HIGHWAY RESEARCH RECORD

Number 453

Traffic: Capacity, Delay, and Quality of Service

5 reports
prepared for the
52nd Annual Meeting

Subject Areas

53 Traffic Control and Operations
55 Traffic Measurements

HIGHWAY RESEARCH BOARD

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

Washington, D.C. 1973

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ISBN 0-309-02186-3
Library of Congress Catalog Card No. 73-15102
Price: $2.20

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CONTENTS

FOREWORD

Highway planning, design, and traffic operations engineers, and in fact all users of the Highway Capacity Manual, will find research results of interest and value in the five papers and four discussions presented in this RECORD. Despite, or perhaps because of, the wide application of the 1965 edition of the Manual, areas of weakness have been identified in its procedures. The work reported here will be useful in modifying current practice and ultimately will be helpful in updating portions of the Manual, especially as related to intersections.

Concerned with the effects of roadside land use on the quality of traffic service, Berg and Anderson studied roadway and land use characteristics for several 4-lane undivided arterials in Madison, Wisconsin. Their analysis revealed that unit travel time on the arterials increased as the number of daily trip ends per mile or the amount of commercial floor area per mile increased, and they concluded that control of roadside development was clearly important in preserving the quality of traffic service.

Using extensive data available from aerial photographs of traffic on the Long Island Expressway and on the Ventura and Santa Ana freeways, Munjal and Hsu present detailed traffic characteristics such as time headway distribution, time speed distribution, space headway distribution, and space speed distribution. Data are given for every lane and for various levels of traffic service, in both graphical and summary table forms.

Sofokidis, Tilles, and Geiger report on their evaluation of two techniques for measuring intersection delay: the Sagi-Campbell and the Berry-Van Til methods. Delay measurements taken from video tapes for each of the methods were compared with delays computed from time-lapse photographs made concurrently with the video tapes. Neither method produced consistent trends in predicting delays as compared to the base methods, and even these failed to give consistent, direct relations. Four extensive discussions by Hutter, McShane, May, and Berry add materially to this important subject matter.

Next, Johnsen and Matthias suggest that the capacity of left-turn lanes protected by signals can be as much as one-third greater than that calculated by current procedures. This finding is based on their study of selected urban intersections having signals with varying cycle lengths.

In the last paper, Berry and Ghandi describe a method for computing capacity of signalized intersections based on measuring starting time delays, entering headways for loaded portions of cycles, and utilization of the yellow interval for loaded cycles. They also present data on use of the method to evaluate effects on capacity of weather and visibility at one intersection.

ANALYSIS OF THE TRADE-OFF BETWEEN LEVEL OF LAND ACCESS AND QUALITY OF TRAFFIC SERVICE ALONG URBAN ARTERIALS

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The objectives of this research were to analyze the rate at which units of traffic service are substituted for increments of land access and to evaluate alternative measures of level of land access. Several four-lane undivided arterials in Madison, Wisconsin, were selected for study. Each arterial was subdivided into sections approximately 0.2 mile in length. Roadway and land use characteristics were recorded for each section, and the average unit travel time within each section was measured for each direction of travel during five different time periods. Multiple linear regression analysis was then used to express unit travel time as a function of level of land access and other influencing variables. The analysis revealed that, as the number of daily trip ends per mile or the amount of commercial floor area per mile increases, unit travel time also increases, the amount of change being dependent on the time of day. Although quantifying level of land access by a trip generation rate is intuitively more desirable, the commercial floor area measure is more easily determined. The results of the research illustrate the importance of the control of roadside development as a means of preserving the quality of traffic service provided by urban arterials.

•PROVISION of land access and movement of traffic are the two primary functions of an urban street system. Because it is impossible for a given roadway to concurrently perform both functions efficiently, streets and highways are classified and designed based on the proportion of land access and traffic movement to be provided. In the case of an arterial roadway, the major purpose is to provide for the safe and efficient movement of relatively high volumes of traffic. Its secondary purpose is to provide relatively low levels of direct access to abutting land uses.

Because of the general absence of land access controls along urban street systems, the performance of newly constructed or improved arterials frequently deteriorates because of increases in the level of roadside development. This not only generates new sources of traffic demand that must use the facility but also reduces the original traffic service capability of the roadway. This process can thus lead to the early functional obsolescence of the highway facility.

Potentially, one of the most effective methods of preserving the utility of urban street systems is by means of a set of highway access and land use controls (9, 10, 11). These controls would define the specific levels of land access to be allowed along any given roadway. Ideally, the measure of land access would consider the number, type, and traffic generating characteristics of abutting land uses having direct highway access.

One of the major obstacles to this type of land use control is the lack of knowledge concerning the precise interrelation between level of land access and quality of traffic service. Research to date has shown that commercial roadside development does affect travel speeds and delay to through traffic. Horn et al. (7) found that, for a given traffic volume, speeds along developed sections of two-lane roadway in North Carolina were lower than those along underdeveloped test sections. In a subsequent study, Cribbins et al. (3) found that the log of travel time per mile along divided highways was inversely related to the number of access points per mile raised to the second power. The access-point parameter was designed to evaluate the number of conflicts introduced into the traffic stream. The number of access points for a particular abutting land use was simply an estimate of the average daily traffic generated at that location. In 1967, Treadway and Oppenlander (12) reported on a research study of speeds and delays along a two-lane urban bypass facility in Lafayette, Indiana. Their analysis revealed that the number of intersecting streets per mile and the number of commercial establishments per mile were both inversely related to mean travel speed.

Thus, although previous research has indicated that level of land access is inversely related to the quality of traffic service provided by a roadway, there is incomplete knowledge regarding the rate at which units of traffic service are substituted for increments of land access or the most suitable measure of level of land access. The research described herein represents an attempt to begin to solve these two problems.

PROCEDURE

The primary hypothesis to be tested was that an inverse relation exists between the traffic generated by roadside development and the quality of service to through traffic. As a corollary, it was also hypothesized that quality of traffic service is inversely related to the type and intensity of land use found along the highway. The approach of this study was to statistically analyze these interrelations using data to be collected for sections of four-lane undivided arterials in the Madison, Wisconsin, urban area.

Study Variables

Those variables that were selected to represent the basic interacting parameters are given in Table 1. Each variable is measured with respect to a given hour and direction of travel.

Quality of Traffic Service—Of the many available measures of quality of traffic service (6), unit travel time was considered to be the most appropriate. The Highway Capacity Manual (5) recommends the use of average overall travel speed in evaluating capacity and levels of service of urban arterials. Unit travel time as used in this study is simply the reciprocal of average overall travel speed. Unit travel time is also consistent with the units of the selected roadside characteristic variables.

Level of Land Access—In general, the traffic delay related to roadside development is caused by turning movements into and out of access points such as driveways and intersections. Because of this, any measure of the amount of abutting land use activity should account for the number of entering and exiting traffic movements over some time frame.

Several trip generation studies of specific land uses have been and are being undertaken to provide this type of information (1, 4, 8, 9). However, because of the wide range of conditions under which field studies or interviews have been conducted and the differences in classification of specific uses, there exists a substantial variation in reported rates for each land use classification. These difficulties notwithstanding, the average number of daily trip ends per mile (the sum of all entering and exiting traffic movements including intersections) was selected as the first measure of level of land access to be examined.

Trip generation studies have also shown that the number of trips generated by a given land use activity is a function of its type and size. Because commercial uses, as opposed to other land uses, are the major generators of roadside traffic over extended periods of time, the amount of commercial floor area adjacent to a given roadway was identified as a second measure of level of land access. Although this variable is intuitively less satisfactory than a trip generation rate, it can be measured and estimated with greater reliability.

Another measure of level of land access is simply a disaggregation of commercial activity into two classifications: retail and office (or service) commercial. The latter two variables require additional data, but they should more accurately reflect actual levels of trip generation. Because the three floor-area land access variables could not account for the delaying effect of intersection movements, the number of intersections per mile was included as a complementary variable for both sets of floor-area per-mile variables.

It was apparent that the potential delay to movement along a given arterial is influenced by the interaction between the adjacent land use activities and the respective directional traffic flows. In effect, the adjacent land use activities attract turning movements, and each directional flow produces turning movements in proportion to its total volume. Figure 1 shows this interaction for the general case. The figure shows the possible vehicular movements into and out of land uses abutting a section of arterial roadway. The respective directional flows are V1 and V2, and the adjacent land access variables are denoted by LA1 and LA2.

In order to account for the composite effect of abutting land use and traffic volume on delay to through traffic, the respective land access variables, LA1 and LA2, were weighted by those directional flows that might produce delaying turning movements with respect to a selected direction of travel. Thus, for travel in the direction of flow V1, the weighted measure of level of land access, $\overline{LA1}$, was expressed as

$$\overline{LA1} = LA1\ (V1 + V2) + LA2\ (V1) \tag{1}$$

As shown by this equation, delaying turning movements to or from a near-side land use activity, LA1, could be produced by traffic moving in either direction. However, turning movements to or from a far-side land use activity, LA2, would only cause delay to flow V1 if they were produced by flow V1.

As a matter of convenience, a weighting of unity for the near-side land access variable, LA1, was obtained by dividing the right-hand side of Eq. 1 as follows:

$$\overline{LA1} = LA1 + \left(\frac{V1}{V1 + V2}\right) LA2 \tag{2}$$

The resulting weighting factor, V1/(V1 + V2), is numerically equivalent to the percentage of the total arterial traffic flow that is moving in the selected direction of travel. Each of the land access variables identified for analysis was then weighted using Eq. 2.

Traffic Characteristics—This parameter can be defined in terms of trip characteristics and flow characteristics. Trip characteristics include factors such as trip purpose, trip length, and time of trip. Flow characteristics can be defined in terms of volume, speed, density, and other quality of service measures.

The trip characteristics of those drivers using a given arterial were not quantified or directly accounted for. However, they were indirectly considered by stratifying the sample data with respect to the a.m., p.m., and noon peak hours and the a.m. and p.m. off-peak hours. It has generally been found that the a.m. peak hour consists primarily of work trips, the p.m. peak hour includes both shopping trips and work trips, and the noon and off-peak hours are characterized by a mixture of shopping, social-recreational, and work trips.

Because quality of traffic service as measured by unit travel time was the dependent variable of interest, volume was the only traffic flow characteristic to be selected. Directional distribution does enter the analysis indirectly as the weighting factor for the level of land access variables.

Roadway Characteristics—The single roadway characteristic directly considered as an independent variable was speed limit. The influence of other roadway features was controlled through selection of the study sites. For example, an arterial was chosen for study only if it was a tangent, level roadway with two lanes per direction of travel and no median. In order to study the relation between unit travel time and level of land access more readily, the sample arterials were also required to have widely spaced signalized intersections. Where a signalized intersection did occur and interrupted flow conditions existed, the section containing the traffic signal and its estimated zone of influence (500 to 1,000 ft along each approach) was deleted.

A variable that was given much consideration but not included was roadway capacity. The capacity of an urban arterial is based in large part on the volume of vehicles capable of moving through a signalized intersection (5). Because signalized intersections were excluded from the study, intersection capacity at various points along an arterial was not judged to be an appropriate variable.

Data Collection

The essential characteristics of the three sites selected for analysis are given in Table 2. Many additional locations were discarded for one or more of the following reasons: (a) absence of commercial development along the roadside, (b) presence of closely spaced traffic signals or other bottleneck locations, (c) substandard roadway alignment, and (d) controlled access to roadside development.

