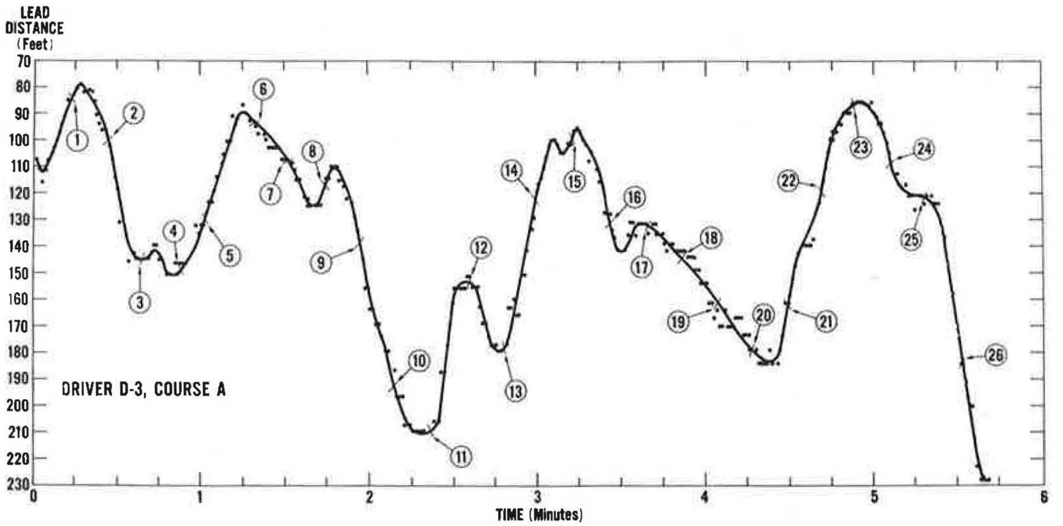

Figure 2. Avoidance reactions of driver D-3.

2. Car-following or pacing—The driver appeared to stay close and roughly match pace with the plant (4 drivers on course A and 2 on course B showed this reaction); and

3. Mixed—Mingled avoidance and car-following (5 drivers on course A and 7 on course B showed mixed reactions).

Avoidance Reactions—Avoidance reactions are shown in Figures 2 and 3. The numbered circles in the figures indicate tenths of miles on the course. Drivers who avoided the plant moved back; but because the plant was moving at a slow pace they tended to

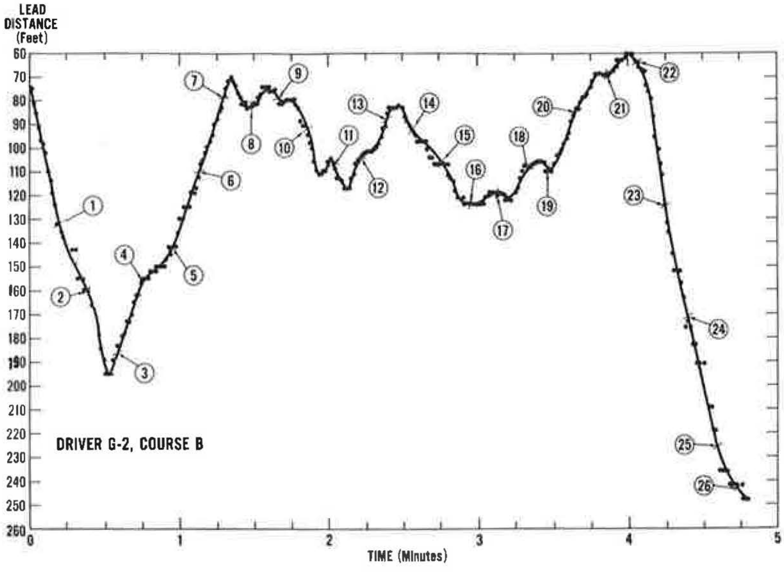

Figure 3. Avoidance reactions of driver G-2.

drift closer again. If a driver moved so far back that he could no longer be seen, the run was discontinued and the record was not analyzed.

The record of driver D-3 (Fig. 2) shows large following distances ranging from 80 to 210 ft. Aside from trying to avoid coming too close, there is little evidence that this driver was trying to maintain any positional relationship with the plant. Driver G-2 also moved out of the influence of the plant car. Lead distances varied from 60 to 195 ft if we exclude the final slowing down at the Williamsburg traffic light. Small adjustive movements are obscured by the large swings away and toward the plant.

On a crowded road, avoidance reactions may result in platooning. Queues of cars would be lined up behind the avoiding vehicle, leaving a space ahead. Platooning may be caused also by reasons other than the desire of the driver to avoid the influence of the vehicle, e. g., by inability to keep up or by engine failure.

Car-Following Reactions (Pacing)—Six of the 20 drivers reacted by pacing or car-following. The behavior is arbitrarily defined here by 3 rules: (a) The driver is in the same lane as the plant; (b) the driver keeps close to the plant (lead distances were always less than 75 ft); and (c) the driver does not show adjustive movements forward or backward of more than 30 ft. The distances found useful in defining car-following might be different under other road conditions. The term "car-following" is used because it is part of the traffic engineering literature; but, it should be understood that these rules are quite different from Herman's car-following equation. In any case, it is not implied that the driver is actively following the car in front.

The record of driver D-1, shown in Figure 4, is an example of car-following. After an initial approach in the first $^3/_{10}$ mile, during which the plant was gaining speed, the record shows continual adjustment of speed and position, at lead distance all closer than 54 ft. In the last $^1/_{10}$ mile of the course, the driver fell back in relation to the plant. Driver H-3 also car-followed (Fig. 5). After a slow approach in the first half mile, the driver paced the plant at a distance that never exceeded 50 ft. There are close approaches at 0.5, 1.4, 2, and 2.5 miles, followed by dropping back. The record is irregular and is not well fitted by a simple mathematical equation.

Mixed Reactions

The majority of drivers (12 of the 20) showed both avoidance and following reactions. Typical mixed reactions are shown in Figures 6 and 7. The record of driver C-1 re-

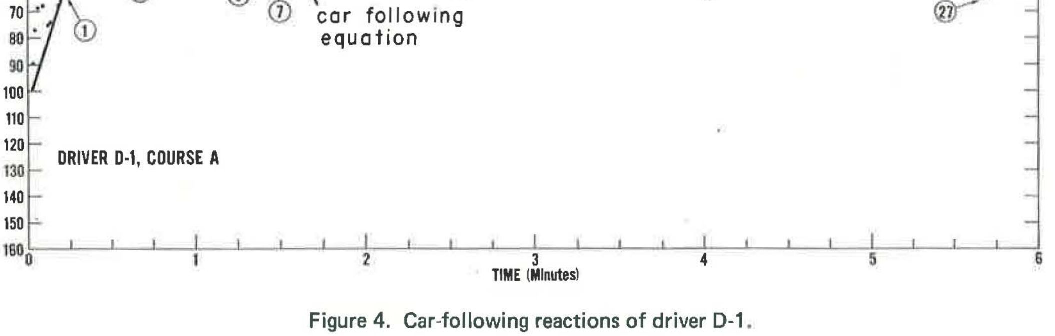

Figure 4. Car-following reactions of driver D-1.

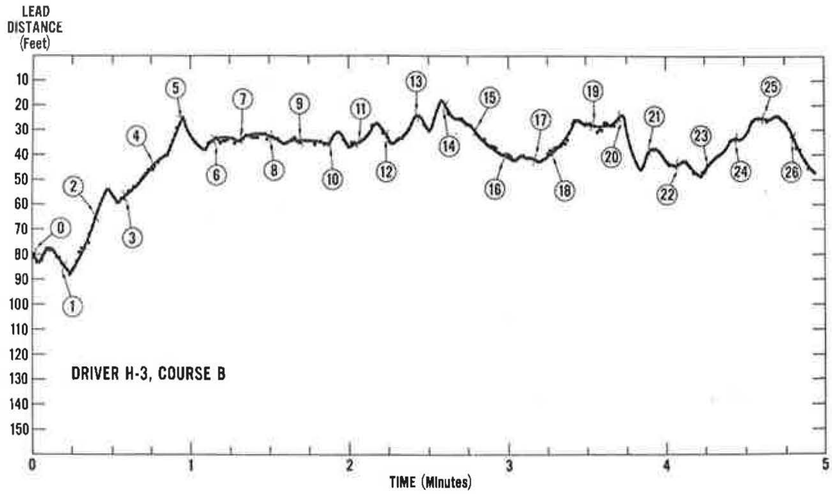

Figure 5. Car-following reactions of driver H-3.

sembles car-following between 0.5 and 1.4 miles, and shows avoidance and drifting up in the next half mile. The car-following (or pacing) portion of the record is at considerably larger distances than those of drivers D-1 and H-3 who car-followed. Another mixed reaction is shown by driver H-1 (Fig. 7). After a slow approach, he dropped back from 41 to 152 ft. The record between 1.5 and 1.8 and between 2.0 and 2.5 miles resembles car-following, although not at very close distance.

Validity of Car-Following Equations

The records of this study are poorly fitted by a car-following equation of the Herman type. Only 7 of the 20 drivers chose to match pace with the plant in front. The others

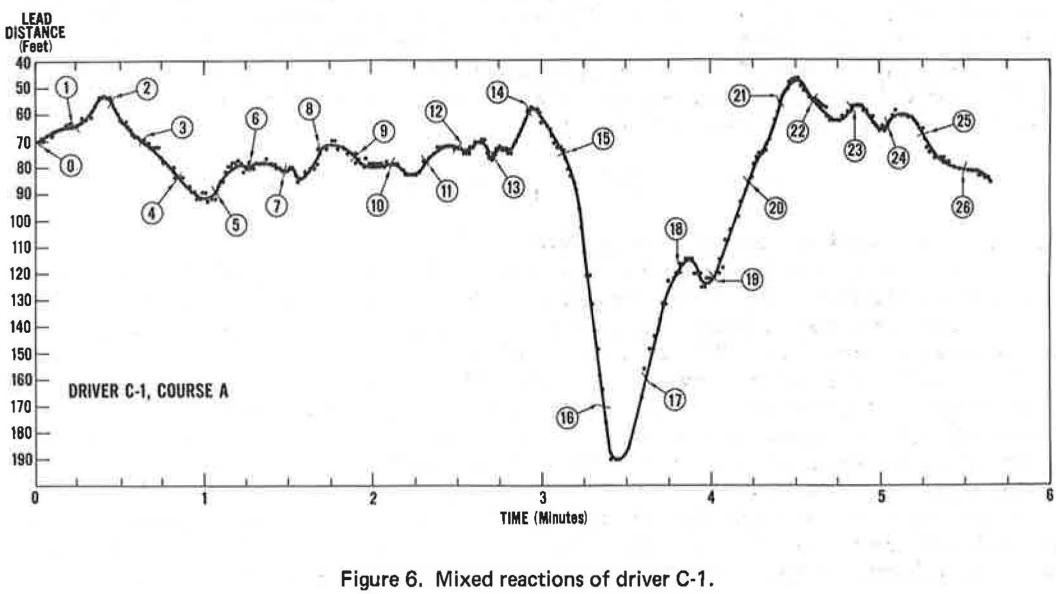

Figure 6. Mixed reactions of driver C-1.

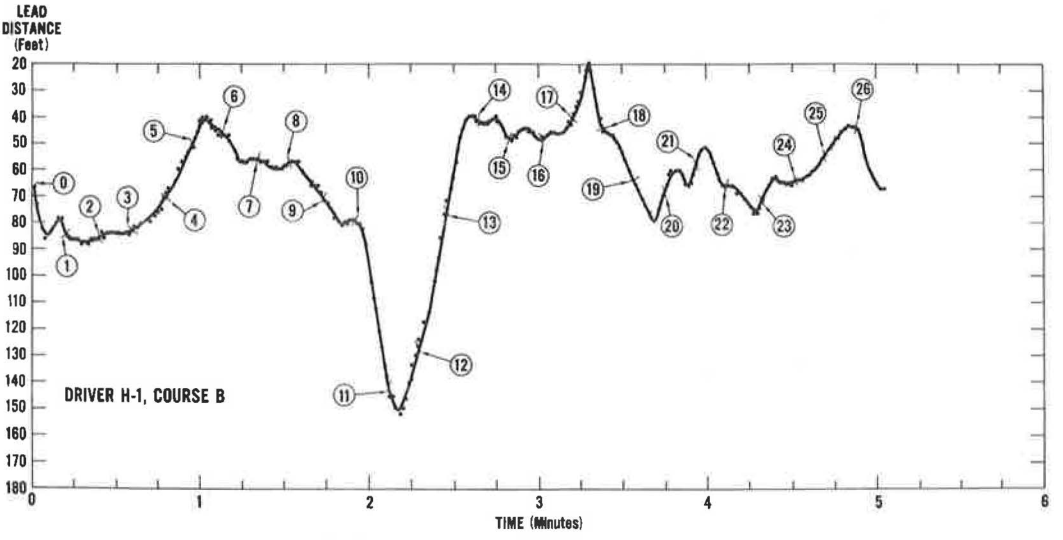

Figure 7. Mixed reactions of driver H-1.

showed avoidance and mixed reactions so irregular that they are not easily matched by an equation.