Data collection began during the summer of 1971. Aerial photographs were used to prepare strip maps of each arterial showing the number of lanes, intersecting streets, access points, and land uses with direct access to the arterial. Each arterial was then divided into a number of sections. The basis for the location of a control point delineating the beginning or end of a section was twofold. First, control points were selected to represent demarcations in type and intensity of land use development along the arterial. Second, the length of the sections was kept generally uniform, approximately $^{1}/_{10}$ to $^{3}/_{10}$ mile long.

A plot of traffic volume versus time of day for each arterial was analyzed to determine characteristic peak and off-peak hours. Five time periods, 7:15 a.m. to 8:15 a.m., 10:00 a.m. to 11:00 a.m., 12:00 noon to 1:00 p.m., 2:00 p.m. to 3:00 p.m., and 4:15 p.m. to 5:15 p.m., were selected for study. A total of 12 travel time runs were then made on various weekdays for each arterial, hour, and direction of travel using the "average-car" method (2). When making these runs, elapsed times were recorded as each control point delineating a section was passed. The total sample thus consisted of travel time data along 56 arterial roadway sections for each of the five 1-hour periods.

Trip generation data compiled from previously published research investigations (1, 4, 8, 9) were utilized for the study sites. The selected trip generation rates given in Table 3 were expressed in average daily trips per 1,000 square feet of gross floor area for commercial and industrial uses, per student for institutions, and per dwelling unit for residential uses. Land use data for the study sites were obtained directly from the aerial photographs or by field measurements. Average daily trip ends on local streets that intersected the study arterials, for which no recent volume count was available, were estimated by considering the surrounding land use, local street pattern, street function, and volumes carried by similar streets in the area.

Data Analysis

Multiple linear regression analysis was used to test several travel time models. Each one was linear in nature and of the general form

$$Y = a + \sum_{i=1}^{n} b_i x_i \tag{3}$$

where

Y = unit travel time for given hour, section, and direction of travel;
x_i = influencing variable;
n = number of influencing variables; and
a, b_i = regression coefficients.

The quality of a model was determined by use of statistics output by a stepwise regression analysis computer program. These included the mean, standard deviation, and correlation coefficients for each variable and the coefficient of determination for the model itself.

A test of significance at the 10 percent level was made as each independent variable entered the equation. These significance tests allowed deletion of those variables that did not measurably contribute to the variance explanation of the model.

Table 1. Variables selected for analysis.

Parameter	Symbol	Variable
Quality of traffic service	UT	Unit travel time, minutes per mile
Level of land access	WTRIPS	Weighted trip ends, thousands per mile for an average day
	WCOMFA	Weighted commercial floor area, thousands of square feet per mile
	WRETFA	Weighted retail floor area, thousands of square feet per mile
	WOFFFA	Weighted office floor area, thousands of square feet per mile
	WINTR	Weighted number of intersections per mile
Traffic characteristics	V	Volume, vehicles per hour
Roadway characteristics	SPEEDL	Speed limit, miles per hour

Table 2. Characteristics of study arterials.

Arterial	Overall Length (mile)	Number of Sections	Length of Sections (mile)	Number of Lanes	Lane Width (ft)	Speed Limit (mph)	Curb Parking	Average Weekday Traffic Volume
Monona Drive	1.97	22	0.08 to 0.23	4	12	30	No	12,270 to 15,760
Sherman Avenue	1.27	14	0.11 to 0.19	4	11	25, 35	No	9,680 to 13,720
University Avenue	2.01	20	0.11 to 0.30	4	11	30, 40	No	12,110 to 20,170

Figure 1. Generalized turning movements along an arterial roadway.

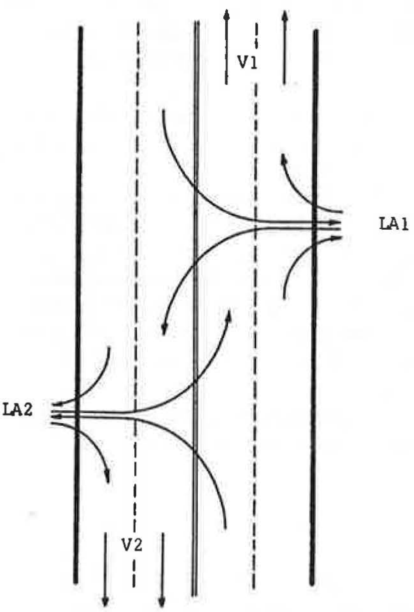

Table 3. Selected daily trip generation rates.

Land Use and Density Unit	Vehicle Trip Ends per Unit: Range	Typical
Residential (dwelling)		
Single-family	7.0 to 12.0	9.0
Apartments	3.0 to 7.9	6.0
Retail commercial (floor area, 1,000 square feet)		
Grocery store	17.6 to 43.4	30.5
Supermarket	70.0 to 240.0	130.0
Furniture store	0.6 to 13.4	5.6
Variety store	9.8 to 18.4	14.4
Drug store	19.0 to 99.8	43.5
Clothing store	10.4 to 55.0	31.3
Hardware store	21.6 to 37.4	29.5
Bank	5.8 to 188.0	61.5
Drive-in restaurant	1,160.0 to 3,260.0	1,160.0
Community shopping center	40.0 to 81.0	58.0
Highway service commercial (floor area, 1,000 square feet)		
Automobile sales and service	8.8 to 10.2	9.5
Service station	4.0 to 12.0	10.0
Motel	4.0 to 12.0	10.0
Office commercial (floor area, 1,000 square feet)		
General	1.6 to 60.0	14.0
Medical office	31.0 to 53.0	34.0
Post office	6.8 to 171.0	20.0
Library	56.6 to 62.4	59.6
Industrial (floor area, 1,000 square feet)		
Light	0.2 to 1.0	0.6
Institutional (student)		
Elementary school	0.4 to 1.0	0.8
High school	1.1 to 2.1	1.4
Recreation (acre)		
Golf course	2.0 to 10.0	8.0

RESULTS

Summary statistics for each of the study variables are given in Table 4. The difference in the means and standard deviations of the land access variables from one hour to another is due to the variation of the weighting factor.

Preliminary analysis of the data revealed that unit travel time was not significantly influenced by traffic volume for the study conditions. This was not totally surprising. The Highway Capacity Manual (5) indicates that, in general, as the volume-capacity ratio of an urban arterial increases up to approximately 0.8, there is only a slight reduction in average overall travel speed. Instead, at locations beyond the influence of a signalized intersection, average travel speed tends to be affected primarily by the posted speed limit and the level of marginal and intersectional friction. In light of these observations, hourly volume was deleted from further analysis. It should be emphasized, however, that the lack of a meaningful and statistically significant relation between travel time and traffic volume for the study sites does not justify the conclusion that traffic volume has no influence on average travel speeds along urban arterials. It would be essential to undertake additional research over a wider variety of traffic flow conditions before the actual relation could be reliably established.

The first travel time model to be tested used the number of weighted trip ends per mile as the measure of land access and speed limit as the controlling traffic flow parameter:

$$UT = a + b(WTRIPS) + c(SPEEDL) \tag{4}$$

The resulting regression coefficients and the coefficient of determination (R^2) are given in Table 5 for each of the five time periods studied. The absence of a coefficient for any variable during a given hour indicates that the variable was not significant at the 10 percent level.

As might be expected, the most significant variable during any 1 of the 5 hours was speed limit. The negative regression coefficients show that an increase in speed limit generally brings about a corresponding decrease in overall travel time.

The number of weighted trip ends was directly related to unit travel time during the a.m. and p.m. off-peak hours and the p.m. peak hour (and only slightly insignificant during the noon hour). This appears reasonable because most commercial uses are closed during the morning peak hour, whereas shopping activity usually declines over the noon hour. The values of the regression coefficients for weighted trip ends indicate that roadside development creates less interference to through traffic during the morning than in the afternoon. This is most likely a reflection of the greater amount of shopping and business activity taking place in the afternoon.

The second travel time model to be tested used weighted commercial floor area per mile and number of intersections per mile as measures of level of land access and speed limit as the controlling traffic flow parameter:

$$UT = a + b(WCOMFA) + c(WINTR) + d(SPEEDL) \tag{5}$$

Table 6 gives the resulting statistics for each of the five time periods studied. The absence of a coefficient for any variable during a given hour indicates that the variable was not significant at the 10 percent level.

Speed limit again proved to be the most significant variable during the 5 hours of study. The regression coefficients were negative, indicating the inverse nature of the relation between speed limit and unit travel time.

The weighted commercial floor area variable was significant in all but the a.m. peak hour. As noted before, this reflects the fact that few businesses are open at that time of day. The regression coefficients again indicate that roadside development has its greatest influence on traffic flow during the afternoon hours.

Weighted number of intersections per mile was significant during each hour except the afternoon off-peak (when it was only slightly insignificant). The regression coefficients suggest that turning movements from intersections tend to become more of a delay problem over the noon hour and during the afternoon peak hour than during the remainder of the day.

The final travel time model to be tested combined the weighted number of intersections per mile with a breakdown of commercial floor area into two categories: weighted retail floor area per mile and weighted office floor area per mile. Speed limit again represented the traffic flow parameter:

$$UT = a + b(WRETFA) + c(WOFFFA) + d(WINTR) + e(SPEEDL) \tag{6}$$

Table 7 gives the resulting regression coefficients and statistics. The absence of a coefficient indicates that the variable was not significant at the 10 percent level. Once again, speed limit had the expected inverse relation with unit travel time.

The separation of commercial floor area into retail and office components yielded a significant increase in variance explanation for the morning peak hour but only minimal improvements for the remaining time periods. The regression coefficients suggest that the major cause of roadside friction during the morning peak hour is commercial establishments. This may be associated with the high percentage of work trips occurring during this hour and the fact that office and professional services begin their activities relatively early in the morning.

Weighted office floor area per mile was the only significant land use variable during the noon hour. This is probably because of the predominance of noon-hour lunch trips with respect to shopping trips. However, during the evening peak hour, retail land uses were the significant activities affecting unit travel time. This would indicate the importance of the many shopping-to-home trips at this time of day. During the two off-peak hours, both retail and office activities proved to be significant causes of delay to through traffic.

Weighted number of intersections per mile was significant at all times except the afternoon off-peak hour. The values of the coefficients indicate that the effect of intersection turning movements tends to peak during the evening rush hour. This would reflect the large number of trips destined for residential areas at that time of day.

CONCLUSIONS

Theoretically, the model containing weighted trip ends per mile as the land access variable should offer the best explanation of the variance in delay to through traffic. This is because trip generation rates directly reflect the entering and exiting traffic movements that actually cause delays. As given in Table 5, the variance explanation for the sample data ranged from a low of 36 percent during the morning peak hour to a high of 51 percent during the afternoon off-peak hour. However, the weighted trip-ends-per-mile variable was only significant in 3 of the 5 hours studied. The reason for these results can probably be attributed to the difficulties in estimating trip generation rates for specific sites based on very generalized data for a broad range of conditions. Nevertheless, the potential usefulness of these data certainly warrants further study of trip generation characteristics of individual land use activities.

The combination of weighted commercial floor area per mile and number of intersections per mile as the measure of level of land access along an arterial offered a measurable improvement in the variance explanation of the travel time models. Table 6 gives a low value of 41 percent during the morning peak hour and values between 50 and 55 percent during the remaining hours. In addition, the commercial floor area variable was significant during all hours except the morning peak. These improvements can be attributed to the strong relation between trip generation and the gross floor area of commercial activities. Thus, the results of the statistical analysis suggest that weighted commercial floor area per mile can be utilized as an acceptable and easily obtained measure of level of land access along arterial roadways.

The final land access parameter to be tested was the combination of weighted retail floor area per mile, weighted office floor area per mile, and weighted number of intersections per mile. Retail and office floor area was simply a dichotomous classification of commercial floor area. The most noticeable improvement in variance explanation

Table 4. Descriptive statistics for the 56 study sections.

	Hour									
	7:15 to 8:15 a.m.		10 to 11 a.m.		12 to 1 p.m.		2 to 3 p.m.		4:15 to 5:15 p.m.	
Variable	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
UT	1.71	0.16	1.74	0.18	1.77	0.19	1.73	0.19	1.78	0.23
WTRIPS	16.8	17.9	16.7	17.5	15.6	16.2	16.5	17.4	16.4	17.4
WCOMFA	154.0	260.0	150.0	239.0	145.0	238.0	149.0	235.0	147.0	231.0
WRETFA	137.0	254.0	133.0	232.0	128.0	231.0	131.0	228.0	130.0	224.0
WOFFFA	17.0	29.0	17.0	28.0	17.0	28.0	17.0	28.0	17.0	28.0
WINTR	8.2	7.6	8.0	7.4	7.7	7.0	7.9	7.4	7.7	7.4
V	610.0	352.0	375.0	93.0	473.0	96.0	442.0	117.0	712.0	266.0
SPEEDL	31.3	5.2	31.3	5.2	31.3	5.2	31.3	5.2	31.3	5.2

Table 5. Regression coefficients and statistics for Eq. 4.

	Factor			
Hour	a	$b \times 10^{-3}$	$c \times 10^{-2}$	R^2
7:15 to 8:15 a.m.	2.29	—	-1.87	0.36
10 to 11 a.m.	2.36	1.95	-2.08	0.43
12 to 1 p.m.	2.57	—	-2.58	0.47
2 to 3 p.m.	2.34	3.45	-2.12	0.51
4:15 to 5:15 p.m.	2.52	3.90	-2.58	0.49

Table 6. Regression coefficients and statistics for Eq. 5.

	Factor				
Hour	a	$b \times 10^{-4}$	$c \times 10^{-3}$	$d \times 10^{-2}$	R^2
7:15 to 8:15 a.m.	2.30	—	4.65	-2.03	0.41
10 to 11 a.m.	2.34	2.12	5.66	-2.17	0.52
12 to 1 p.m.	2.51	1.49	6.22	-2.61	0.55
2 to 3 p.m.	2.36	2.44	—	-2.11	0.50
4:15 to 5:15 p.m.	2.55	2.99	6.89	-2.76	0.54

Table 7. Regression coefficients and statistics for Eq. 6.

	Factor					
Hour	a	$b \times 10^{-4}$	$c \times 10^{-3}$	$d \times 10^{-3}$	$e \times 10^{-2}$	R^2
7:15 to 8:15 a.m.	2.18	—	1.86	3.91	-1.73	0.51
10 to 11 a.m.	2.29	1.76	1.36	5.22	-2.03	0.55
12 to 1 p.m.	2.50	—	1.39	5.43	-2.56	0.55
2 to 3 p.m.	2.30	2.04	1.42	—	-1.98	0.53
4:15 to 5:15 p.m.	2.57	2.94	—	7.00	-2.81	0.54

was again during the morning peak hour. The information given in Table 7 shows that this model was able to explain between 51 and 55 percent of the variation in unit travel time for the study conditions. The breakdown of commercial floor area into retail and office components did provide a better understanding of the roadside development-traffic service interrelation by revealing which types of commercial uses were significant during which hours. However, from the standpoint of reliably estimating unit travel time, the disaggregation of commercial floor area is probably an unnecessary refinement.

A comment is in order regarding the weighting of the land access measures. The weighting scheme was hypothesized prior to the analysis, and no attempt was made to statistically test the suitability of unweighted measures. Nevertheless, intuitively and in view of the results of the statistical modeling, the weighting scheme would appear to be justified. Certainly, further investigation under a wider variety of locations and conditions would be most desirable.

In summary, this research has shown the importance of roadside development as a constraint on the quality of traffic service provided by urban arterials. Furthermore, the travel time models that were developed can offer a starting point for the systematic evaluation of the consequences of alternative development proposals. Such a procedure could allow local engineers, planners, and public officials to arrive at better land use control decisions in areas of new urban growth.

ACKNOWLEDGMENTS

The authors would like to acknowledge the financial support of the National Science Foundation and the cooperation of the city of Madison Transportation Department, Traffic Engineering Division, for making this study possible.

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CHARACTERISTICS OF FREEWAY TRAFFIC AND OF FREEWAY LANE-CHANGING BEHAVIOR

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This paper gives a detailed account of important freeway traffic characteristics based on an analysis of aerial photographic data obtained on the Long Island Expressway and the Ventura and the Santa Ana freeways. These freeway traffic characteristics include time headway distribution, time speed distribution, space headway distribution, space speed distribution, space relative speed distribution, and bivariate histograms of space speed and relative space speed with space headways. This information was obtained for every lane and for various levels of traffic service and is presented both in graphical and in summary table forms. The report also gives an account of the nature of various traffic characteristics distributions and relates the bivariate histograms to safe driving rules. Also reported are microscopic aspects of lane-changing behavior obtained from the Long Island data. Discussions and summary statistics were made for the speeds, space headways, and relative speeds of the lane-changing vehicle and its neighboring vehicles. This was done for each lane and for levels of service B and C; some interesting relations were found among the different variables. Three risk criteria were applied that provided hazard measures of lane changes.

•IN THIS paper we shall present some important freeway traffic characteristics and discuss their relations and usefulness. Traffic data used are all aerial photographic data. Each filmed period (usually a few minutes) is considered as a constant flow period. By this we mean that the flow is influenced only by natural fluctuations of traffic rather than the change of actual flow level (say, from service level A to service level B). We shall first analyze in detail the headway, speed, relative speed, and the bivariate relations between the headway and speed and between the headway and relative speed and then follow the analysis of freeway lane-changing behavior. These terms as well as other terms used in the paper are defined as follows.

L is the stretch of the road section, and T is the time period in which we are interested. $C_i(x, t)$ is the i th vehicle passing x at time t such that x is in L and t is in T.

The time headway is the time difference of successive vehicles passing a fixed point x in L during the time period T. Or, we may write that

$$h_{ti} = t_i - t_{i-1},\ i = 1, 2, \ldots, M(T, x) \tag{1}$$

is the time headway of vehicle C_i at point x, where $M(T, x)$ is the total number of vehicles passing x in T, and t_i is the instant that C_i passes x.

The space headway is the distance between two successive vehicles at a given time instant t in T [measured from front-to-front bumper (1)].

$$h_{si} = x_i - x_{i-1},\ i = 1, 2, \ldots, N(t, L) \tag{2}$$

is the space headway of vehicle C_i at time t where $N(t, L)$ is the total number of vehicles in L at the instant of time t.

The traffic is time-homogeneous (constant flow) in T at x if the time headways (h_{ti}) form an independent and identically distributed random sequence during all time arrival epochs t_i at point x.

The traffic is space-homogeneous in L at t if the space headways form an independent and identically distributed random sequence for any pair (x_{i-1}, x_i).

The time speed v_{ti} of C_i is the speed of C_i at a fixed point x in L.

The space speed v_{si} of C_i is the speed of C_i at a fixed time t in T.

The time relative speed r_{ti} is the difference in speed of successive vehicles passing a fixed point x in L, or

$$r_{ti} = v_{ti} - v_{t,i-1} \tag{3}$$

The space relative speed r_{si} is the difference in speed of successive vehicles at a fixed time t in T, or

$$r_{si} = v_{si} - v_{s,i-1} \tag{4}$$

For time-homogeneous traffic, we define the flow q(x) at any point x in L to be

$$q(x) = \lim_{T\to\infty} \frac{M(T,x)}{T} \tag{5}$$

We also define the instantaneous concentration k(t, L) at time t in L to be

$$k(t, L) = N(t, L)/\|L\| \tag{6}$$

where $\|L\|$ is the length of the road stretch L.

The concentration over L is defined as the mean of k(t, L) over all t in T, or

$$k(L) = \lim_{T\to\infty} \frac{1}{T} \int_0^T k(t, L)dt \tag{7}$$

and the instantaneous average speed at fixed time t in L is given by

$$v(t, L) = \frac{\sum_{i=1}^{N(t,L)} v_{si}}{N(t, L)} \tag{8}$$

The time mean speed at x is

$$\bar{v}_t(x) = \lim_{T\to\infty} \sum_{i=1}^{M(T,x)} \frac{v_{ti}}{M(T,x)} \tag{9}$$

The space mean speed over L is defined by

$$\bar{v}_s(L) = \lim_{T\to\infty} \frac{\int_0^T v(t, L)k(t, L)dt}{\int_0^T k(t, L)dt} \tag{10}$$

It can be shown that, in time-homogeneous traffic flow,

$$q(x) = \bar{v}_s(L) \times k(L) \tag{11}$$

For space-homogeneous traffic flow, we define concentration and space mean speed in the usual way, i.e.,

$$k(t) = \lim_{L \to \infty} k(t, L) \tag{12}$$

and

$$\bar{v}_s(t) = \lim_{L \to \infty} v(t, L) \tag{13}$$

For time- and space-homogeneous traffic, $k(t)$ and $\bar{v}_s(t)$ in Eqs. 12 and 13 are independent of t and equal to $k(L)$ and $\bar{v}_s(L)$ respectively. We refer to Breiman (2) for detailed discussions and proofs of all the preceding relations.

Time statistics are fundamental statistics that are obtained directly from aerial data (with only data reduction and smoothing). On the other hand, space statistics are difficult to obtain even with the aid of aerial data. This is because we have neither an infinite length of road stretch with homogeneous traffic nor a means of photographing it—even if it were to exist. However, space statistics are more important than time statistics in describing traffic flow conditions and in analyzing relations among traffic characteristics with regard to freeway traffic operations and control. Approximate methods to obtain space mean speed and concentration from time statistics are given in Eqs. 14 and 15 respectively:

$$\bar{v}(x) = \frac{n}{\sum_{i=1}^{n} \frac{1}{v_{ti}}} \tag{14}$$

$$k(x) = q(x)/\bar{v} \tag{15}$$

where n is the number of vehicles passing the point x per unit time, and q(x) is the hourly flow at x.

In traffic engineering the operating characteristics are generally described in terms of level of services, i.e., operating conditions that are related to driving speed, comfort, convenience, economy, and safety. Detailed procedures are given in the Highway Capacity Manual (1) for the determination of level of service.

The Long Island Expressway aerial data showed little effect of either an on- or an off-ramp traffic flow; the Ventura Freeway and Santa Ana Freeway data showed an off-ramp traffic flow effect. Detailed descriptions of the physical sites and summary of data collected are given in the next section.

DESCRIPTION OF FREEWAY SITES AND DATA

Aerial data from three different locations are being presented in this report. The first is the three-lane westbound Long Island Expressway in New York. Various factors that affected the selection of this site were the level grade, free of curvature, nonsignificant effects of on- and off-ramps, and medium to high flow of traffic. The section of the Long Island Expressway between the interchange at Guinea Woods Road and Jericho Turnpike was the best possible site found after an extensive search.