A comparison of results with the predictions of Herman's car-following equation has been made for the record of driver D-1 who stayed close to the plant vehicle (Fig. 4). A car-following equation of the form

$$(dx/dt)_{n+1} = \alpha_0 \log \left[\frac{x_n(t) - x_{n+1}(t)}{L} \right]$$

was fitted to the data of driver D-1 under the condition that D-1 be 53 ft behind the plant at 17 miles and 40 ft behind the plant $^3/_{10}$ mile farther. These were positions actually assumed by the rear driver. Initial relative velocity was taken as 1.5 fps in the direction of the plant and final velocity at the 1-mile position was taken as zero. It seemed reasonable that the driver has an initial velocity toward the plant; and from the data, it appeared that this driver matched pace at a distance of about 40 ft from the plant. The required equation, solved for α_0 and L is,

$$(dx/dt)_{n+1} = 12.3 \log \left[x_n(t) - x_{n+1}(t)\right] - 19.7$$

The path presented by this equation is shown in Figure 4.

It may be seen that the car-following equation does not give a good fit to the movements of driver D-1. The equation appears to describe the responses to velocity changes of the driver of the car in front rather than in the rear. In the fitted equation, the rear driver cannot come closer than 40 ft to the plant.

The record does not conform to Herman's car-following equation; but it does present some orderly characteristics. The trace consists of a series of small waves or scallops that make up larger movements of advance and retreat from the plant. The driver dithered the accelerator to change distance from the car in front. Each dither served to move the car forward or backward in relation to the plant.

It may be seen that after the initial approach phase, the record consists of a series of back-and-forth movements. The trough of each scallop, from the 0.2- to 0.7-mile position, is below or more distant from the plant than the previous one. This recession

TABLE 2

SIMILARITY OF CAR-FOLLOWING PATTERNS—CHI-SQUARE CALCULATION

Course (1)	Occurrence Frequency: Driver Reaction to Plant (2)	Occurrence Frequency: No. of Tenth-Miles (3)	Percent (4)	Percent Expected by Chance (5)	Difference (Col. 5 - Col. 4) (6)	Difference Squared (7)	Col. 7/Col. 5 (8)
A	4 closer, 0 farther	2	9.1	3.7	-5.4	29.16	7.88
	3 closer, 1 farther	3	13.6	19.1	5.5	30.25	1.58
	2 closer, 2 farther	8	36.4	36.4	0	-	-
	1 closer, 3 farther	6	27.3	30.9	3.6	12.96	0.42
	0 closer, 4 farther	3	13.6	9.9	-3.7	13.69	1.38
Total		22	100.0	100.0			11.26[a]
B	3 closer, 0 farther	2	9.1	10.4	1.3	1.7	0.16
	2 closer, 1 farther	11	50.0	35.1	-14.9	222.0	6.32
	1 closer, 2 farther	3	13.6	39.6	26.0	676.0	17.07
	0 closer, 3 farther	6	27.3	14.9	-12.4	153.8	10.27
Total		22	100.0	100.0			33.82[b]

[a] $11.26 = \Sigma(D^2/Th)$; $\chi^2 < 0.05$ level. [b] $33.82 = \Sigma(D^2/Th)$; $\chi^2 < 0.01$ level.

is followed by a movement forward to 0.8 mile. Lead distance increases again to 1.2 miles and so forth. The bounds of backward-and-forward movement are distinct and without overlap. The same sort of pattern is also found in the other (5) car-following records. The driver appears to dither his accelerator to control a larger oscillation of separation distance from the car in front.

Similarity of Drivers' Reactions

A resemblance was found in the lead distance patterns of drivers who car-followed. Table 2 gives a summary of how drivers who car-followed approached or increased distance from the plant in each tenth-mile from the fourth to the twenty-sixth. (The record of driver G-1, who showed mixed reactions, was also included in the computations on course B.) Drivers' initial approach and final slowing down were not analyzed. There were 4 drivers who car-followed on course A and 3 on course B. It may be seen that, on course A, 5 of the 22 tenth-miles analyzed show all the following drivers making the same reaction (2 in which drivers were moving closer to the plant, 3 in which they were moving farther from the plant). The theoretical expectation is that in only 1.4 (13.6 percent) of the tenth-miles analyzed would this occur. The total chi-square of departure from randomness is beyond the 0.05 level for both groups, indicating that drivers tended to perform similarly. Analyses were made to determine the effects on drivers' approach patterns of road features such as horizontal and vertical curves, passing zones, and intersections. No uniform effects of these features were shown. Probably the effects of curves and the like are superimposed on ongoing approach and receding movements. Moreover the driver tends to anticipate and reacts to road features before they are reached. In the extreme, on steeper hills and sharper curves, some effect would probably have been shown.

DISCUSSION OF RESULTS

The driver's reactions on single-lane roads is understandable in the light of the traffic problem faced. If a driver is impeded and cannot pass, he must accept the situation in one way or another. He can drop back to a comfortable rear position or he can stay close to the car in front to pass when possible. He will not generally press so close as to be damaged if the front driver slows unexpectedly. Nor will he lag so far that he delays or is passed by the driver in back. Within these limits, there is considerable room for individual differences such as found in this study.

From this and other studies (5, 6) the driver emerges as a strategist who makes more or less rational responses that reconcile his need to move ahead with the particular road condition he faces. Considering the wide variety of driver missions and highway conditions, it seems evident that the driver is not successfully modeled by a simple equation or represented as a "black box" element. On the freeway, the driver moves at high speed and does not slow down unless he has to do so (5). On single-lane roads, as shown here, he takes a quite different approach. There is good reason to believe that the driver reacts differently when he is sightseeing, hurrying home from work, bringing an injured person to the hospital, or looking for an unfamiliar address. A simplistic approach to driver modeling appears unwarranted.

SUMMARY AND FINDINGS

A study was made of the reactions of drivers who were impeded and were not aware that they were being observed. The photographic records indicated the distances of the drivers behind the plant car over the 3-mile course.

Three modes of driver response were noted: (a) avoidance, where drivers moved backward out of the influence of the plant, 10 percent of the drivers; (b) car-following, where drivers stayed close to the plant and did not execute large backward or forward movements, 30 percent of the drivers; and (c) a combining of avoidance and car-following, 60 percent of the drivers. Drivers who avoided the plant may have been trying to ease their driving task. Those who followed closely may have been looking for the opportunity to pass.

Driver's lead distance patterns did not conform to Herman's car-following equation. The equation may apply better to the situation where a driver reacts to disturbances introduced by the car in front. The records of drivers who car-followed consist of small waves or scallops that combine into larger movements of approach and recession. The driver appears to dither his accelerator to control a larger oscillation in separation distance from the car in front. There was also a tendency for drivers to approach and move back from the plant at similar parts of the course.

This and previous studies suggest that the driver should be regarded as a strategist who continually adjusts his actions to fit his travel purpose and the road conditions he faces. He is not a stereotyped reacting element or "black box" to be simply described by a fixed equation.

ACKNOWLEDGMENT

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INFLUENCE OF INCIDENTS ON FREEWAY QUALITY OF SERVICE

Merrell E. Goolsby, Wilbur Smith and Associates

Disabled vehicles in moving traffic have a significant effect on freeway operations, particularly during peak periods. This report quantifies, for the Gulf Freeway in Houston, the impact on operations by relating frequency, duration, and flow passing freeway incidents. Data were collected for weekdays only during daylight hours by utilizing the $6\frac{1}{2}$-mile coverage of the Gulf Freeway television surveillance system. During a 2-year period, 1,154 accidents and 1,117 stalls in moving lanes were observed. It was found that the average accident required 19 min from the time of reporting to removal from traffic lanes and an additional 26 min for police investigation. Stalled vehicles were removed within 18 min after being reported to the police. One-minute traffic volumes were measured for normal conditions and at bottlenecks created by incidents. The study section has 3 lanes in each direction of travel with a normal directional peak-period flow of 5,560 vehicles per hour. It was found that incidents created a reduction in flow disproportionate to the physical reduction in roadway width. The average flow rate was 2,750 vehicles per hour with 1 lane blocked by an accident; 2,880, with 1 lane blocked by a stalled vehicle; 4,030, with an accident on the shoulder; and 1,150, with 2 lanes blocked by an accident. Delay for hypothetical morning peak-period incidents are presented to illustrate the magnitude of motorist delay. A stalled vehicle caused a delay of of 1,610 vehicle-hours, while the 1-lane accident and 2-lane accident caused delays of 2,940 and 4,620 vehicle-hours respectively.

•ONE of the greatest losses of efficiency experienced on urban freeways results from disabled vehicles in moving traffic lanes. The congestion and accompanying delay to other freeway vehicles resulting from a reduction in capacity are, in most cases, more significant than the incident that causes the congestion. This paper correlates 2 studies relating to traffic incidents on the Gulf Freeway in Houston. These studies were conducted by the Texas Transportation Institute as part of the research project, Freeway Control and Driver Information Systems, sponsored by the Texas Highway Department in cooperation with the Federal Highway Administration. The studies were concerned with the frequency and characteristics of lane-blocking freeway incidents, and the measurement of traffic flow through the bottlenecks created by freeway traffic incidents. A correlation of the 2 studies gives an indication of the total impact of traffic incidents on freeway operation.

STUDY SITE

The Gulf Freeway in Houston was selected for the studies because of the extensive surveillance system existing there. The Gulf Freeway Surveillance and Control System includes an entrance ramp control system, a data acquisition and control computer, and a 14-camera, closed-circuit television system with video tape recorder. In the $6\frac{1}{2}$-mile study section, the Gulf Freeway cross section consists of six 12-ft lanes and a 4-ft wide median with guardrail.

Sponsored by Committee on Highway Capacity and Quality of Service and presented at the 50th Annual Meeting.

STUDY METHOD

Incident Characteristics Study

An accurate log of freeway incidents was maintained for 2 years (1968-1969) on the $6\frac{1}{2}$-mile section of the Gulf Freeway under television surveillance. A high degree of accuracy in maintenance of the log was possible because of the existence and use of the television system. Police officers assigned to the center maintained the log on weekdays between 6:00 a. m. and 6:00 p. m. When an incident occurred, and during its handling and clearance, the officer made appropriate entries in the log of time, events, and conditions (Fig. 1). If police action was required at the scene for incidents such as accidents and vehicles stalled in traffic lanes, the officer used a base station radio to report the incident to the police dispatcher. An accurate time trace of events was then available for all incidents occurring during daylight hours on weekdays. Data from the incident logs for 1968 and 1969 were coded, keypunched, and analyzed by digital computer.

Flow Study

This study measured traffic flow through the bottleneck created by an incident. Because the freeway incident is an unscheduled and essentially random event, the collection of data in similar studies has been difficult, if not infeasible (1). The television surveillance system, equipped with a video tape recorder, provided a means of studying an incident almost from the time of occurrence. When an incident was observed, the appropriate remotely controlled television camera was positioned on the scene, and the video tape recorder was switched on. Normally the recorder was on within 2 to 3 min after the incident occurred. The incident scene was recorded while the vehicles were in moving lanes and, in the case of accidents, while the accident remained on the freeway shoulder during police investigation. It was then possible to manually transcribe flow data from the video recording at a later time. These volume counts taken from the video recording were summarized for 1-min periods. In order that flow through

HOUSTON POLICE DEPARTMENT
GULF FREEWAY TRAFFIC - SURVEILLANCE

DATE June 18, 1968 DAY OF WEEK Tuesday

OFFICER R. G. Gartman HOURS 6:30-3 OFFICER E. E. Ellis HOURS 3^{00}-6^{30} OFFICER ______ HOURS ______

TYPE INCIDENT	LOCATION & DIR. OF TRVL.	LANES INVOLVED	LANES BLOCKED	TIME OF DAY: OBSERVED	UNIT ARRIVED	OBSTRUCTION REMOVED	TOTAL TIME OBSTRUCTED	TRAFFIC NORMAL	INVEST. COMPLETED	WEATHER CONDITIONS	REMARKS
Stall Truck	IB Griggs Ramp	Ramp	Ramp	7^{40} AM	8^{01} AM	9^{02} AM	82 min.	9^{05} AM	9^{05} AM	Dry/Clear	Mech. Prob.
Minor Accident	IB Griggs	#1	#1	8^{35} AM	8^{39} AM	8^{45} AM	10 min.	9^{12} AM	9^{26} AM	Dry/Clear	4 veh. Rear End
Minor Accident	OB Lombardy	#1	#1	10^{50} AM	—	10^{57} AM	7 min.	10^{59} AM	10^{59} AM	Dry/Clear	2 veh. Left scene
Minor Accident	OB Wayside	#1	#1	2^{20} PM	2:27 PM	2^{31} PM	11 min.	2^{55} PM	3^{12} PM	Dry/Clear	3 veh. Guard rail
Minor Accident	OB Lombardy	#3	#3	2^{58} PM	3^{00} PM	3^{00} PM	2 min.	3^{08} PM	3^{31} PM	Dry/Clear	Rear End

Form No. 18259 7/68

Figure 1. Daily incident log for Gulf Freeway Surveillance and Control Center.