The second is the four-lane westbound Ventura Freeway at White Oak Avenue in Los Angeles. The third is the three-lane southbound Santa Ana Freeway at Washington Boulevard in Los Angeles. Both Los Angeles sites have exit-ramp effects. All three sites are unidirectional with six, eight, and six lanes respectively in both directions.

Data Collection and Reduction

Aerial data of the Long Island Expressway were analyzed by U.C.L.A. and SDC for the purpose of a freeway analytical model study. The Santa Ana and Ventura Freeway data were analyzed by U.C.L.A. for a separate study of freeway exit-ramp effects.

Details of the photographic instruments, film-reading equipment and techniques, and the associated computer software used to develop trajectories of vehicles relative to an actual ground-based coordinate system are given by Tashjian and Knobel (3).

Summary of Data Collected

We shall give a summary of the time, number of frames and vehicle trajectories, and other pertinent information for each filmed period on the three sites for those films that are completely processed.

In Table 1, the first character of the film number indicates the freeway site; i.e., L indicates Long Island, V indicates Ventura, and S indicates Santa Ana. They are ordered within each site according to increasing amounts of flow (number of trajectories per frame).

The flow, concentration, and space mean speed given in Table 1 are computed based on Eqs. 14 and 15, and the level of service in the table is explained in detail in the Highway Capacity Manual (1).

PROCEDURES FOR COMPUTING SPACE STATISTICS

If one were to look at a fixed section of road, e.g., a 1-mile section, then he would miss all headways larger than 1 mile and possibly many of those that are less than 1 mile. This obviously would bias samples of headways, speeds, and other statistics that are obtained from aerial photographic data. Thus, aerial photographic data directly provide samples that can only be regarded as time statistics. However, space statistics are regarded as being more important than time statistics. For example, the relation $q = k\bar{v}$ for homogeneous traffic holds only for $\bar{v} = \bar{v}_s$ (space mean speed). This is one of the relations that underly the flow of traffic.

In order to obtain $\bar{v}_s$, one must study the relations of speeds and headways in the space domain for utilization in space relations governing traffic. However, one can obtain a measure of $\bar{v}_s$, if one selects a point of origin and looks at selected sequences of speeds downstream. It is noted that $\bar{v}$ using Eq. 14 involving harmonic mean of the time speeds would be an alternate way of obtaining the space mean speed. However, Eq. 14 does not provide probability distributions of space speeds.

Time statistics are easy to obtain. Only a smoothing of trajectories is required to get speed, headway, and relative speed at any fixed point on the road stretch. However, the calculation of space statistics is far more than trivial. The difficulty, basically, is that our data do not cover a long enough stretch of homogeneous freeway to be space data. They are actually time data taken at a fixed point with a small extension (on the order of 1 mile) into space. A naive approach to the estimation leads to serious errors. For instance, suppose we want to estimate the distribution of space headways in the outer lane. The obvious thing to do is to measure all space headways in each frame of a given freeway stretch and construct a histogram from these. Then the histogram should resemble the density of the underlying space headway distribution. However, the conclusions reached using this approach will not have a sound statistical basis. The main reasons are that there is a strong dependence between the headways of any car on successive frames as well as the dependence between successive car headways on the same frame. Also, the data are strongly biased in favor of the smaller headways because of the finite stretch of freeway covered. We refer to Breiman (4) for detailed arguments. Therefore, we use the following procedures to calculate space statistics.

For space headways, we looked at the pairs consisting of the R th and (R + 1) st space headways downstream every other time that a car passed a fixed space origin for the lane in question. Here R was taken as large as possible but fixed for every lane and for every run (in the range of 5 to 8, depending on lane and flow). These were recorded as space headways.

The distribution of space speeds was determined by looking at the sequence of speeds of the R th car downstream every time a car passed the origin.

The distribution of space relative speed distribution was determined by looking at the sequence of the differences of the speed of the (R + 1) st car and the speed of the R th car. The justification for this procedure is given elsewhere (4).

The mean and variances of the preceding statistics can be obtained by the standard procedures provided below:

$$\mu = \sum_{i=1}^{n} \frac{x_i}{n} \tag{16}$$

$$\sigma^2 = \sum_{i=1}^{n} \frac{(x_i - \mu)^2}{n} \tag{17}$$

where x_i is the i th sample of headway or speed (time or space), or space relative speed. The corresponding mean and variance are μ and σ^2.

GRAPHICAL REPRESENTATION OF FREEWAY TRAFFIC BEHAVIOR FOR SELECTED FILMS

For the Long Island Expressway data, there exist service levels A, B, and C. We have used films L1, L2, L6, and L8 as typical examples to represent the three service levels. Films L1 and L2 have an average flow of 2,213 vph, which is close to the maximum allowable volume in service level A. Film L6 has a flow of 2,852 vph, which is in the middle of service level B. Film 8 has a flow of 4,381 vph, which is at the maximum allowable volume in service level C for a peak-hour factor slightly higher than 0.91.

For the Ventura Freeway data, there exist service levels B, C, D, and E. Among them, B and C are chosen for detailed study because we are more interested in comparing them (four lanes with exit ramp) with the Long Island data (three lanes with no ramp). Films V1, V2, and V3 are used for service level B, and V4 and V5 are used for service level C. Films V1, V2, and V3 have an average flow of 4,533 vph, which is in the upper half of the volume range in service level B. Films V4 and V5 have an average flow of 5,453 vph, which is in the middle of service level C for a peak-hour factor of 0.91.

Because Santa Ana Freeway is also a three-lane freeway, it will also be interesting to compare its results with the Long Island and Ventura data. Unfortunately, only service level C is in common for all three sites. We thus select film S1 for detailed study. Film S1 has a flow of 3,808 vph, which is in the lower half of the volume range in service level C.

We note here that all probability density functions in the report are smoothed curves fitted over the empirical distributions. Only a limited number of density functions are presented graphically in this paper as examples because of space limitation. All others can be found elsewhere (5). However, our discussions are not limited to the examples.

Time Headway Distributions

Figure 1 shows the time headway distribution for the Long Island Expressway in service level C.

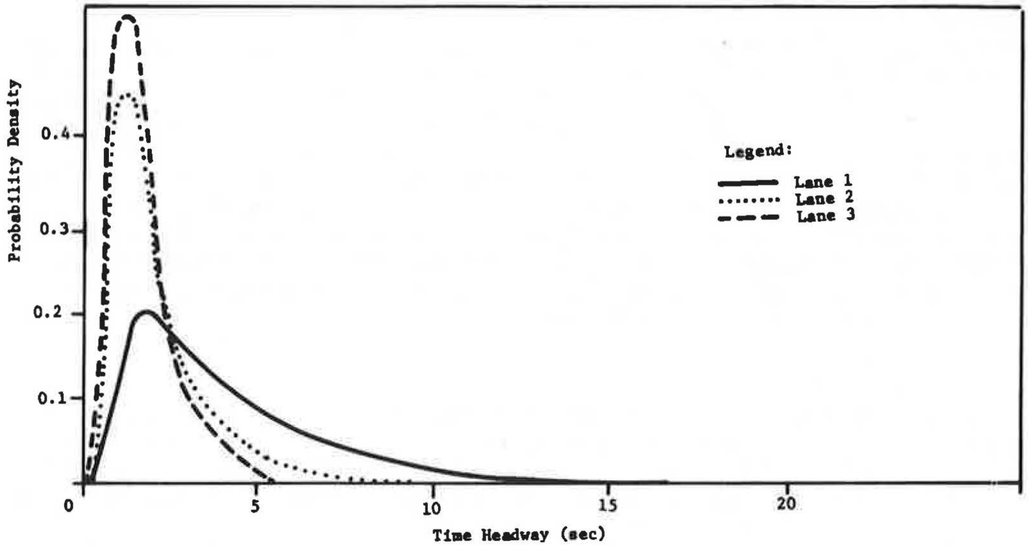
It was observed [for the Long Island Expressway data (6)] that, out of a total of 24 cases (three lanes, eight films), only 3 were rejected independence at the 5 percent level by the Spearman and Kendall rank correlation tests. This means that, for practical purposes,

Table 1. Summary of all films.

Film Number[a] (service level)	Date and Time	Number of Frames	Number of Trajectories	Flow (cars/hour)	Concentration (cars/mile)	Space Mean Speed (mph)
L1 (A)	6/10/69, 2:00 p.m.	297	340	2,135	35.7	59.9
L2 (A)	6/10/69, 10:30 a.m.	276	242	2,290	38.2	60.0
L3 (B)	6/10/69, 6:15 p.m.	351	508	2,633	42.0	62.7
L4 (B)	8/22/69, 9:20 a.m.	366	536	2,659	44.6	59.7
L5 (B)	6/10/69, 6:15 p.m.	176	268	2,839	47.3	60.1
L6 (B)	8/22/69, 5:55 p.m.	821	1,307	2,852	48.3	59.1
L7 (B)	8/21/69, 9:50 a.m.	849	1,393	2,953	51.1	57.8
L8 (C)	6/10/69, 8:15 a.m.	464	1,113	4,381	79.1	55.5
V1 (B)	4/21/70, 12:40 p.m.	492	661	4,443	69.9	63.6
V2 (B)	3/14/69[b]	515	715	4,532	73.1	62.0
V3 (B)	3/19/70, 12:40 p.m.	202	305	4,625	70.0	66.1
V4 (C)	9/18/68[b]	484	740	5,214	84.8	61.6
V5 (C)	3/25/69, 2:42 p.m.	478	823	5,691	89.6	63.6
V6 (D)	5/19/70, 4:00 p.m.	492	1,035	6,879	111.9	61.5
V7 (D)	3/31/70, 4:00 p.m.	406	896	7,170	118.2	60.7
V8 (E)	3/25/69, 4:35 p.m.	497	1,112	7,206	127.4	56.6
V9 (E)	9/18/68[b]	463	1,042	7,255	127.2	57.0
V10 (E)	5/5/70, 4:00 p.m.	477	1,063	7,369	126.9	58.1
V11 (E)	6/12/70, 4:25 p.m.	370	875	7,588	125.6	60.4
S1 (C)	7/9/68, 2:40 p.m.	404	652	3,808	64.0	59.5
S2 (D)	7/9/68, 1:30 p.m.	284	369	5,092	96.0	53.1

[a]L = Long Island, V = Ventura, and S = Santa Ana. [b]Times are unavailable.

Figure 1. Time headway distribution (Long Island Expressway, service level C).



we can assume the sequence of time headways to be independent. It was also observed (6) that the tail of the time headway distribution could be fitted to an exponential function. That is,

$$P(h_{t1} \geq t) = Ce^{-(t-t_0)/\beta} \qquad t \geq t_0 \tag{18}$$

where C is the total proportion of the headways that are greater than or equal to t_0, and t_0 and β are constants that depend on flow levels. The general trend is that the smaller the flow is, the larger the value of C is. A better fit could be obtained by other probability density functions, e.g., a semi-Poisson function (7). However, its form is much more complicated than the exponential type of distributions.

The outer lane (lane 1) always has a longer tail than do the inner lanes. The mean and the standard deviation of headway decrease with an increasing lane number. The headway distribution of the Long Island site (no ramp) is quite different from the other sites (with exit ramp). The former has a larger mean and standard deviation. This phenomenon occurs because fewer cars are using lane 1 for exiting purposes.

Time Speed Distribution

Unlike time headways, the successive time speeds are heavily correlated, even at moderate flow. The correlation is lane-dependent, increasing with increasing lane numbers. It was found (6) that, for the Long Island Expressway data, time speeds in all lanes are normally distributed.

The mean and the standard deviation are heavily lane-dependent. Lane 1 has the smallest mean and largest standard deviation, and the inner lane (lane 3 or lane 4, corresponding to a three- or four-lane freeway) has the largest mean and smallest standard deviation.

Because the level of service C is common for all three freeways, we want to compare the speeds for this service level only. Except for lane 1, traffic on the Santa Ana Freeway moves faster than the traffic on the Long Island Expressway. This could be due to the different driving characteristics of the drivers on these two freeways. The exceptional case (lane 1) results from the exit-ramp effect, in which more cars are using lane 1 at the Santa Ana site and cause larger disturbances than at the Long Island site. In comparing the four-lane Ventura Freeway to the three-lane Santa Ana Freeway, we observed that the former has higher mean speeds for all lanes than the corresponding lanes in the latter. Because these two freeways are in the same metropolitan area, it is safe to say that a four-lane freeway has better performance than a three-lane freeway under the same volume/lane traffic.

Space Headway Distributions

It was observed that the space headway distributions are very similar to those of the corresponding time headways. For the Long Island Expressway data, it was observed (6) that, out of a total of 24 tests, independence of space headways was rejected at the 5 percent level only three times—the same as in the case of time headways, except the rejected cases were different.

In fact, when the speeds are constant, the space headway distribution is exactly the same as the time headway distribution (1). Small variations in speeds will not affect the space distribution in such a way as to make it significantly different from the time distribution.

Space Speed Distribution

Figure 2 shows the space speed probability density distribution of the Long Island Expressway in service level C.

As in the case of time speed distributions, the sequence of space speeds is heavily correlated and normally distributed (6). However, because (1)

$$f_s(v) = \frac{\bar{V}_s}{V} f_t(v) \tag{19}$$

where $f_s(v)$ and $f_t(v)$ are the density functions of speeds measured in space and time respectively, $f_s(v)$ and $f_t(v)$ cannot both be normal except when $\bar{v}_s = v$. But, when $\bar{v}_s/v$ is very close to one, the normality of $f_t(v)$ implies that $f_s(v)$ is approximately normal. In fact, because $\sigma/\bar{v}_s$ is generally less than 0.1, it follows that most speeds are in the range where $\bar{v}_s/v$ varies from 0.9 to 1.1, and we conclude that both $f_s(v)$ and $f_t(v)$ have approximately the same type of probability density functions, i.e., the normal distribution.

Space Relative Speeds

Figures 3 is the probability density function of the relative speed in space of successive pairs of cars on the Long Island Expressway for service level C.

As in the case of space speeds, the relative space speeds are all approximately normally distributed with close-to-zero mean and standard deviation decreasing with increasing lane number. For service level C, the standard deviation of the relative speed for the Long Island Expressway lane 3 is significantly smaller than those of the corresponding lanes in the Ventura and Santa Ana freeways. This difference is reduced with the reduction in the lane number. Because relative speed is an indication of freeway disturbances, the preceding results indicate that the freeway traffic disturbances decrease from outer lanes to inner lanes. However, at light to medium traffic, the general base value of these differences in the standard deviation of the relative speed merely indicates the differences in the desired speeds of the successive drivers in the various lanes. The lower values in the inner lane indicate that drivers in that particular case tend to have the same driving habits as those in the shoulder lanes. As the traffic concentration increases, different successive drivers are forced to give away their individual desired driving habits in favor of the average driving habits and safety. This results in the lower values of the standard deviations of the speeds of the successive cars as we encounter higher concentration of traffic.

The standard deviations of the relative speeds for lanes 2 and 3 of the four-lane Ventura Freeway are almost the same, and consequently the lane-3 values for this freeway are much larger than the corresponding values for this lane in the three-lane Long Island and Santa Ana freeways. Furthermore, the lane-4 values of the four-lane Ventura Freeway are comparable to the lane-3 values of the three-lane Santa Ana Freeway.

Bivariate Histograms

We may intuitively expect that either one or both of the following conditions hold: Space headway is positively correlated to the follower's speed, and relative speed of successive cars (speed of the leader and speed of the follower) is positively correlated to the space headway between them. However, the bivariate histograms of speed versus space headway and relative speed versus space headway do not appear to have any obvious correlations for all cases except that the higher the average speed is, the higher the average space headway. Therefore, we only present a bivariate histogram of the Long Island Expressway service level C as shown in Figure 4. The hazard region shown in Figure 4 is derived from the California safe-driving rule, which is discussed later in the paper. It is observed that the proportion of cars violating the California safe-driving rule increases from about 13 percent in lanes 1 and 2 to more than 50 percent in lane 3, and the average for the three lanes is 33 percent.

Summary Tables

The aforementioned probability density functions, plus others that are given elsewhere (5), are summarized in tabular form for easy comparison.

Tables 2 and 3 are the summary of mean and standard deviation of the time and space statistics respectively for the Long Island Expressway data, whereas Tables 4 and 5 are those for the Ventura Freeway data. Table 6 gives the mean and variances of time and space statistics for the Santa Ana Freeway data, and Table 7 gives the mean and standard deviation of relative space speeds for all three sites.

Figure 2. Space speed distribution (Long Island Expressway, service level C).

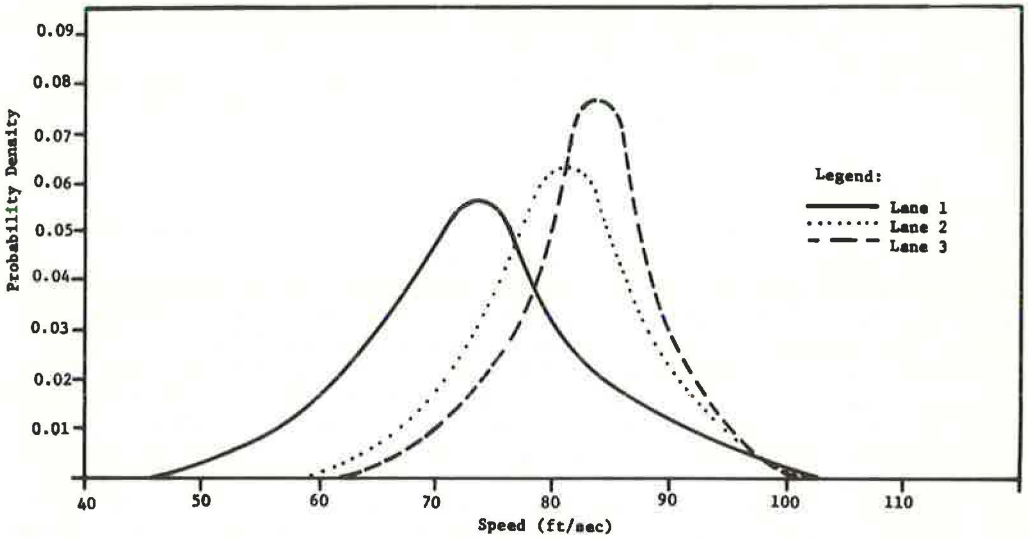


Figure 3. Relative speed distribution (Long Island Expressway, service level C).

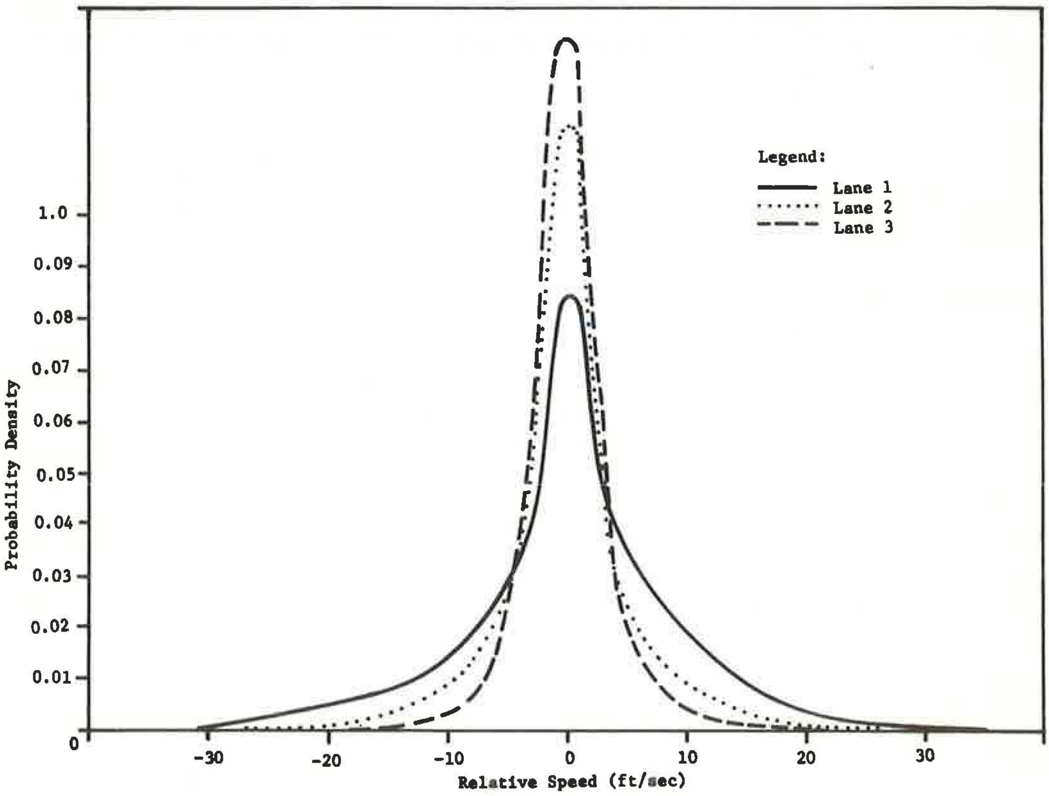


Figure 4. Speed versus space headway.