TABLE 1

INCIDENT SUMMARY FOR 1968-1969 ON WEEKDAYS FROM 6:00 a. m. TO 6:00 p. m.

Incident	Frequency	Percent
Stalls	1,117	47.6
Injury accidents	63	2.7
Noninjury accidents	1,091	46.6
Lost load	37	1.6
Other	35	1.5

TABLE 2

LANES BLOCKED BY INCIDENTS

Lane Blocked	Stalls		Accidents	
	Number	Percent	Number	Percent
Outside	432	38.7	244	21.2
Center	231	20.7	204	17.7
Median	299	26.8	284	24.6
Two lanes	8	0.7	111	9.6
Three lanes	0	0.0	22	1.9
Ramp	134	12.0	238	20.6
Other	13	1.1	51	4.4

the bottleneck would be a reflection of capacity rather than demand, volume counts were made only when a queue of unsatisfied demand existed upstream from the incident.

DATA ANALYSIS

Freeway Incident Log

Results of an analysis of the incident log for a 2-year period are presented in this section. The magnitude of the disabled vehicle problem is given in Table 1. It should be remembered that the log was maintained on weekdays from 6:00 a. m. to 6:00 p. m. and does not include night or weekend incidents on the Gulf Freeway. Lane-blocking accidents and stalls are approximately equal in frequency. Approximately 4.5 lane-blocking incidents occur each weekday during daylight hours.

The number of lanes blocked by stalls and accidents is given in Table 2. A greater proportion of stalls occur in the outside lane, whereas accidents are more uniformly distributed across the 3 lanes and ramps.

The average noninjury accident directly affects traffic for approximately 45 min. The sequence of events of a typical accident with average duration of each step is as follows: (a) detection and reporting of accident to police, estimated to be 1.0 min by using television; (b) location, dispatch, and travel by police to scene unit, 12.0 min; (c) clearing of accident vehicles from traveled lanes, 7.0 min; and investigation by police, 25.6 min.

Statistical data for these elapsed times are given in Table 3. Accident data were not stratified for weather conditions, accident severity, police work load, or other factors that influence accident removal. These factors contributed to the relatively high variances given in Table 3. Data for stalled vehicles were treated in a similar manner, and the results are also given in Table 3.

Flow Study

Bottleneck flow data were available for 27 incidents that yielded 1-min volume counts for a total of 517 min. Flow data were classified into 6 categories of incidents for analysis: 1a, noninjury accidents blocking outside lane only; 1b, noninjury accidents blocking center lane; 1c, noninjury accidents blocking median lane; 2, stalled vehicle blocking 1 lane; 3, accident blocking 2 lanes; and 4, accident removed to shoulder. In

TABLE 3

ELAPSED TIME STATISTICAL DATA FOR ACCIDENTS AND STALLS

Time Interval	Sample Size	Mean Time (min)	Standard Deviation
Accidents			
Observed to police arrival	810	12.0	10.8
Police arrival to accident removal	597	7.0	11.9
Removal to investigation complete	903	25.6	19.1
Stalls			
Observed to police arrival	314	9.4	9.8
Police arrival to stall removed	285	8.9	14.5

TABLE 4

SUMMARY OF FLOW DATA AT FREEWAY INCIDENTS

Condition	Sample Size (no. of min)	Range (veh/min)	Average Flow (veh/min)	Standard Deviation (veh/min)	Average Flow Rate (veh/hour)
Normal flow	312	70 to 108	92.6	6.3	5,560
Noninjury accident, 1 lane blocked					
Outside lane	46	28 to 75	48.1	11.7	2,880
Center lane	42	18 to 77	42.7	11.4	2,560
Median lane	79	21 to 77	46.1	8.8	2,770
Combined	167	18 to 77	45.8	10.5	2,750
Stalled, 1 lane blocked	43	30 to 77	47.9	8.6	2,880
Accident, 2 lanes blocked	53	4 to 39	19.1	7.3	1,150
Accident on shoulder	254	20 to 102	67.1	13.1	4,030

TABLE 5

RESULTS OF F- AND t-TESTS FOR 1-LANE BLOCKAGES

Hypothesis[a]	F-Test			t-Test		
	F	$F_{0.95}$	Result	t	$t_{0.95}$	Result
Outside-center lane accident	1.05	1.47	Accept	2.07	1.99	Reject
Outside-median lane accident	1.80	1.53	Reject	1.12	1.98	Accept
Median-center lane accident	1.72	1.52	Reject	1.88	1.98	Accept
Any lane accident-any lane stall	1.46	1.55	Accept	1.23	1.96	Accept

[a]Flows passing the incidents are equal when lanes indicated are blocked.

addition to the incidents, 312 min of normal flow were measured downstream of the inbound Telephone Road entrance ramp (a geometric bottleneck) to provide a frame-of-reference service volume for the incident flow data collected. Table 4 gives the statistical data for each condition.

A 1-lane blockage by a minor accident or stall reduces flow by 50 percent, even though the physical reduction is only 33 percent (Table 4). The presence of an accident on the freeway shoulder reduces flow by 33 percent of normal flow because of the effect of the "gapers-block" phenomenon. An accident that blocks 2 lanes reduces flow by 79 percent (compared to 67 percent reduction of available freeway width). Thus, freeway incidents create a reduction in capacity that is disproportionate to the physical reduction of the facility.

Data indicated the possibility that flow past a 1-lane accident was independent of the lane blocked by the accident. To test this hypothesis F- and t-tests were conducted on paired data by lane (Table 5). This hypothesis was rejected by one or both of the tests at the 95 percent confidence level for all 3 pairs, indicating that it does make a difference which lane is blocked. The data for accidents blocking 1 land and for stalls were combined, and the hypothesis was tested that there was no significant difference in flow between accidents and stalls. This hypothesis was accepted at the 95 percent confidence level.

DISCUSSION OF RESULTS

The studies have defined, in operational terms, lane-blocking incidents (1-lane accidents, 2-lane accidents, and stalls) as well as the effect on flow of an accident removed to the freeway shoulder. A plot of cumulative volume versus time (2) was used to illustrate the time-flow-delay relationships.

Illustration of the effect of a 1-lane accident is shown in Figure 2 by using peak-period demand data for the inbound Gulf Freeway at Telephone Road. The slope of the service volume curve was derived from the average flow determined in the flow studies.

TABLE 6

DISTRIBUTION OF INCIDENTS BY TIME OF DAY ON WEEKDAYS

Hour Ending	Stalls (percent)	Accidents (percent)	Hour Ending	Stalls (percent)	Accidents (percent)
7:00 a. m.	4.4	3.8	1:00 p. m.	4.8	5.0
8:00 a. m.	13.8	11.7	2:00 p. m.	4.4	6.6
9:00 a. m.	8.5	10.1	3:00 p. m.	4.7	7.0
10:00 a. m.	3.3	3.9	4:00 p. m.	8.2	10.6
11:00 a. m.	3.2	6.0	5:00 p. m.	16.9	16.9
12:00 noon	4.7	5.2	6:00 p. m.	23.1	13.3

The area between the demand and service volume curves is an aggregate delay. Elapsed times for police incident-handling shown on the abscissa are averages obtained in the analysis of incident logs. It should be pointed out that these elapsed times are characteristic of the operation of a single police department only and no attempt was made to compare them with those in other cities. The time between occurrence of an incident and reporting it to the police was very short in this study because a police officer in the control center observed and reported the incident before the motorists reported it. Reporting time by motorists has been found to be on the order of 5 min (3).

For the 1-lane accident shown in Figure 2, delay to other motorists is 2,940 vehicle-hours. Similar computations for the other 2 incident types yield delays of 4,620 vehicle-hours for a 2-lane accident and 1,610 vehicle-hours for a stall. These examples are extreme, although not unrealistic, because they represent peak-period incidents. Incidents occur throughout the day (Table 6), and delay from them is highly variable, depending on demand. In addition, the estimated delay is liberal because no diversion of demand from the freeway was assumed. It has been observed that diversion of demand from the freeway to arterial streets frequently occurs when freeway level of service deteriorates below normal.

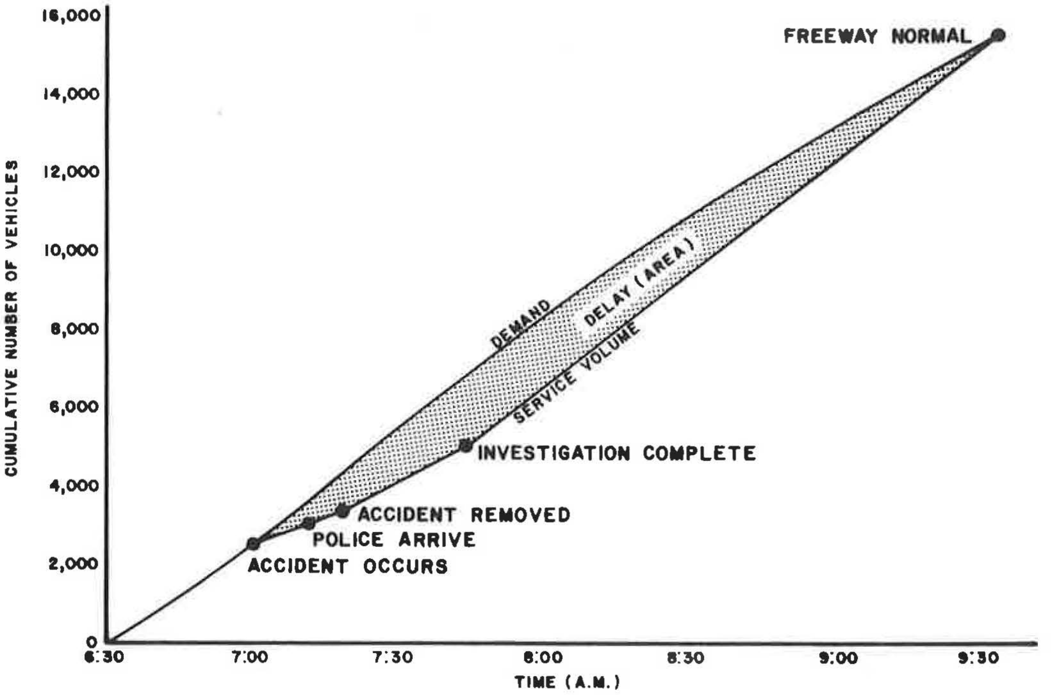

Figure 2. Example of time-flow delay relationships for an accident blocking one lane.

Operational procedures that reduce the time a freeway incident affects traffic will lead to substantial reductions in motorist delay. For example, a reduction of 2 min in the time required for accident detection or police response results in motorist savings of 411 vehicle-hours for the 1-lane accident shown in Figure 2.

SUMMARY

This report has quantified the frequency of incidents and illustrated the magnitude of motorist delay resulting from them on the Gulf Freeway. Even the most insignificant incidents that momentarily block traffic lanes may have a profound influence on freeway operations. An accident or stall that blocks 1 lane causes a reduction in capacity of 50 percent, even though the physical reduction is only 33 percent. The gapers-block phenomenon reduces capacity 33 percent, even though no physical obstruction exists.

Softening the impact of incidents can be accomplished in several ways: (a) rapid detection methods such as police patrols, surveillance systems, and motorist-aid systems; (b) reduction of reaction and response times of servicing agencies; and (c) streamlined handling and clearance procedures such as improved wrecker service, motorist removal of minor accidents, and accident investigation away from the freeway (4). Every means for reducing the frequency and impact of freeway incidents must be explored if freeways are to function as the high type of facility they were designed to be.