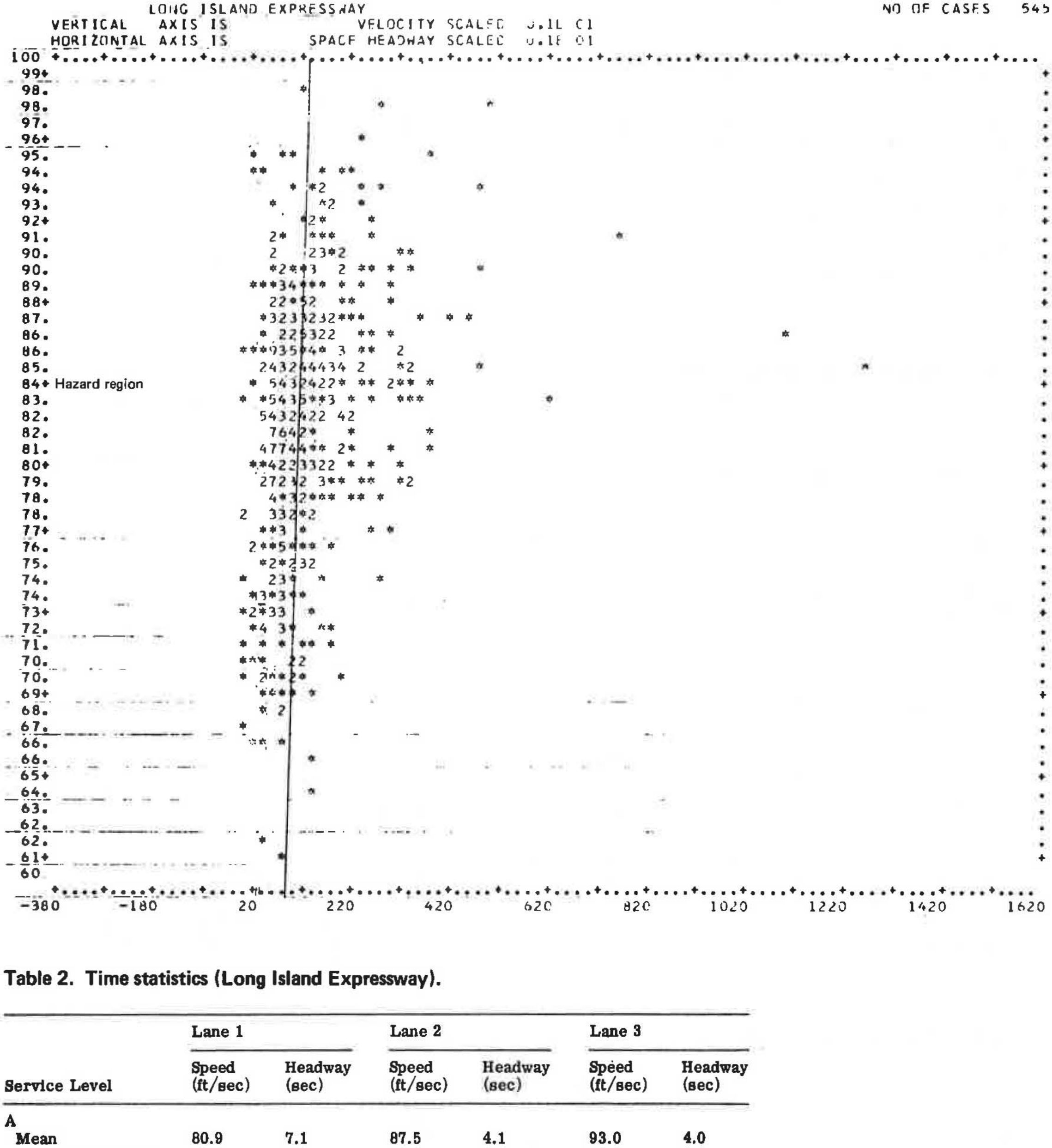


Table 2. Time statistics (Long Island Expressway).

	Lane 1		Lane 2		Lane 3	
Service Level	Speed (ft/sec)	Headway (sec)	Speed (ft/sec)	Headway (sec)	Speed (ft/sec)	Headway (sec)
A						
Mean	80.9	7.1	87.5	4.1	93.0	4.0
Standard deviation	9.6	5.3	6.6	4.1	5.8	4.1
B						
Mean	79.6	6.5	86.6	3.3	90.7	2.9
Standard deviation	10.1	4.9	7.1	2.5	5.4	2.8
C						
Mean	76.1	5.3	82.0	2.3	83.8	6.5
Standard deviation	10.8	4.4	7.2	1.8	1.7	1.2

Table 3. Space statistics (Long Island Expressway).

	Lane 1		Lane 2		Lane 3	
Service Level	Speed (ft/sec)	Headway (ft)	Speed (ft/sec)	Headway (ft)	Speed (ft/sec)	Headway (ft)
A						
Mean	79.2	572	86.0	338	91.4	393
Standard deviation	9.2	441	6.9	317	6.4	388
B						
Mean	78.3	460	85.5	286	89.5	257
Standard deviation	9.5	393	6.7	238	5.1	250
C						
Mean	74.4	343	81.9	182	82.5	152
Standard deviation	10.7	281	7.5	138	6.4	112

Table 4. Time statistics (Ventura Freeway).

	Lane 1		Lane 2		Lane 3		Lane 4	
Service Level	Speed (ft/sec)	Headway (sec)	Speed (ft/sec)	Headway (sec)	Speed (ft/sec)	Headway (sec)	Speed (ft/sec)	Headway (sec)
B								
Mean	81.2	4.3	91.7	3.2	100.6	3.0	105.2	2.9
Standard deviation	10.1	3.5	7.1	2.4	6.4	2.3	6.1	2.6
C								
Mean	83.8	3.1	91.4	2.8	98.1	2.5	102.3	2.3
Standard deviation	9.6	2.4	6.6	2.4	6.6	2.0	5.0	2.0

Table 5. Space statistics (Ventura Freeway).

	Lane 1		Lane 2		Lane 3		Lane 4	
Service Level	Speed (ft/sec)	Headway (ft)	Speed (ft/sec)	Headway (ft)	Speed (ft/sec)	Headway (ft)	Speed (ft/sec)	Headway (ft)
B								
Mean	79.3	334.9	90.6	296.2	99.3	294.0	104.1	294.9
Standard deviation	9.5	300	6.6	227	6.1	223	7.5	271
C								
Mean	80.8	253.1	90.0	247.9	97.2	248.0	101.2	237.6
Standard deviation	9.0	230	6.6	204	6.8	196	5.3	214

Table 6. Time and space statistics (Santa Ana Freeway, service level C).

	Lane 1		Lane 2		Lane 3	
Statistics	Speed (ft/sec)	Headway	Speed (ft/sec)	Headway	Speed (ft/sec)	Headway
Time						
Mean	74.0	3.3 sec	85.4	2.6 sec	93.1	2.2 sec
Standard deviation	10.0	2.8 sec	7.8	1.9 sec	5.0	1.7 sec
Space						
Mean	69.6	226.7 ft	84.7	199.2 ft	90.2	193.2 ft
Standard deviation	10.5	193 ft	10.8	121 ft	15.9	148 ft

It is noted that, in all cases, the mean of space speeds in a lane is smaller than the mean of time speeds in the corresponding lane. This is not surprising because

$$\bar{v}_t = \bar{v}_s + \frac{\sigma_s^2}{\bar{v}_s} \tag{20}$$

CHARACTERISTICS OF FREEWAY LANE-CHANGING BEHAVIOR

Lane changing is a common and important phenomenon in freeway traffic. It is the only operation that a driver can exercise other than the change in speeds. Both lane changing and speed changing are key traffic operations of multilane freeways; it is very important to understand them so that they can be predicted under various circumstances such as bottlenecks and other freeway disturbances. The extensiveness of lane changing has great impact on the quality of service and safety. In another study (8) we used aerial experimental data for freeway lane-changing model validation. In that study, we were more interested in the macroscopic aspects of the lane-changing behavior; that is, we are interested in the average frequency of lane changing for a given traffic situation. In this study, our focus is now on the microscopic aspects in order to understand various factors such as lane changer's speed relative to its neighboring vehicles, its clearance, etc., which may have caused lane-changing maneuvers. Use of Figure 5 can help us to define all the pertinent parameters. In this figure the circled numbers indicate vehicles that constitute the local environment of a lane changer. Vehicle 1 is the lane changer in its original lane, with the long arrow heading its destination lane. Vehicle 2 is the leader. Vehicle 3 is the accepted gap leader. Vehicle 4 is the accepted gap follower. Vehicle 5 is the opposing gap leader. Vehicle 6 is the opposing gap follower. Case A represents a lane change from lane 1 (the outer lane) to lane 2 (the middle lane), case B from lane 2 to lane 1, case C from lane 2 to lane 3 (inner lane), and case D from lane 3 to lane 2. Other variables shown in Figure 5 are defined in the following:

v_i = speed of vehicle i, i = 1, 2, . . . , 6, at the instant, e.g., t, vehicle 1 is making a lane change;
x_1 = the distance between vehicles 1 and 2 at t;
x_2 = the length of the accepted gap at t;
x_3 = the distance between the gap leader 3 and the lane changer 1 at t;
x_4 = the distance between the lane changer 1 and the gap follower 4 at t;
x_5 = the length of the opposing gap (x_5, x_6, and x_7 applicable in cases B and C only) at t;
x_6 = the distance between the opposing gap leader 5 and the lane changer 1 at t; and
x_7 = the distance between the lane changer 1 and the opposing gap follower 6 at t.

Aerial photographic data of the Long Island Expressway were used to obtain the aforementioned variables in a lane-changing process.

SUMMARY OF LANE-CHANGING STATISTICS

Because of space limitations we shall only present, in table form, all pertinent lane-changing statistics. Detailed graphical data are given elsewhere (9).

Gap Statistics

The mean values and standard deviations of gaps x_1 through x_7, for service levels B and C of the Long Island Expressway data, are given in Table 8. It was found that the type of distribution of the various spatial distances (x_i's) among the different vehicles during the lane-changing process are very similar to the space headway distribution of all vehicles (e.g., details are all negatively exponential), except that the mean and standard deviations are quite different.

Let us first look at service level B. The mean gap of x_1 in each lane is much lower than the mean gap of all vehicles in corresponding lanes. This shows that most lane changers are in a position too close to the leader prior to lane changing. The distribu-

Risk Criterion 1

The first criterion considered is based on measurement of the closing speed between a pair of car-following vehicles and the separation distance between them, namely v_{31} versus x_3 and v_{41} versus x_4. The pair would fall within the hazardous region if the separation distance between them was so small that the follower would have to decelerate at a rate greater than $a_c = 5$ ft/sec^2 to avoid hitting the leader. More precisely, for a pair of vehicles 1 and 3, we calculate

$$a = \frac{(v_1 - v_3)^2}{2(x_3 - 17.6)} u(v_1 - v_3) \tag{21}$$

and compare a with a_c. The number 17.6 is the average vehicle length (ft) and $u(\cdot)$ is the unit step function. Vehicle 1 is in the hazard region if $a \geq 5$ ft/sec^2. This is similarly done for the vehicle pair 1 and 4.

Risk Criterion 2

This criterion was defined (10) by considering a driver's response to a rapid braking maneuver of the leader, including an allowance for a lag in his response time. For the vehicle pair 1 and 3, the hazard region is given by

$$v_1T - (x_3 - L) + \frac{1}{2a}(v_1^2 - v_2^2) > 0 \tag{22}$$

where a is taken to be 10 ft/sec^2 and T to be 1 sec. The hazard region for the vehicle pair 1 and 4 can be similarly carried out.

Risk Criterion 3

The third criterion is commonly known as the California safe-driving rule, which requires that the following vehicle maintain a distance of at least one car length for every 10 mph of speed. In other words, the hazard region is defined as

$$v_1 > \frac{15}{L}(x_3 - L) \tag{23}$$

for the pair 1 and 3, and

$$v_4 > \frac{15}{L}(x_4 - L) \tag{24}$$

for the pair 1 and 4.

The percentage of vehicles that fall in the hazard regions by using the three criteria is summarized in Table 11.

As we note from Table 11, the values of a and b are very small in both service levels for criterion 1. This criterion may not provide an effective risk measure. The risks under criteria 2 and 3 are all significantly higher than the risk of all vehicles that form pairs of leader and follower. Approximately 37 percent and 42 percent of the drivers violated the California safe-driving rule at service levels B and C respectively as compared to 33 percent at service level C when all pairs of leader-followers are included. We can thus say that, based on the California rule, the lane changer is likely to take more risk to perform a lane change as compared to the usual following process. This is conceivable because lane changing accounts for only a brief moment in the entire travel period of a vehicle, and the driver can afford such a high risk. Furthermore, the 33 percent figure for drivers violating the California rule for service level C indicates that this rule is too conservative, and many drivers pay little attention to it.