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AN ANALYSIS OF DIAMOND INTERCHANGE SIGNALIZATION

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This paper presents a systematic approach to the signalization of diamond interchanges. Attention is directed so that the resulting signalization concepts are applicable to the real-time control of diamond interchanges, which is the ultimate objective of this project. This paper provides a critical review of the existing practices for diamond interchanges. One of the principal products of this study is the identification of all the basic elements that constitute a phasing pattern. Each of the phasing patterns is then described by a set of parameters that have one-to-one correspondence with the signal-controller adjustments. These basic phasing parameters are used as aids to develop and synthesize the various possible phasing patterns. After each phasing pattern developed is evaluated, 2 preferred sets of phasing patterns are identified and a typical diamond interchange phasing pattern from the field is described. Applicability of these phasing concepts to the real-time control of diamond interchanges is also discussed. Finally, operational characteristics of full diamond interchanges from signalization data available for the city of Los Angeles are presented.

•THE GOAL of this research study was to determine an objective way of signalizing a diamond interchange in order to obtain high levels of operational performance. Attention was directed not only to fixed signalization concepts but also to concepts applicable to the real-time control of diamond intersections, which is a long-term objective of a broader research program on signalized diamond interchanges currently under way.

There are approximately 100 full diamond interchanges and another 100 partial diamond interchanges in the Los Angeles metropolitan area, about half of which are signalized. This sample gives an indication of the national incidence of this type of interchange in urbanized areas and emphasizes the importance of providing good signal control at such facilities. However, there seems to be anything but full agreement regarding the best way to signalize a diamond interchange. This is probably due to the lack of a methodology to address the problem, although we note that significant attempts have been made to structure such a framework.

In performing this study, we made an investigation of the principal and distinctive characteristics of phasing patterns for signalized diamond interchanges. A systematic review was made of the pertinent literature, and an operational study was conducted on signalized diamond interchanges in order to identify major signalization parameters and to determine their degree of correspondence with the signal-controller adjustable variables. With the identification of all these basic elements, any possible phasing pattern can be synthesized.

The Los Angeles Department of Traffic, generally considered a user of good signalization practice, provided signal timing and other pertinent information for local signalized diamond interchanges. This set of data has formed the basis for a detailed investigation into phasing patterns. Attention was limited to full diamonds signalized at both ramp intersections, which is, of course, the most difficult case.

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DEFINITIONS

The following definitions are used in this paper.

A phase is defined as a distinct state of a traffic signal or set of signals, where distinct signal light states (green, red, or amber) are attached to each direction of flow. Traffic engineers use the term "phase" in 2 distinct ways in describing each different state of traffic movement when referring to (a) a phasing pattern of a diamond interchange complex or (b) the signal activation of distinct traffic movements at an individual intersection. However, the meaning of the term is believed to be clear in each instance that it is used in this paper.

A traffic movement is the distinct entry or exit mode of vehicles that requires a green traffic signal at an intersection.

A phasing pattern is a combination of different phases, including all the possible traffic movements of the intersection or set of intersections (diamond interchange) where any traffic movement can extend for more than one consecutive phase but no traffic movement is generally repeated twice through a separation of amber and red lights within a given cycle.

BASIC TYPES OF CURRENT PHASING PATTERNS

Of the various possible phasing patterns or phasing sequence (1) types, 3 phasing patterns are most frequently cited: Two are 3-phase patterns, and one is a 4-phase pattern with 2 overlaps. Figures 1 and 2 show the 2 kinds of 3-phase phasing patterns. Figure 3 shows the 4-phase phasing pattern with 2 overlaps, and Figure 4 shows the 4-phase pattern with no overlaps.

In fact, there are only 3 distinct phases at each diamond ramp intersection, making a total of 9 distinct possible phases, called the 9 basic phases, for the 2-ramp intersections of a diamond interchange. Detailed descriptions of the 9 basic phases are given in the following and shown in Figure 7. For purposes of comparison, the 9 basic phases are also shown in Figures 1 through 5. The WALK, flashing DON'T WALK (WAIT), and steady DON'T WALK (WAIT) phases for pedestrians, as well as the amber phases for motorists, have been excluded from these and all of the phasing patterns discussed in this paper. However, these movements can be accounted for by the application of the same principles.

Three-Phase Phasing Pattern

The 3-phase phasing pattern shown in Figure 1 seems to give preference to the straight-through traffic on the arterial street. The left-turning vehicles on the arterial

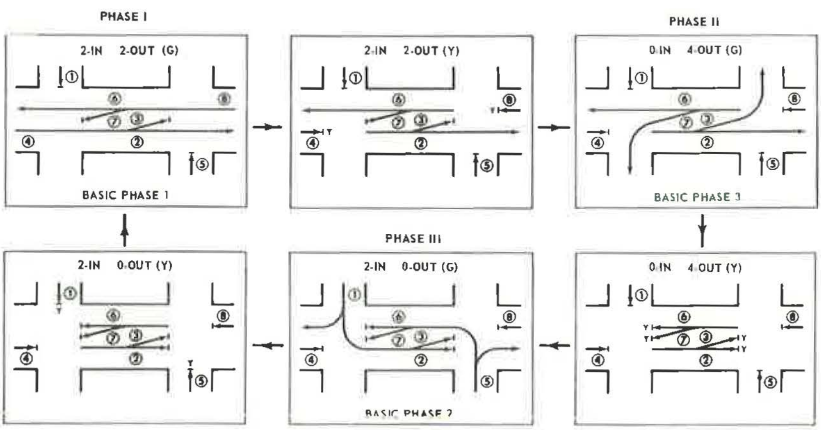


Figure 1. Three-phase phasing pattern, fully symmetrical.

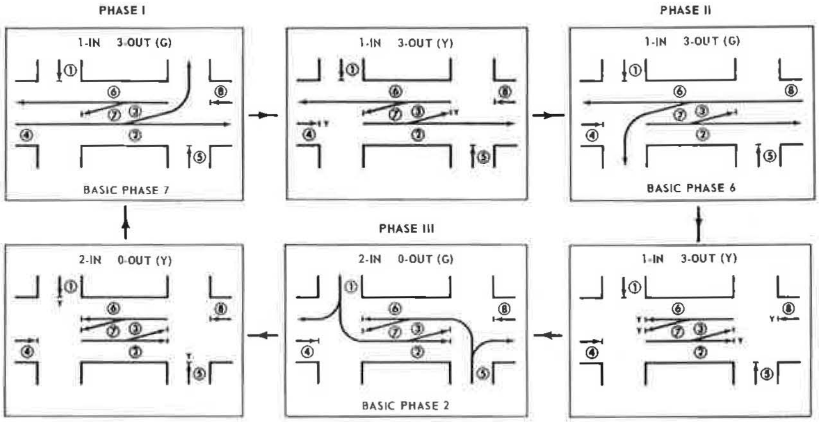


Figure 2. Three-phase phasing pattern, partly symmetrical.

are stored during Phase I and only begin to clear in Phase II when the incoming traffic from the arterial to the diamond is stopped. During Phase II, the stored left-turn vehicles are cleared. The number of such vehicles is limited by the storage capacity of the diamond. During Phase III, the left-turning off-ramp traffic is again limited by the storage capacity of the diamond. Note that this type of phasing pattern is symmetrical for both directions of traffic movement; e. g., in Phase I, the 2 straight-through traffic movements are given the same green time. This type of phasing pattern seems suitable where there is heavy traffic movement on the straight-through arterial street in both directions and where there is very little traffic coming from the freeway through the off-ramps and entering the freeway from the arterial street through the on-ramps.

The second kind of 3-phase phasing pattern shown in Figure 2 also has limited storage capacity for the off-ramp traffic (Phase III). During Phase I, the initial green

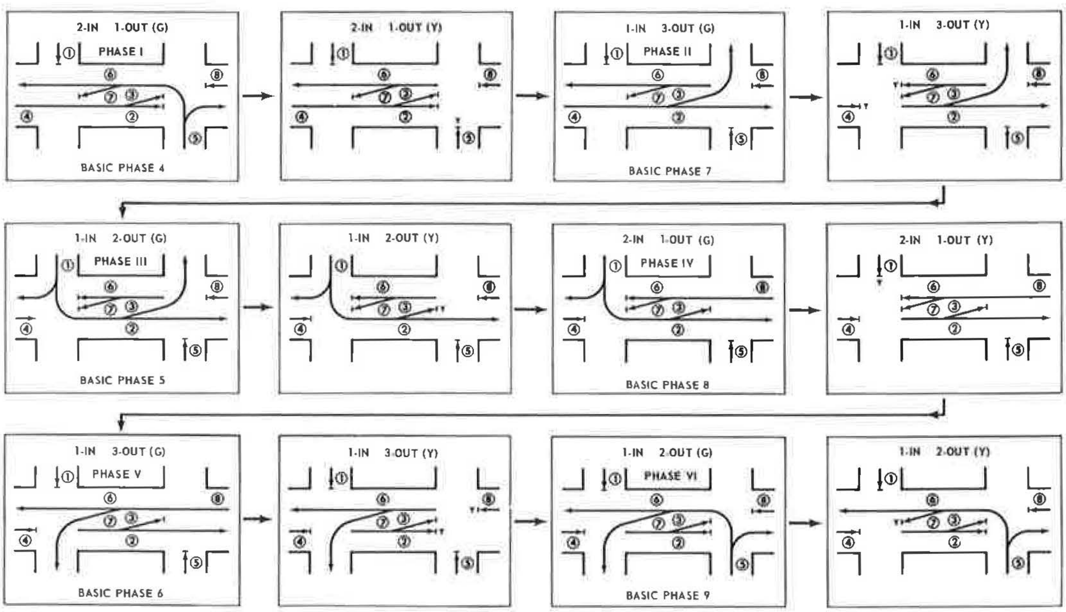


Figure 3. Four-phase phasing pattern with 2 overlaps.

period for movement 4 will be of little use because it will still take some time before the last vehicle in the interior approach queue starts moving because of the response times of all the vehicles in front of it. At the same time, vehicles will be discharged in movement 6 during this time period without any input of vehicles. During Phase II, when the green signal is given for movement 8, the interchange will be practically empty. During Phase III, cars are not completely cleared in the internal movements 6 and 7 of the arterial traffic of Phase II, before the off-ramp traffic starts moving inside the diamond interchange. It may again be observed from this phasing pattern that the green times allocated for different movements are symmetrical in each phase. For example, the green times for movements 4 and 3 in Phase I are the same. Pinnell and Capelle (2) have pointed out that clearance phases should be added following Phases I and II to clear the interior approaches for storage of the off-ramp traffic coming from the freeway in Phase III. As shown in Figure 5, they used this type of phasing pattern in their operational study of signalized interchanges in the Berry I study.

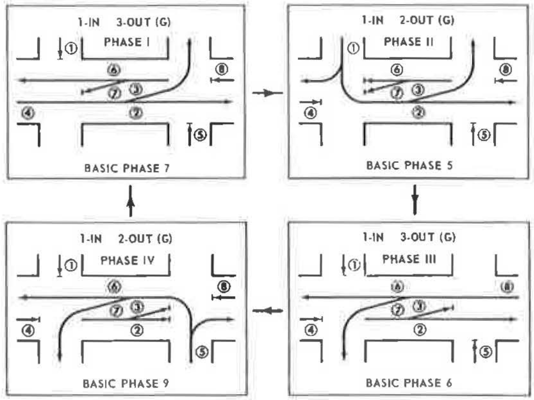


Figure 4. Four-phase phasing pattern with no overlaps.

Four-Phase Phasing Pattern With Overlap

The other phasing pattern widely discussed in the literature is the 4-phase with overlap shown in Figure 3. As the name implies, 2 movements that would normally be conflicting without the spacing between the 2 intersections of the diamond are given a green signal simultaneously. Phases I and IV are the overlap phases (Fig. 3). The main features of this type of phasing pattern are the nearly continuous movement of straight-through traffic between the 2 intersections of the diamond and the virtual elimination of stopped vehicles in the interior approaches. However, the lengths of the overlap phases are very critical to the efficient operation of this scheme. The overlap phases can operate effectively only if the traffic entering the interchange from both the arterial approach and an off-ramp approach maintain a continuous progression without running into the red signal at the other intersection or encountering waiting vehicles between the intersections.

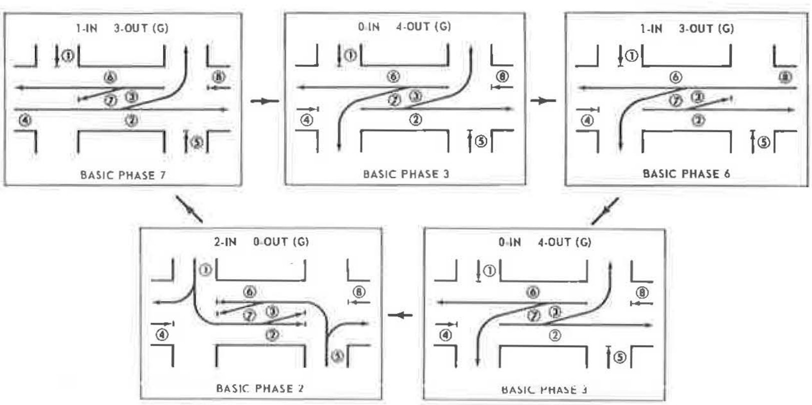


Figure 5. Three-phase phasing pattern with 2 clearance phases.

Four-Phase Phasing Pattern Without Overlap

Figure 4 shows the 4-phase phasing pattern with no overlap phases used by Pinnell and Capelle (2) in their Berry II study of the diamond interchange. Comparison of this phasing pattern with the 4-phase-with-overlap phasing pattern reveals some interesting advantages in the latter. For example, Figure 4 shows that, in Phase I of the 4-phase phasing pattern, the signals for movements 4, 2, and 3 turn green simultaneously at the beginning of the phase. The green signals for movements 2 and 3 produce little advantage to the traffic movement in the early seconds of this phase inasmuch as there are no vehicles to be served because the interior space of the diamond was cleared earlier in Phases II and III. In the 4-phase-overlap phasing pattern this time period is allotted to the overlap phase, where traffic signals for movements 2 and 3 stay red and the off-ramp movement 5 continues green.

MERITS OF DIFFERENT PHASING PATTERNS

Critique on Limitations of 4-Phase-Overlap Signalization

Certain contradictory statements in publications on diamond signalization may be added to the differences of opinion over the choice of phasing patterns. As an example, Woods (3) points out some limitations of the 4-phase-with-overlap phasing pattern. Regarding the first limitation, Woods comments, "The four-phase signal system illustrated in Figure 1 can only show an advantage over a three-phase system if the amount of overlap is greater than the time lost in the added clearance phase and starting delay." (Figure 1 in the quote refers to 4-phase with 2 overlaps and is equivalent to Figure 3 in this paper.)

A closer inspection of the 4-phase-overlap and the 3-phase phasing patterns shown in Figures 3, 1, and 2 reveals that there is no added lost time in the clearance phase or the starting delay in the overlap phase. In every cycle each movement in these phasing patterns is given only 1 green signal, followed by an amber period. Because the 8 movements shown in the various phasing patterns are given only 1 continuous green signal every cycle, the vehicles corresponding to each of these movements experience only 1 starting delay; similarly, only 8 clearance intervals correspond to these 8 traffic movements. In the 3-phase phasing pattern these amber signals for the 8 movements occur in the 3 clearance phases, while in the 4-phase-overlap signalization they occur in 6 clearance phases. However, the lost time in clearing the traffic in all these phasing patterns remains unchanged. As a matter of fact, as pointed out by Spitz (4), the starting delay for movements 2 and 6 is effectively reduced to a minimum for the 4-phase-overlap signalization.

Critique on Cycle Length and Capacity of Diamond Interchange

Another statement made by Woods (3) refers to the cycle length and the capacity of the diamond interchange. Woods concludes by commenting, "... since the system is more than 100 percent efficient, the more cycles that can be completed in a given time period, the greater the capacity of the system, i.e., use of very short cycles is desirable." He builds this argument by starting, "The inclusion of an overlap phase means that a period of time equal to the length of the overlap is available to offset the time lost during the amber phases." He then gives the following relationship for total effective green time for the 4-phase-with-overlap phasing pattern: "Effective green time = cycle length - 4 (length of amber phase) + 2 (length of the overlap)." Length of the overlap takes into account the time periods of the green and amber phases associated with the overlap phase. Referring to these relationships, Woods states, "... when the overlap time is equal to twice the length of the amber phase the effective green time is equal to the cycle length. Increasing the overlap to more than twice the length of the amber phase results in an effective green time greater than the length of the cycle, i.e., more than 100 % efficient."

From this he concludes that the system is more than 100 percent efficient, and the more cycles that can be completed per given time period, the greater the capacity will be. A closer look at the effective green time indicates that it is the sum of the green time for movements 1, 4, 5, and 8. Note that many phasing patterns other than that considered by Woods will have green time for movements 1, 4, 5, and 8 greater than the cycle length and, according to Woods' arguments, would be more than 100 percent efficient. Hence, this type of analysis of phasing patterns might be misleading.

Actually, the capacity of the diamond interchange for any phasing pattern will increase as the cycle length is increased. This has been acknowledged by Ridgeway (5). Use of the minimum cycle length, however, is recommended for different reasons. Webster (6) and others have indicated that the minimum cycle length that will handle the traffic volumes will minimize the delay.

Critique on Coordination of Diamond Intersection Signals With Nearby Arterial Signals

Another feature of diamond interchange signalization is the relative value of coordination of the diamond intersections with signals of the nearby intersections of arterial streets. Pinnell and Capelle (2) note the desirability of maintaining the progression of the through-traffic but cite the diamond interchange area as a bad timing point in the coordinated system because the signal system at the interchange must operate in a multiphase sequence. They further argue, "The through-traffic for which progression is desired represents a minor percentage of the total traffic entering the interchange area."

This through-traffic was approximately 25 percent of the total interchange traffic when they performed the analysis of volume counts for the Berry Street interchange in Fort Worth and the Cullen and Wayside interchanges in Houston. They conclude by remarking, "Therefore, efficient operation of the entire interchange system should receive more priority than that of providing progression for the through traffic on the major street [arterial street]."

However, others tend to disagree with these remarks. For example, Hutchison (7), of the Los Angeles Department of Traffic, remarks, "It is an absolute must that these adjacent intersections be controlled at peak efficiency and be interconnected to the controls at the freeway ramps." Skiles (8), also of the Los Angeles Department of Traffic, points out that most of the diamond interchanges in Los Angeles are located near a signal-controlled arterial intersection at distances which vary from 200 to 1,000 ft. Under these spacing conditions, he points out that coordination with adjacent signals is essential and further comments,

> Usually, coordination approaching the interchange has been the most critical because of the effect of starting delay at the beginning of the input green interval. . . . Unfortunately, as intersection spacing reduces, effective coordination becomes increasingly difficult. . . . in most applications, the importance of signal coordination is not to provide coordinated flow through the interchange, although that may be a secondary benefit. The value is in improving overall interchange operation by improving the flow into and out of the interchange.

Eckhardt (9), of the California Division of Highways, comments, "Coordination is of little or no value unless traffic can be handled. If a lack of capacity results in an accumulated backup of traffic, then the value of coordination is lost. If efficiency at the diamond intersections is sacrificed in an attempt to maintain coordination, . . . then no reason for coordination exists."

Similar opinions exist in favor of and against the coordination of diamond intersections with arterial streets. Generally, people working for the city tend to favor the coordination, while those responsible for freeway systems support effective handling of traffic from the off- and on-ramps by the diamond interchange. The usual conclusion is that greater attention should be given to coordination of diamond intersections with the nearby arterial intersection where the traffic volume from the arterial street constitutes a large percentage of the total traffic feeding the diamond.

Until now we have talked about various phasing patterns where the traffic volumes in the 2 through-directions, the 2 off-ramps, and the 2 left-turn lanes are of approximately the same magnitude. In real-life, variations exist that along with other considerations of the diamond, can result in several phasing patterns.

BASIC SIGNAL-PHASING CONCEPTS OF DIAMOND INTERCHANGES

As stated earlier and shown in Figures 1 through 5, there are only 9 distinct basic signal phases. We will now examine the basis that constitutes a phase. We will then show that there are, indeed, 9 basic phases at the diamond interchange that form the basis for all the phasing patterns and that these constitute an exhaustive set.

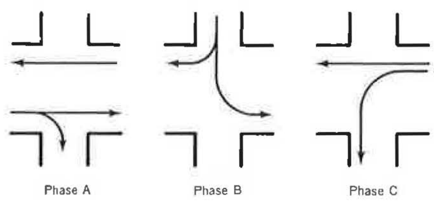


Figure 6. Three basic phases at individual ramp intersections.

Three Basic Phases at Individual Ramp Intersection

Let us now look at the left-ramp intersection of the diamond interchange shown in Figure 6 and see how many different signal phases this intersection can have where there will be no conflicts between movements. One phase at this intersection would exist when the off-ramp and the left-turn traffic from the arterial is stopped and the straight-through traffic is moving. We call this Phase A, as shown in Figure 6. Another phase results when the traffic from the off-ramp is given a green signal. To do this we have to stop all other movements at this intersection. We call this Phase B. The other phase occurs when arterial left-turn traffic is given a green signal. To obtain this, we have to stop all the incoming conflicting arterial traffic that may feed the diamond at this intersection. This is called Phase C. We can readily conclude that there are no additional phases at this intersection. In addition, there are only 3 similar basic phases at the right-ramp intersection of the interchange; these form the basis for all different possible phasing patterns. Furthermore, it follows that these 3 phases at the individual diamond intersections, in different combination, produce the 9 basic phases for the configuration covering both diamond intersections.

Nine Basic Phases at Diamond Interchange

Juxtaposition of the 3 basic phases of one intersection with the similar counterpart phases of the other intersection gives the 3 symmetrical phases designated as basic

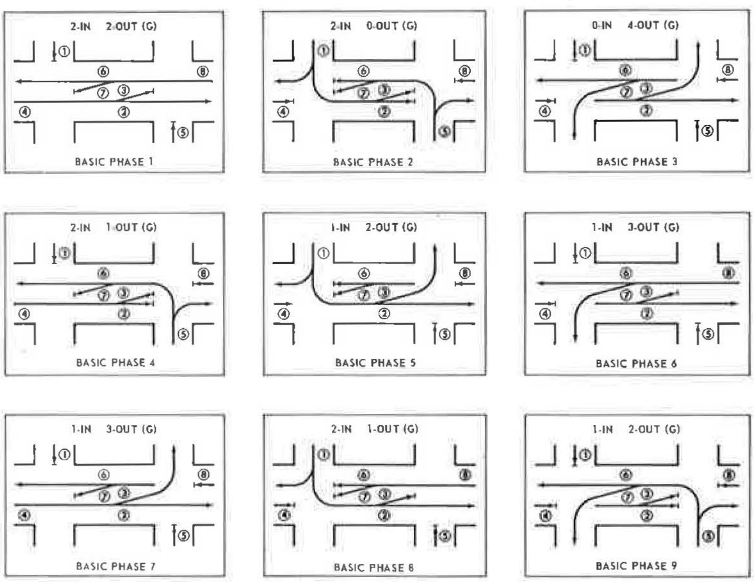


Figure 7. Nine basic phases.

phases 1, 2, and 3 and shown in Figure 7. We refer to these phases as the symmetrical phases because the traffic movements at one intersection are similar to those at the other intersection. Note that these phases constitute the 3-phase phasing pattern shown in Figure 1.

Again, corresponding to each of the 3 phases at one intersection are 2 dissimilar phases at the other intersection. Juxtaposition of these phases will give 2 sets of 3 asymmetrical phases. They are designated as basic phases 4, 5, 6, 7, 8, and 9 and are shown in Figure 7. The combination of these phases is arranged to constitute the 4-phase-overlap phasing pattern.

Thus, we have 3 basic phases at each ramp intersection that together constitute the 9 basic phases for the diamond interchange and that form the basis for all possible phasing patterns. We will now examine the function of each of these basic phases.

Characteristics of 9 Basic Phases at Diamond Interchange

As Figure 7 shows, we have a total of 8 traffic movements in the diamond interchange. We will refer to movements 1, 4, 5, and 8 as the external movements, which feed the diamond, and to movements 2, 3, 6, and 7 as the internal movements, which clear traffic out of the diamond. We will now characterize each phase in terms of how many external movements are feeding the diamond and how many internal movements are clearing the diamond. Thus, phase 1 (Fig. 7) has 2 external movements, 4 and 8, feeding the diamond, while 2 internal movements, 2 and 6, clear the diamond. In abbreviated form, we can write this as 2-in, 2-out, where in refers to the external approaches, out refers to the internal approaches, and the number before the words indicates the number of approaches involved in feeding or clearing the diamond.

Therefore, we can characterize the 9 basic phases in terms of the number of traffic movements coming into and out of the diamond. These are given in Table 1.