SUMMARY AND APPLICATIONS

Aerial photographic traffic data of three different sites have been analyzed in detail. Probability density distributions of some important traffic parameters and their sum-

Table 9. Speed statistics (in feet per second).

Service Level	Variable	Lane 1 Mean	Lane 1 Standard Deviation	Lane 2 Mean	Lane 2 Standard Deviation	Lane 3 Mean	Lane 3 Standard Deviation
B	v_1	82.6	11.3	89.3	9.1	93.5	6.9
	v_2	73.0	11.1	84.5	7.6	90.5	6.0
	v_3	87.0	7.8	88.7	8.5	91.0	7.5
	v_4	84.8	8.3	85.1	10.9	85.3	6.7
	v_5			85.4	11.2		
	v_6			84.0	9.1		
Mean speed		78.3	9.5	85.5	6.7	89.5	5.1
C	v_1	79.9	11.9	82.3	10.1	82.8	8.8
	v_2	71.9	10.1	79.4	7.8	81.4	7.2
	v_3	81.4	7.9	81.5	9.6	82.4	8.1
	v_4	79.7	6.6	76.9	9.6	80.0	7.9
	v_5			80.2	9.1		
	v_6			80.5	9.0		
Mean speed		74.4	10.7	80.9	7.5	82.5	6.4

Note: Lanes 1, 2, and 3 are the original lanes of the lane changer except in the case of the mean speed of all vehicles.

Table 10. Relative speed statistics (in feet per second, service level C only).

Variable	Lane 1 Mean	Lane 1 Standard Deviation	Lane 2 Mean	Lane 2 Standard Deviation	Lane 3 Mean	Lane 3 Standard Deviation
v_{21}	-7.9	7.2	-3.0	7.8	-1.36	6.2
v_{31}	1.6	9.8	-0.9	9.8	-0.67	6.0
v_{32}	9.5	9.0	2.1	9.4	0.75	6.7
v_{41}	-0.13	10.1	-5.4	11.4	-3.04	8.0
v_{51}			-2.7	9.1		
v_{52}			0.7	7.4		
v_{61}			-2.4	10.2		

Table 11. Proportion of vehicles in hazard region.

Service Level	Criterion 1 a[a]	Criterion 1 b[b]	Criterion 2 a	Criterion 2 b	Criterion 3 a	Criterion 3 b
B	0	0.006	0.1536	0.1867	0.3253	0.4217
C	0.0125	0	0.3053	0.2741	0.3956	0.4486

[a]Risk of the vehicle pair 1 and 3.

[b]Risk of the vehicle pair 1 and 4.

mary tables are presented for some selected films.

The time and space headways have approximately the same type of probability density functions with decreasing mean and standard deviation when traffic volume increases or when the lane number increases (outer lane is lane 1). The successive headways are independent (with very few exceptions).

The speeds (time and space) are approximately normally distributed with increasing mean and decreasing standard deviation according to increasing lane number. When traffic volume increases, the mean and standard deviation both decrease (for the same lane). Successive speeds are heavily correlated.

The relative speeds (space measurements) are normally distributed with close-to-zero mean and standard deviation decreasing with increasing lane number and increasing volume.

Space speeds and relative speeds do not appear to bear any correlation with space headways. However, it does appear that more cars are violating the California safe-driving rule in the inner lane than in the outer lane.

Analysis of relevant parameters (speeds and space headways of the lane changer and its neighboring vehicles) at the moment a lane change was performed were made, employing the Long Island Expressway aerial photographic data of service levels B and C. The mean and standard deviations of the relevant parameters were tabulated. Discussions were made in terms of the relations among the parameters and the risk measure when a lane change is made.

Some of the materials we found in this study may be helpful in terms of traffic operation and control. Knowing the time and space headway distributions of vehicles in each service level enables us to have a better estimate of the available gap between successive vehicles. This will result in upgrading on-ramp control performance. That is, we could estimate the available number of gaps per hour for a given level of service and efficiently control the on-ramp flow.

The lane-changing analysis could be similarly carried out in the vicinity of a freeway on- or off-ramp, and the gap statistics can be compared with the current no-ramp freeway section. With the help of risk analysis and accident records, we would be able to identify the most hazardous region for operational movement.

The microscopic gap characteristics are also very useful in the digital simulation of freeway traffic. One of the most important elements in a digital simulation model is lane changing. The stochastic character of the lane changing can be easily taken into account by incorporating the probability distribution functions of all relevant parameters in the simulation model. In the long run, the lane-changing behavior in the simulation model will be statistically equivalent to the real traffic at corresponding service levels.

Moreover, the exact forms of speed and headway distributions are more useful and realistic than their mean values in characterizing vehicle speeds and headways. These distributions could be incorporated into a freeway traffic simulation model to produce more meaningful results.

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EVALUATION OF INTERSECTION-DELAY MEASUREMENT TECHNIQUES

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The purpose of this study was to evaluate two intersection-delay measurement techniques. As a result of an extensive literature review, two methods were chosen to be the most practical for field application: the Sagi-Campbell method, which determines "aggregate" intersection delay from measurements of inflow, outflow, and length of queues at various points during each cycle, and the Berry-Van Til sampling method, which measures stopped delay counts of the number of stopped vehicles at predetermined time intervals. A two-lane intersection approach in Arlington, Virginia, was recorded on closed-circuit real-time television and filmed simultaneously on time-lapse super-8 movie film for four 1-hour periods. Traffic conditions varied from extremely low to very high saturated flow. Data were extracted while videotapes were played back. Delays computed from the time-lapse photography were used as bases for comparison of results from either of the two test methods. In addition, results by the Sagi-Campbell method were compared with delays measured by the traffic flow meter. Relations between volume and delay and queue length and delay were investigated. Neither one of the methods produced consistent trends in predicting delays as compared to the traffic flow meter or photographic methods used as bases. Even the base methods failed to give consistent, direct relations.

•SEVERAL years of widespread use of the signalized intersections section of chapter six of the Highway Capacity Manual (HCM) (1) have revealed a variety of problems ranging from difficulties with adjustment factors for some specific conditions to entire concepts such as load factor, peak-hour factor, and level of service. Also, users of the HCM indicate that it tends to predict higher volumes than are actually attainable at a given level of service, which means that underdesign may well be occurring where the HCM is used for design purposes (2, 3, 4). The HCM is also being criticized because it does not give ample weight and direct consideration to factors such as delay, effect of pedestrians, number and width of lanes in each approach, and opposing traffic volumes.

The Federal Highway Administration (FHWA) in cooperation with the Highway Research Board (HRB) is sponsoring a new nationwide data-gathering and analysis effort to update the intersections data in the HCM. It is intended that the new data will be collected and analyzed while taking into consideration all possible factors that might prove relevant in the performance, operation, and capacity of intersections.

In order to determine the state of the art and to single out for evaluation potential study methods that might lead to simpler and more accurate results, we conducted a thorough literature review on the subject of intersection capacity and performance. This review provided very limited concrete conclusions as to better methods of determining levels of service at individual intersections. The majority of the authors seemed to conclude, however, that delay was the most desirable and tangible measure.

Ideally, it was desirable to find a method whereby, given a set of easily and simply observable traffic conditions at an intersection, it would be possible to determine the delay experienced under those conditions.

A special HRB advisory subcommittee working with FHWA staff reviewed several available methods of measuring level of service for individual intersections. The subcommittee concluded that delay was potentially the best general measure of level of service at intersections and that stopped delay was the most practical measure. Two methods of study of delay, the Sagi-Campbell method (5) and the Berry-Van Til sampling method (6), appeared to be most promising so far as simplicity and ease of uniform application throughout the country are concerned. The subcommittee recommended a two-stage pilot study: the first stage to select one of these methods of observation and data-collection techniques and the second stage to determine the variance of delay and service volumes measured by the selected method of observation under different conditions.

In the first stage, in addition to observations by the Sagi-Campbell method and the Berry-Van Til sampling method, data were to be collected by the traffic flow meter method (7) and the photographic method (8). These data were to be used as a base, and results by the other two methods were to be compared to either one of the base measurements. All four methods were to be tried concurrently on one intersection approach for a period of about 4 hours under variable traffic flow conditions, i.e., from light, off-peak flows to heavy peak-hour traffic observations. The selection of the study method for nationwide application was also to take into consideration factors such as comparable manpower requirements, complexity of field measurements and data analysis, and uniformity of results under varying traffic conditions.

During the second stage, the selected method would be applied to several intersection approaches and under several traffic flow conditions. Several replications would be conducted at each intersection approach.

This report describes the findings of the first phase of the recommended study.

DATA COLLECTION AND ANALYSIS

Because of the high manpower requirements of conducting all four study methods of observation concurrently and the unreliability of the weather in the Washington, D.C., area during the winter months, it was decided to investigate the possibility of using real-time closed-circuit television photography. The data would then be extracted while playing back the video tapes for each of the study methods. The television real-time method was tested for a 1-hour study. Results from data extracted from the television playback varied by less than 5 percent from the field manual counts taken during the same period (the difference most likely being due to errors in the field manual counts); therefore, the television real-time method was accepted as a satisfactory substitute to manual field observations.

The photographic field data were collected on February 8, 1972, from 10:30 a.m. to 5:30 p.m. The southbound approach of the intersection of Jefferson Davis Highway and 23rd Street in Crystal City, Arlington, Virginia, was photographed for four 1-hour periods from the top floor of a high-rise office building. The approach is a two-lane, typical approach providing right- and left-turn movements without any special turning lanes or signal phases reserved for turning movements. The distance to the upstream signalized intersection is approximately 700 ft. The signal, although traffic-actuated, operated as fixed-time throughout the study period. There was no evidence of any signal coordination or traffic progression. The cycle length was 74 sec with a 36-sec green-and-yellow phase and a 38-sec red phase for the southbound approach. Figure 1 shows a photograph of the intersection taken from the television screen. Traffic conditions varied from extremely low volumes at about 11:00 a.m. to very heavily congested (oversaturated) conditions between 4:00 p.m. and 5:00 p.m. The weather was clear and cold. A Shibaden video camera, recorder, and monitor were used for the real-time closed-circuit television recording, and a Minolta super-8 movie camera system, operating at one frame per second, was used for the time-lapse photography.

During the last two observation periods (from 3:00 p.m. to 4:02 p.m. and from 4:15 p.m. to 5:17 p.m. when traffic conditions were changing from light to heavy and extremely heavy), short-duration manual counts were conducted at the street level by both the Sagi-Campbell method and the Berry-Van Til sampling method to identify any difficulties that might be experienced by field personnel during periods of heavy traffic-flow conditions and evaluate comparative ease of conducting each method. Data from these short-period counts were compared with data extracted from the video tapes and the movie film as a further test.

Data from the video tapes were extracted by viewing each 1-hour tape on a 16-in. television screen for each method of analysis. Data were collected by lane and by cycle for the traffic flow meter method and for the Sagi-Campbell method and by 1-min intervals for the Berry-Van Til sampling method. Tables 1 and 2 give a summary of the data collected by the various methods.