Arrangements of 3 Basic Phases at Individual Ramp Intersections

We will now examine the 3 basic phases at each intersection of the diamond to determine the different ways in which they can be arranged. We readily see that they can be arranged in only 2 distinct ways, ABCABC or ACBACB. There is no other way to arrange these 3 basic signal phases. In the ABCABC (or CAB or BCA) arrangement, first, the straight-through and the left-turn movements begin at the same time; later, the left-turn movement is stopped, while the straight-through movement continues to move. We call this arrangement the leading left turn. Here, Phase C precedes Phase A. The other arrangement, ACBACB (or BAC or CBA), is called a lagging left turn, and here Phase C follows Phase A. Therefore, the 3 basic phases at an individual intersection of the diamond can be arranged in only 2 different ways, the leading left turn and the lagging left turn.

Through combinations of the 2 intersections, these 2 kinds of phasing arrangements will give all the possible types of phasing patterns. Once the 3 basic phases have been arranged at each intersection of the diamond, different ways for their juxtaposition with the other intersection can be accomplished by using only one other parameter, the offset. In this paper, an offset for the diamond has been defined as the time difference in seconds between the starting green time of movements 4 and 8 of the diamond interchange (Fig. 7). This time difference can also be expressed in terms of the percentage of the cycle length of the diamond signal.

TABLE 1

BASIC PHASES IN TERMS OF TRAFFIC MOVEMENTS

Phase	Operational Function	Movements at Green Signal
1	2-in, 2-out	2, 4, 6, 8
2	2-in, 0-out	1, 5
3	0-in, 4-out	2, 3, 6, 7
4	2-in, 1-out	4, 5, 6
5	1-in, 2-out	1, 2, 3
6	1-in, 3-out	2, 6, 7, 8
7	1-in, 3-out	2, 3, 4, 6
8	2-in, 1-out	1, 2, 8
9	1-in, 2-out	5, 6, 7

POSSIBLE PHASING PATTERNS

First of all, we will demonstrate the formation of the 4-phase-overlap phasing pattern shown in Figure 3. This can be formed if both intersections have leading left turns and if the cycle of phases is shifted in such a way that the start of the cycle at one intersection takes place in the middle of the other intersection cycle. This is shown in Figure 8. The bars represent the sequencing of the 3 basic phases at each ramp intersection. At the left ramp intersection, Phases A and B move the traffic into the diamond, while Phases A' and C' move this traffic out of the diamond at the left ramp intersection (Fig. 6). The corresponding 4-phase-overlap phasing pattern that results from this arrangement is shown in Figure 3.

The formation of this 4-phase-overlap phasing pattern is also shown in a compact form in Case 1 of Figure 9. Cases 2 through 5 (Fig. 9) represent the formation of other possible types of phasing patterns that can result from different offsets when both of the ramp intersections have leading left turns. Case 2 represents the phasing pattern when the offset is 0 percent of the cycle length; Cases 3, 4, and 5 represent the phasing patterns when the offsets are $\frac{1}{2}$, 1, and 2 phases in length respectively (where the phases are assumed to be uniform in length). The phasing patterns corresponding to Cases 2 through 5 are shown in Figure 10. This figure shows that the phasing pattern that results from the offset of 1 phase length is similar to that which results from the offset of 2 phase lengths (with the intersections reversed). Similarly, the phasing pattern that would result from the offset of $2\frac{1}{2}$ phases would be similar to the one resulting from a $\frac{1}{2}$ phase offset. Further inspection of this figure shows that none of these phasing patterns is as efficient as the one corresponding to Case 1 (4-phase-overlap).

Inefficiency results when cars in the internal approaches (movements 2, 3, 6, and 7) must wait unnecessarily, when there are no cars in the internal approaches to be served by the early seconds of a green signal and/or when cars in the external

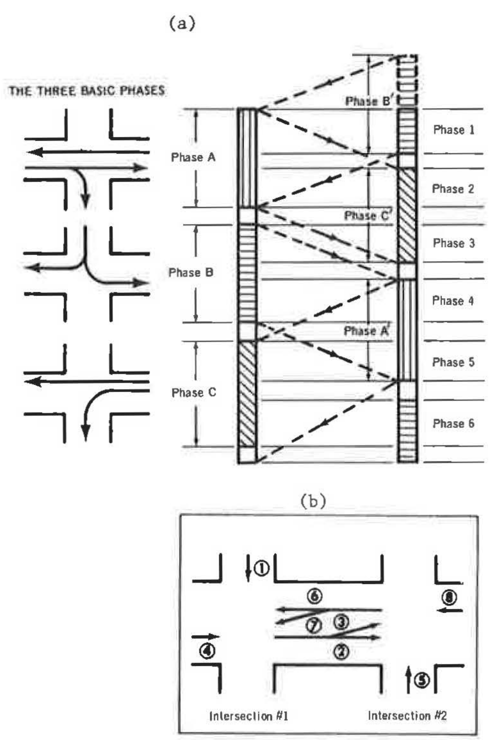


Figure 8. Formulation of 4-phase-overlap phasing pattern from 3 basic phases at each ramp intersection.

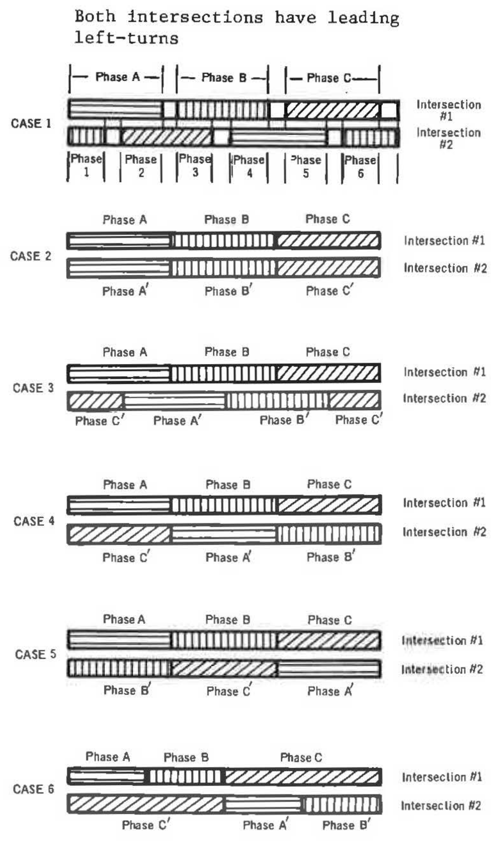


Figure 9. Formulation of various possible phasing patterns where both ramp intersections have leading left turns.

approaches (movements 1, 4, 5, and 8) cannot make efficient use of green signals because of insufficient space in the diamond to accommodate them.

However, it is not always possible to provide an ideal 4-phase-overlap phasing pattern in the field because of various demand and geometric constraints. An example of this is shown in Case 6 of Figure 9. This is a 4-phase-without-overlap phasing pattern shown in Figure 4. This phasing pattern results when both the intersections have leading left turns and, furthermore, the length of Phase C at each intersection (where there are left turns) is equal to the length of Phases A and B and superimposed on these phases at the other intersection. This arrangement again results in the starting of the cycle in each intersection at the middle of the cycle of the other intersection, except that here

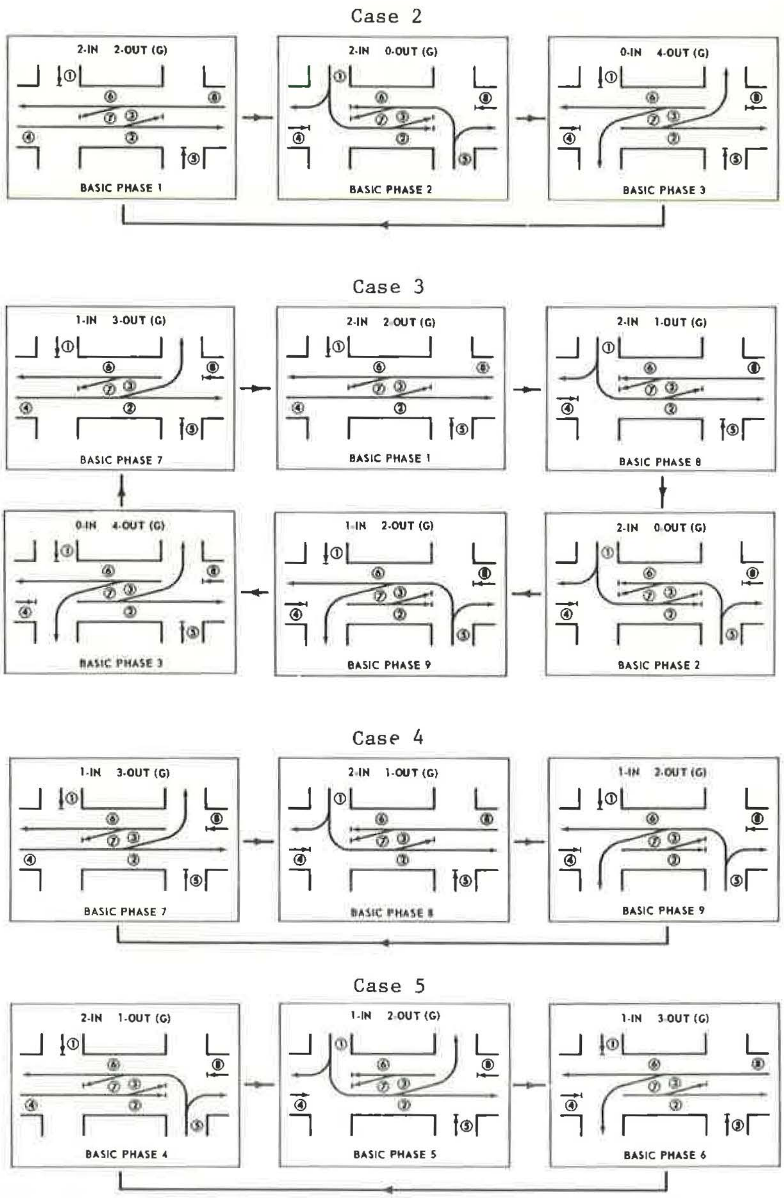


Figure 10. Phasing patterns where both ramp intersections have leading left turns.

the length of Phase C at each intersection is equal to the combined lengths of Phases A and B at the other intersection.

We will now examine the formulation of other possible types of phasing patterns where both intersections have lagging left turns. Case 1 shown in Figure 11 represents 0 offset—the simplest case. The phasing pattern that results from this arrangement is shown in Figure 1. Note that the 3-phase phasing pattern that results when both intersections have lagging left turns (Fig. 1) is better than the other 3-phase phasing patterns where both the intersections have leading left turns (Fig. 10, Cases 2, 4, and 5).

Cases 2 through 5 shown in Figure 11 represent the formation of other types of phasing patterns that can result from different offsets when both the intersections have lagging left turns. The various offsets considered in Cases 2 through 5 are ½, 1, 1½, and 2 phase lengths respectively (where phase lengths are assumed to be uniform). The corresponding phasing patterns that result from these arrangements are shown in Cases 2 through 5 in Figure 12. Note that the 3-phase phasing pattern with 0 offset (Case 1, Fig. 11) is more efficient than those obtained by other offsets (Cases 3 and 5, Fig. 11), when both intersections have lagging left turns.

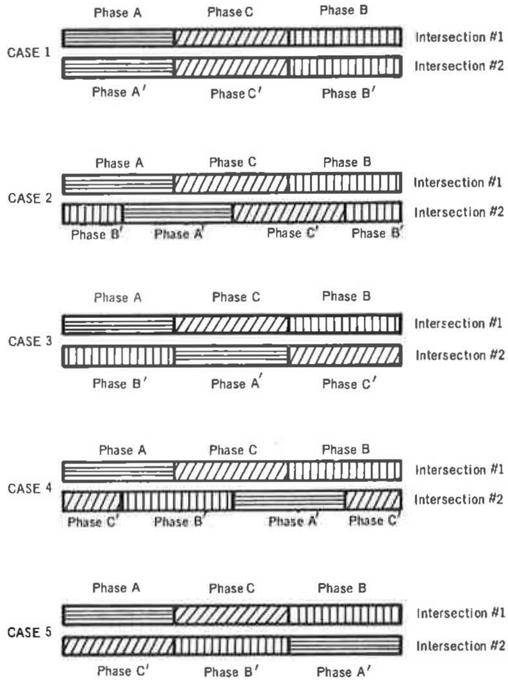


Figure 11. Formulation of various possible phasing patterns where both ramp intersections have lagging left turns.

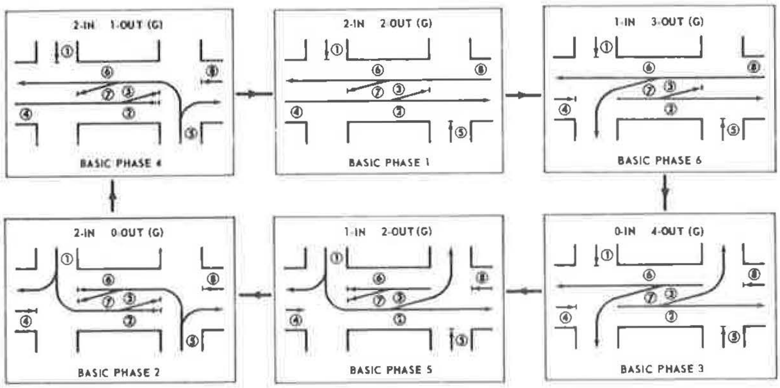
Other possible types of phasing patterns occur where one ramp intersection has a leading left turn and the second ramp intersection has a lagging left turn. The inefficiency of these phasing patterns is evidenced by similar analysis. Thus, the various possible phasing patterns at diamond interchanges result from the following 4 arrangements of the 3 basic phases at each ramp intersection:

1. Two ramp intersections having leading left turns,
2. Two ramp intersections having lagging left turns,
3. The first ramp intersection having a leading left turn and the second having a lagging left turn, and
4. The first ramp intersection having a lagging left turn and the second having a leading left turn.

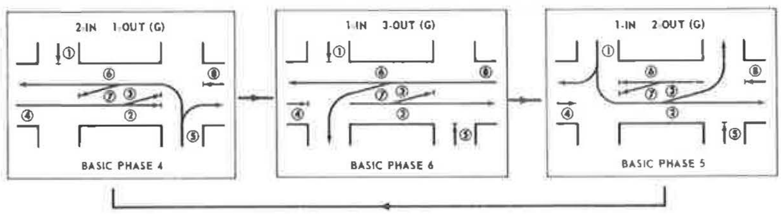
Each of these arrangements can have several different values of offsets and, corresponding to each arrangement with a given offset, a distinct phasing pattern.

We have determined 2 preferred sets of phasing patterns. One results when both intersections have a leading left turn and the offset is approximately 50 percent of the cycle length. The phasing patterns that result from this configuration belong to the general family of 4-phase-overlap phasing patterns. It may be pointed out that a correct time length within the constraints for each phase (especially the 2 overlap phases, Phases I and IV, Fig. 3) is important to the efficiency of the 4-phase-overlap phasing pattern.

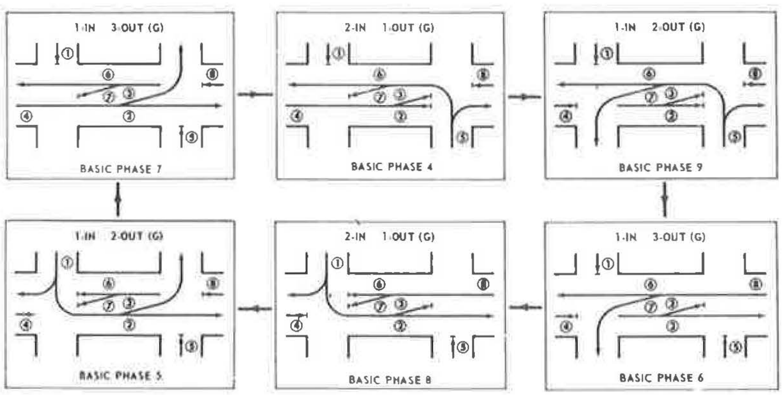
The other preferred phasing pattern results when both intersections have lagging left turns and the offset is about 0 percent of the cycle length. The phasing patterns that result from this configuration belong to the general family of 3-phase phasing patterns. This pattern will be efficient when the traffic interacting with the freeway is only a small percentage of the overall traffic in the diamond interchange or when the overall traffic is very light. In the latter case, the storage capacity of the diamond, Case 2



Case 3



Case 4



Case 5

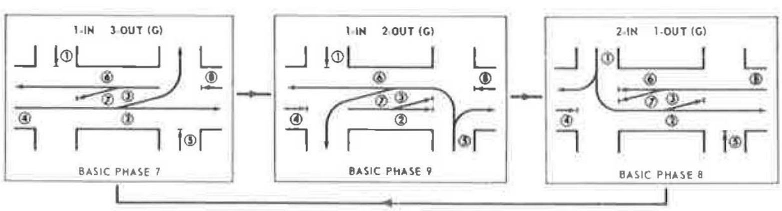


Figure 12. Phasing patterns where both ramp intersections have lagging left turns.

which normally restricts the lengths of Phases II and III (Fig. 1) in each intersection, is not critical because the cycle length is short.

A digital computer simulation model will determine the correct situations for the employment of each of these preferred phasing patterns.

REAL-TIME CONTROL OF DIAMOND INTERCHANGES

Unbalanced and nonuniform traffic demands and irregular geometric characteristics may require changes in the durations of the 3 basic phases (the time split) at each ramp intersection and make it impossible to adhere to the "ideal" fixed offsets for minimizing overall system delay. Furthermore, the especially complex and ever-changing character of the traffic in diamond interchanges makes it impossible to adhere to one set of phasing patterns (time splits, cycle lengths, sequencing, and offsets) and also obtain the highest possible efficiency for prevailing traffic conditions. Again, the degree of synchronization of the diamond ramp intersections with the arterial street signals should vary from time to time depending on the amount and percentage of drivers involved in the straight arterial movements. However, this coordination should also consider the effect on freeway traffic.

This study has analyzed all the possible signalization schemes that may be applied to the real-time control of diamond interchanges. Furthermore, the analysis given in this paper has specified the preferred set of phasing schemes that should be applied for real-time control of different traffic situations. A real-time control system may be defined as one that controls an environment by receiving measured data, describing the state of the environment, processing the data, and then using the results to execute an action that affects the environment with fast response time.

A control algorithm interlinks the measured data to the commanded action. A real-time control algorithm may be defined as a set of logical steps that control the traffic signals on a real-time basis by assigning a distinct control alternative (a means of activating the traffic signal system, where every traffic movement is given a specific amount of green time) to given traffic situations. These traffic situations are governed by a control strategy that determines the selection of the appropriate control alternative.

This analysis is being used in developing efficient real-time control algorithms, whereby different desired control alternatives (time splits, cycle length, sequencing, and offsets) will be applied on a continuous basis to match various traffic situations.

Several control algorithms have been developed (10) that determine the minimum cycle length that will handle traffic demands at each individual intersection in the system under consideration. The maximum of these minimum cycle lengths is taken as the common cycle length. Subsequently, green times (the splits) allocated for the basic phases depend on which algorithm is being used. These algorithms also determine the theoretical ideal offset for each pair of intersections (both leading, both lagging, or one leading and one lagging for full diamonds) by assigning proper weights to different intersections and to each direction in terms of its demand criticality.

These control algorithms are evaluated by a digital computer simulation model (11) that simulates traffic operations in a diamond interchange and is thus a powerful tool to measure the effectiveness of each control algorithm in terms of total system delay. This simulation model has been developed and validated according to macroscopic traffic statistics; that is, this model is accurate based on long-run statistics or large sample data. Another simulation model has also been developed (12, 13) that keeps track of the trajectory of each individual car and is thus accurate even in a small sample size.

The first simulation model, specially designed to evaluate real-time control algorithms is employed to filter out real-time control algorithms and to obtain fast simulation results. Based on this, an optimum set of control algorithms is chosen and a higher level evaluation obtained by using the second simulation model, which has more detailed mechanisms of car movements but requires longer simulation time.

FIELD EXAMPLES OF PHASING PATTERNS

Figure 13 shows the arrangement of the 3 basic phases for the Coldwater Canyon Avenue-Ventura Freeway interchange in Los Angeles and the time lengths of the phases

TABLE 2

OPERATIONAL CHARACTERISTICS OF SIGNALIZED DIAMOND INTERCHANGES IN LOS ANGELES

Interchange	Cycle Length (sec)	Offset		Green Time (sec)					
				Left Intersection			Right Intersection		
		Per-cent	Sec-onds	Phase A	Phase B	Phase C	Phase A	Phase B	Phase C
Ventura Freeway									
Whiteoak Avenue[a]									
Dial 1	60	43	25.8	15.0	12.0	24.0	24.0	15.0	12.0
Van Nuys Boulevard[a]									
Dial 1	60	53	31.8	21.0	10.2	19.8	23.4	11.4	16.8
Dial 2	60	47	28.2	21.0	10.2	19.8	21.0	16.2	13.8
Dial 3	60	50	30.0	21.0	12.0	18.0	23.4	11.4	16.8
Woodman Avenue[a]									
Dial 1	80	43	34.4	19.2	16.8	34.4	25.6	24.8	20.0
Dial 2	80	37	29.6	17.6	15.2	37.6	37.6	16.0	16.8
Balboa Boulevard[a]									
Dial 1	60	47	28.2	19.2	13.2	18.6	22.8	10.2	18.0
Dial 2	60	55	33.0	24.0	13.2	13.8	18.0	10.2	22.8
Coldwater Canyon[a]									
Dial 1	60	43	25.8	21.0	15.0	15.0	24.0	15.0	12.0
Dial 2	60	38	22.8	18.0	15.0	18.0	24.0	15.0	12.0
Dial 3	60	48	28.8	24.0	15.0	12.0	21.0	15.0	15.0
Laurel Canyon[a]									
Dial 1	60	50	30.0	21.0	12.0	18.0	21.0	12.0	18.0
Dial 2	60	51	30.6	16.8	16.8	17.4	21.6	10.8	18.6
Dial 3	60	56	33.6	23.4	13.2	14.4	16.2	13.2	21.6
Winnetka Avenue[a]									
Dial 1[f]	60	48	28.8	16.8	10.2	24.0	13.8	18.6	9.6
Dial 2[f]	60	39	23.4	10.2	10.2	30.6	15.6	15.0	7.2
San Diego Freeway									
Nordhoff Street[a]									
Dial 1	60	50	30.0	21.0	12.0	18.0	24.6	18.0	8.4
Roscoe Boulevard[a]									
Dial 1	60	49	29.4	24.0	9.0	18.0	21.0	18.0	12.0
Santa Monica Boulevard[a]									
Dial 1	60	44	26.4	23.4	16.2	11.4	25.8	16.8	8.4
Dial 2	60	42	25.2	18.0	21.0	12.0	19.8	22.2	9.0
Dial 3	60	57	34.2	19.8	16.2	15.0	25.8	16.8	8.4
La Tijera Boulevard[b]									
Dial 1	60	0	0.0	29.4	6.0	15.6	25.8	9.6	15.6
Dial 2	90	94	84.6	54.0	8.1	17.1	41.4	24.3	13.5
Dial 3	90	93	83.7	53.1	9.0	17.1	41.4	19.8	18.0
Hollywood Freeway									
Hollywood Boulevard[d]									
Dial 1	60	8	4.8	33.0	12.0	6.0	33.0	21.0	–
Alvarado Street[d]									
Dial 1	60	0	0.0	36.0	18.0	–	36.0	13.8	1.2
Magnolia Boulevard[e]									
Dial 1	60						36.0	18.0	–
Dial 2	60						36.0	18.0	–
Dial 3	60						30.0	24.0	–
Santa Monica Freeway									
Normandie Avenue[a]									
Dial 1									
Case 1	60	51	30.6	20.4	20.4	10.2	20.4	20.4	10.2
Case 2	60	52	31.2	20.4	20.4	10.2	20.4	20.4	10.2
Dial 2	80	49	39.2	36.0	16.8	17.6	32.0	16.8	21.6
Dial 3	80	42	33.6	34.4	17.6	18.4	41.6	16.8	12.0
Dial 4	60	51	30.6	20.4	20.4	10.2	20.4	20.4	10.2
Dial 5	80	50	40.0	28.8	20.8	20.8	36.0	17.6	16.8
Dial 6	80	57	45.6	36.8	16.8	16.8	27.2	17.6	25.6
Arlington Avenue[a]									
Dial 1									
Case 1	60	47	28.2	21.0	15.0	15.0	21.0	16.2	13.8
Case 2	60	0	0.0	21.0	15.0	15.0	21.0	16.2	13.8
Dial 2	80	51	40.8	35.2	16.8	18.4	37.6	16.8	16.0
Dial 3	80	55	44.0	36.8	16.8	16.8	29.6	29.6	11.2
Dial 4									
Case 1	60	37	22.2	21.0	15.0	15.0	21.0	16.2	13.8
Case 2	60	47	28.2	21.0	15.0	15.0	21.0	16.2	13.8
Dial 5	80	50	40.0	28.8	20.8	20.8	36.0	17.6	16.8
Dial 6	80	54	43.2	36.8	16.8	16.8	27.2	16.8	26.4

TABLE 2 (Cont'd)

Interchange	Cycle Length (sec)	Offset		Green Time (sec)					
				Left Intersection			Right Intersection		
		Per-cent	Sec-onds	Phase A	Phase B	Phase C	Phase A	Phase B	Phase C
Western Avenue[a]									
Dial 1									
Case 1	60	27	16.2	24.0	16.2	10.8	27.0	15.0	9.0
Case 2	60	0	0.0	24.0	16.2	10.8	27.0	15.0	9.0
Case 3	60	25	15.0	24.0	16.2	10.8	27.0	15.0	9.0
Dial 2									
Case 1	80	51	40.8	33.6	16.0	20.8	40.0	16.0	14.4
Case 2	80	52	41.6	33.6	16.0	20.8	40.0	16.0	14.4
Case 3	80	2	1.6	33.6	16.0	20.8	40.0	16.0	14.4
Dial 3									
Case 1	80	47	45.6	32.0	16.0	22.4	43.2	16.0	11.2
Case 2	80	39	31.2	32.0	16.0	22.4	43.2	16.0	11.2
Case 3	80	33	26.4	32.0	16.0	22.4	43.2	16.0	11.2
Vermont Avenue[a]									
Dial 1	60	49	29.4	18.0	22.8	10.2	16.8	22.8	11.4
Dial 2	80	40	32.0	23.4	33.0	14.6	23.4	33.0	14.6
Dial 3	80	50	40.0	23.4	33.0	14.6	23.4	33.0	14.6
Crenshaw Boulevard[c]									
Dial 1	60	20	12.0	24.0	12.0	15.0	20.4	20.4	10.2
Dial 2	60	28	16.8	27.0	15.0	9.0	30.0	10.2	10.8
Dial 3	60	23	13.8	27.0	12.0	12.0	28.2	14.4	8.4
Harbor Freeway									
Anaheim Street[a]									
Dial 1	60	50	30.0	27.0	15.0	9.0	27.0	15.0	9.0
Dial 2	60	55	33.0	24.0	15.0	12.0	18.0	15.0	18.0
Dial 3	60	50	30.0	21.0	15.0	15.0	24.0	15.0	12.0
Alondra Boulevard[a]									
Dial 1	60	50	30.0	24.0	14.0	12.0	24.0	15.0	12.0
Dial 2	80	43	34.4	35.2	19.2	16.0	30.4	16.0	24.0
Dial 3	80	42	33.6	26.4	24.0	20.0	38.4	16.0	16.0
Century Boulevard[a]									
Dial 1	60	69	41.4	27.0	18.0	6.0	18.0	12.0	21.0
Dial 2	60	69	41.4	21.0	24.0	6.0	15.0	10.2	25.8
Vernon Avenue[a]									
Dial 1									
Case 1	60	54	32.4	21.0	18.0	12.0	27.0	18.0	6.0
Case 2	60	90	54.0	21.0	18.0	12.0	27.0	18.0	6.0
Dial 2									
Case 1	80	54	43.2	33.6	16.8	20.0	24.0	22.4	24.0
Case 2	80	91	72.8	33.6	16.8	20.0	24.0	22.4	24.0
Dial 3									
Case 1	80	88	70.4	36.8	19.2	14.4	32.0	22.4	16.0
Case 2	80	46	36.8	36.8	19.2	14.4	32.0	22.4	16.0
Dial 4									
Case 1	60	58	34.8	21.0	18.0	12.0	27.0	18.0	6.0
Case 2	60	90	54.0	21.0	18.0	12.0	27.0	18.0	6.0
Dial 5	80	93	74.4	26.4	17.6	26.4	26.4	26.4	17.6
Dial 6	80	61	48.8	36.8	19.2	14.4	30.4	20.8	19.2
Florence Avenue[d]									
Dial 1	60	0	0.0	34.2	19.8	—	22.2	12.0	13.8
Dial 2	60	0	0.0	30.0	24.0	—	25.2	12.0	10.8
Dial 3	60	0	0.0	28.2	25.8	—	19.2	12.0	16.8
Rosecrans Avenue[e]									
Dial 1	60			33.0	21.0	—			
Dial 2	70			42.0	21.0	—			
Dial 3	80			40.0	32.0	—			
Imperial Highway[c]									
Dial 1	60	82	49.2	27.0	18.0	6.0	27.0	6.0	18.0
Dial 2	60	80	48.0	22.8	19.7	9.0	22.8	9.0	19.2
Dial 3	60	80	48.0	22.8	19.2	9.0	22.8	9.0	19.2
Golden State Freeway									
North Broadway[c]									
Dial 1	60	72	43.2	29.4	12.0	9.6	24.0	12.0	15.0
Dial 2	90	76	68.4	43.2	15.3	20.7	45.9	15.3	18.0
Dial 3	90	90	81.0	52.2	12.6	14.4	38.7	20.7	19.8
Sunland Boulevard[a]									
Dial 1	60	47	28.2	21.0	15.0	15.0	21.0	15.0	15.0
Dial 2	60	47	28.2	15.0	15.0	21.0	24.0	15.0	12.0
Dial 3	60	47	28.2	24.0	15.0	12.0	18.0	18.0	15.0

[a]Left and right intersections leading left turn.
[b]Left and right intersections lagging left turn.
[c]One intersection leading and other intersection lagging left turn.
[d]One intersection has unprotected left turn (no Phase C) and other intersection leading left turn.
[e]One intersection has unprotected left turn (no Phase C) and other intersection traffic-actuated.
[f]Includes and extra Phase AB (dial 1, 6.0, and dial 2, 10.2) between Phases B and C, a situation where movement 5 has a right-turn green arrow, movement 8 is green, and movements 2 and 3 are red.

in relation to the various offsets for the different times of day. Note that the 3 basic phases at each intersection have been arranged in a leading left-turn order. Case 2 refers to January 15, 1970, when the signalization of this diamond interchange was last set. The offsets for the off-peak, morning peak, and evening peak periods (also referred to as dials) are 25.8, 22.8, and 28.8 sec respectively. Case 1 (Fig. 13) refers to August 20, 1969, when the offsets for the interchange were originally set. The old offsets for the 3 peak periods were 33, 30, and 36 sec respectively. Case 1A (Fig. 13) for the morning peak hours refers to a situation where the offset between the 2 intersections of the diamond interchange was changed from the original 30 sec to 33 sec.

Thus we see that, although the green time for each of the movements (1, 2, 3, 4, 5, 6, 7, and 8 in Fig. 8) does not change with the use of different offsets, we do get different phasing patterns. The efficiency of these different phasing patterns will vary for different offset values, even though the green time for each basic phase or movement and the arrangement of the 3 basic phases at each leading left-turn intersection remain the same. Thus, there can be numerous phasing patterns (depending on the different values of offsets) that will have the same green time for each movement. We have analyzed a case in which the offset is close to half the cycle length and both the

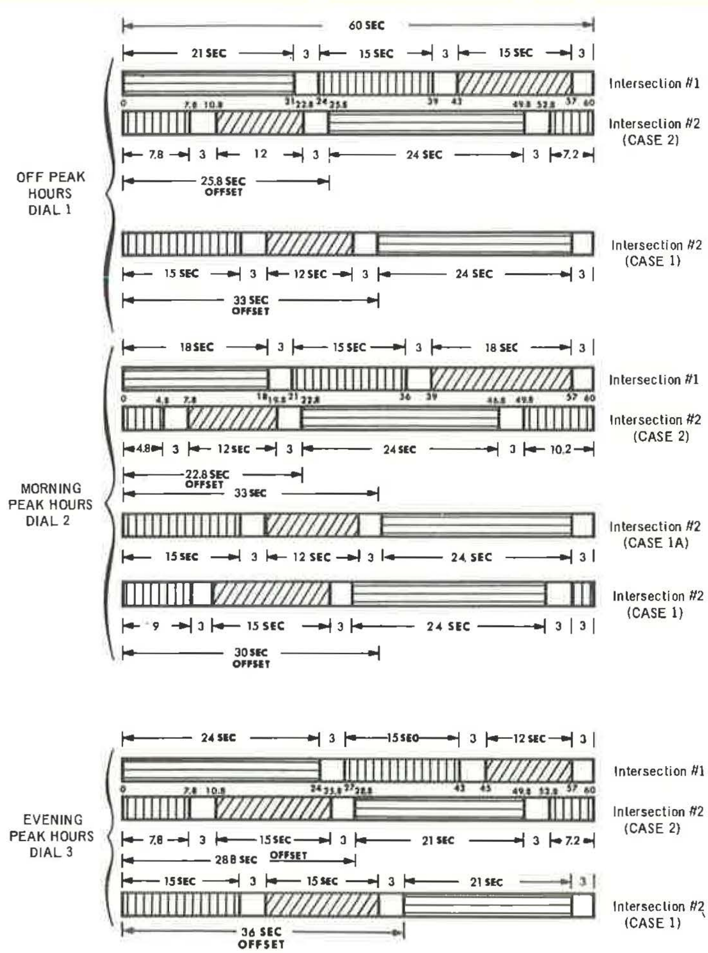


Figure 13. Formulation of different phasing patterns for Coldwater Canyon Avenue-Ventura Freeway diamond interchange.

intersections have leading left turns. This phasing pattern belongs to the family of 4-phase-overlap phasing patterns. Therefore, the value of the offset in terms of cycle length, the order in which the 3 basic phases have been arranged (leading left turn or lagging left turn), and the associated time lengths of the phases quite accurately describe the kind of phasing pattern resulting from a given situation.

Based on these parameters describing a phasing pattern, the timing charts of the various signalized full-diamond interchanges in the city of Los Angeles were studied, and it was found that almost all of these diamonds have leading left turns in both intersections, with offsets roughly on the order of one-half the cycle length. It is concluded, therefore, that the phasing pattern of most of these interchanges belong to the 4-phase-overlap family. An exception is the La Tijera-San Diego Freeway interchange, where both intersections have lagging left turns and the offset is close to full cycle length. Table 2 gives a summary of the investigation regarding full diamonds in the city of Los Angeles.

Of the 28 signalized full diamonds for which information is available at this time, only 1 diamond (La Tijera Boulevard-San Diego Freeway) has both lagging left turns; 3 diamonds (Crenshaw Boulevard-Santa Monica Freeway, Imperial Highway-Harbor Freeway, and North Broadway-Golden State Freeway) have 1 leading left turn and 1 lagging left turn; and 2 diamonds (Magnolia Boulevard-Hollywood Freeway and Rosecrans Avenue-Harbor Freeway) have an unprotected left turn (no Phase C) in one intersection and are traffic-actuated in the other intersection. Three diamonds (Hollywood Boulevard-Hollywood Freeway, Alvarado Street-Hollywood Freeway, and Florence Avenue-Harbor Freeway) have an unprotected left turn (no Phase C) in 1 intersection and leading left turn on the other intersection. The Winnetka Avenue-Ventura Freeway interchange includes an extra Phase AB between Phase B and C in its east ramp intersection. In this latter case, Phase AB refers to a situation where movement 5 has a right turn green arrow, movement 8 is green, and movements 2 and 3 are red.

For the 28 full diamonds, there are 92 dials. Sixty have cycle lengths of 60 sec, 27 have cycle lengths of 80 sec, 4 have cycle lengths of 90 sec, and 1 has a cycle length of 70 sec. Of the 67 dials having leading left turns in both intersections, 54 have offsets between 35 and 65 percent of cycle length (Vernon Avenue and Century Boulevard, both influenced by Coliseum traffic, provide the majority of offsets outside this range), 49 have offsets between 40 and 60 percent, and 35 have offsets between 45 and 55 percent of cycle length.

CONCLUSION

A systematic approach to the signalization of diamond interchanges has been developed by critically reviewing the existing practices used for signalizing diamond interchanges. A detailed analysis was made to identify all the basic elements that constitute a phasing pattern. After each of the phasing patterns had been described by a set of parameters that had one-to-one correspondence with the signal-controller adjustable variables, the various possible phasing patterns were developed and synthesized. An evaluation of each phasing pattern was made, and 2 preferred sets with respect to both efficiency and practicality were identified: one in which both intersections have leading left turns with offset about 50 percent of the cycle length, and one in which both intersections have lagging left turns with offsets of about 0 percent of the cycle length. One typical diamond interchange phasing pattern from the field was described. In addition, a summary was made of the set of parameters describing the operational characteristics of diamond interchange signalization in the city of Los Angeles.

These signalization concepts were developed for application to the development of a real-time control system, as well as for providing an objective and explicit basis for evaluating fixed signalization of diamond interchanges. This analysis is currently being used to develop real-time control algorithms in which different desired control alternatives will be applied on a continuous basis to match varying traffic situations, thereby promoting the efficient operational performance of diamond interchanges.

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