Traffic Flow Meter Method

Basically, the traffic flow meter consists of four digital counters and one elapsed-time recorder. The digital counters record the number of vehicles entering the section of a single traffic lane under study, the number of vehicles leaving the study section, the difference between the number of entering and leaving vehicles, and the number of accumulated vehicle-seconds to pass from the "in" point to the "out" point. A vehicle was recorded in the in counter as it joined the end of a queue at the intersection approach, and it was maintained there until it cleared the intersection. If there was no queue, the vehicle was recorded in at the instant its front wheels crossed the stop line, and it was recorded in the out counter the instant it cleared the intersection. In cases where a vehicle was not delayed while going through the study section, the in and out actuations were almost simultaneous. For every second the vehicle was within the study section, 1 vehicle-sec was accumulated on the vehicle-second counter. If two vehicles were within the section, 2 vehicle-sec were accumulated every second. At the end of every cycle, the number of in and out vehicles and the accumulated vehicle-seconds were recorded. The data were summarized for intervals of eight cycles, adding up to 10 min. The average delay per vehicle in seconds, for every 10-min interval, was calculated by dividing the total vehicle-seconds accumulated during the interval by the number of the vehicles out.

Sagi-Campbell Method

The Sagi-Campbell method of calculating intersection delay requires the following measurements by lane:

1. The number of vehicles in the queue in the approach of the intersection during the whole length of the cycle or until the queue is dissipated (count includes all the vehicles waiting when the light turns green plus the vehicles joining the queue during the green phase, and vehicles are considered to be in the queue when they are noticeably slowed down as they approach the tail of the queue),
2. The number of vehicles in the queue waiting at the beginning of each red phase,
3. The number of vehicles going through the intersection during each green plus yellow phase,
4. The total cycle length, and
5. The length of the red phase.

In this study, when volumes were low, one observer per lane could make all the required measurements. During periods when traffic volumes were high and queue lengths exceeded 20 to 25 vehicles per lane, one observer counted the queue length and another the outflow per lane. The number of vehicles waiting in the queue at the beginning of each red phase was determined by subtracting the outflow from the total queue length during each oversaturated cycle. The data were summarized for intervals of eight cycles, and the total delay for each interval was calculated by the equation

$$D = \sum_{j=1}^{N} D_j = \frac{R}{2}\left(\sum_{j=1}^{M} Q_j + \sum_{j=1}^{P} V_{oj}\right) + C \sum_{j=1}^{P} A_{j-1}$$

Figure 1. Study intersection.

Table 1. Traffic characteristics and delays (lane 1).

Period Number	Cycle	Beginning of Period	Total Volume	Trucks and Buses (percent)	Left Turns (percent)	Average Queue Length (vehicle per cycle)	Delay per Vehicle (sec) Flow Analyzer	Sagi-Campbell	Berry-Van Til Sampling	Photo Stopped	Photo Aggregate
1	1 to 8	10:53 a.m.	35	8.6	20.0	1.8	8.4	7.2	6.5	5.3	6.3
2	9 to 16	11:03 a.m.	47	8.5	8.5	2.5	9.2	8.1	–	6.3	8.0
3	17 to 24	11:13 a.m.	37	10.8	8.1	2.1	10.9	10.3	10.8	7.1	9.2
4	25 to 32	11:23 a.m.	61	9.8	16.4	4.2	13.2	10.3	–	8.2	11.0
5	33 to 40	11:33 a.m.	51	15.7	9.8	3.8	12.7	11.2	9.4	–	–
6	41 to 48	11:43 a.m.	55	18.2	3.6	2.8	10.8	7.5	–	–	–
7	1 to 8	12:03 p.m.	65	9.3	6.2	4.6	15.3	10.7	12.1	9.6	14.8
8	9 to 16	12:13 p.m.	46	4.3	13.0	3.2	12.3	10.5	–	7.2	9.8
9	17 to 24	12:23 p.m.	46	15.2	24.0	4.1	19.7	11.9	16.2	13.5	18.2
10	25 to 32	12:33 p.m.	59	8.5	13.5	3.5	8.0	8.7	–	4.6	6.7
11	33 to 40	12:43 p.m.	44	13.6	4.5	1.6	7.4	5.9	7.5	5.1	6.9
12	41 to 48	12:53 p.m.	47	14.9	29.8	5.2	26.7	18.6	–	18.1	25.2
13	1 to 8	3:01 p.m.	52	10.9	2.2	2.8	12.1	8.2	11.3	8.7	11.7
14	9 to 16	3:11 p.m.	57	5.3	10.5	3.1	10.7	8.5	9.8	7.3	10.2
15	17 to 24	3:21 p.m.	59	18.6	11.9	3.0	10.1	7.5	8.1	6.9	9.6
16	25 to 32	3:31 p.m.	79	19.0	1.3	4.9	12.0	9.5	12.7	7.5	12.1
17	33 to 40	3:41 p.m.	68	16.2	1.5	3.5	11.9	8.1	9.0	8.3	11.3
18	41 to 48	3:51 p.m.	72	12.5	1.4	4.1	10.0	8.8	8.4	6.8	9.1
19	1 to 8	4:16 p.m.	90	4.4	1.1	9.1	22.0	19.8	18.4	12.1	–
20	9 to 16	4:26 p.m.	98	11.2	1.0	14.6	41.3	36.4	34.2	23.4	–
21	17 to 24	4:36 p.m.	103	3.9	2.9	27.0	77.9	93.4	75.7	44.4	–
22	25 to 32	4:46 p.m.	105	4.8	4.8	30.6	102.8	119.4	77.2	57.7	–
23	33 to 40	4:56 p.m.	121	5.8	3.3	26.5	70.6	87.2	50.2	36.9	–
24	41 to 48	5:06 p.m.	115	4.3	3.5	27.0	72.8	78.4	54.2	39.3	–

where

D = total delay in vehicle-seconds for the interval;
M = the number of undersaturated cycles;
P = the number of oversaturated cycles;
N = $M + P$ = the total number of cycles in the interval;
R = the length of the red phase in seconds;
j = any cycle (jth) in the interval, $j = 1$ to $j = N$ (here, 8);
Q_j = the number of vehicles in the queue (includes both stopped vehicles and those that were noticeably slowed down and applies only to undersaturated cycles where the queue is dissipated before the beginning of the red phase);
V_{oj} = the outflow per cycle (applies only to oversaturated cycles where the queue does not dissipate before the beginning of the red phase);
A_j = the number of vehicles in queue at the beginning of the red phase;
A_0 = the initial A_j at the beginning of the study period; and
C = the cycle length in seconds.

The average delay per vehicle for every 8-cycle or 10-min interval was determined by dividing the total delay for that interval by the total number of vehicles that went through the intersection.

Berry-Van Til Sampling Method

Data for the Berry-Van Til sampling method were collected during three 10-min alternate intervals for each of the first two 1-hour light traffic periods and for six 10-min intervals for each of the last two 1-hour periods. One of the two observers per lane counted and recorded the number of stopped vehicles at the approach at the end of every 20-sec interval. The other observer recorded the outflow volume during each minute and classified the vehicles as stopping and nonstopping. The total delay for each 10-min interval was determined by adding all the stopped vehicles counted in the 20-sec intervals and multiplying the total by the interval period (20 sec). The average delay per vehicle for each 10-min sample was determined by dividing the total delay by the outflow, i.e., the number of vehicles counted during the 1-min intervals.

Time-Lapse Movie Film Method

Two different types of delay were calculated from the time-lapse movie films: a pure stopped delay, where a vehicle was considered being delayed only if it was actually stopped (locked wheels), and an "aggregate" delay, where a vehicle was considered being delayed from the time its speed was affected by the intersection condition (when it slowed down to join the end of the queue) until it cleared the intersection. This aggregate delay includes deceleration time, certain travel time while the vehicle is moving slowly in a platoon, acceleration time while the vehicle leaves the intersection, and stopped time.

Stopped delay was determined by counting the number of movie film frames in which each vehicle was stopped during every cycle. The total number of frames counted for all stopped vehicles during each cycle provided the delay per cycle in vehicle-seconds because the movie was taken at one frame per second. The average stopped delay per vehicle was determined by adding the total delays per cycle for eight cycles and dividing by the number of vehicles going through the intersection during the eight-cycle period.

Similarly, the aggregate delay per cycle was determined by summing the products of the number of vehicles in the queue by the number of frames of the same queue length. For example, if there were 3 vehicles in a queue for 6 frames, 4 vehicles for 20 frames, 3 vehicles for 3 frames, 2 vehicles for 1 frame, and 1 vehicle for 1 frame before the queue was dissipated, the total aggregate delay for that cycle would be 110 vehicle-sec.

The average aggregate delay per vehicle for each 10-min interval was determined by adding the total delays for each cycle as calculated previously and dividing by the number of vehicles going through the intersection during that interval.

RELATIONS EXAMINED

Although the HRB advisory subcommittee concluded that stopped delay was the most practical measure of level of service at intersections, it was not made clear what was considered to be stopped delay. Also, the Sagi-Campbell and the traffic flow meter methods were found not to lend themselves well for "pure stopped" delay analyses. Therefore, the relations of several degrees of delay were examined. The Sagi-Campbell, the traffic flow meter, and the photographic aggregate delay methods reflect delays that include some deceleration and acceleration periods. The Berry-Van Til sampling method was thought to reflect pure stopped delay, and it is comparable to the photographic stopped delay method that is considered to represent the minimum conceivable delay at the intersection.

Six types of relations were investigated for each lane:

1. A comparison of delays computed by the various methods of analysis (the five methods described previously) for each 10-min interval of the study period (Figs. 2, 3, and 4),
2. The volume-delay relation computed by the five methods (Figs. 5 and 6),
3. The average queue length-average delay relation computed by the five methods,
4. The volume-percentage variation of delay relation (using the traffic flow meter as base),
5. The volume-percentage variation of delay relation (using the photographic aggregate delay as base), and
6. The volume-percentage variation of delay relation (using the photographic stopped delay as base).

RESULTS AND FINDINGS

The following findings reflect interpretation of the initial plots of the field data:

1. During the low traffic-volume periods, from 11:00 a.m. to about 4:00 p.m., the delays computed by any of the five methods were generally uniform, ranging from 6 sec per vehicle to 35 sec per vehicle (Figs. 2, 3, and 4). The photographic stopped delay method produced the lowest delays. The Sagi-Campbell method generally was the next higher, although in some periods it dropped below the photographic stopped delay and in some others it produced the highest calculated delay. The photographic aggregate delay method was higher than the photographic stopped delay, and its curve followed the same pattern. The Berry-Van Til sampling method produced lower delays than the photographic aggregate method; however, it was not consistent. During several periods it was higher than the photographic aggregate method although the range of its variation was narrower. The traffic flow meter method produced the highest delays, and its curve followed the general pattern of the two photographic methods.

During the high traffic-flow period of the study, the data showed neither uniformity in, nor consistent relations among, the delays computed by the various methods. Although the Sagi-Campbell method produced the highest delay for the left lane (lane 1) during this period, it was considerably lower than the traffic flow meter in the right lane (lane 2). The Berry-Van Til sampling method produced delays that did not follow a consistent pattern of variation from either of the base delay methods.

2. When average delays, determined by the five methods, were plotted against volumes for 10-min periods (Figs. 5 and 6), there was considerable variation in delays, not only among methods but even within each method for the same volume. The Sagi-Campbell method, for example, produced delays of 8, 12, 19, and 26 sec per vehicle at a volume range of 46 to 50 vehicles per 10-min period and delays of 13 and 16 sec per vehicle at a volume range of 61 to 66 vehicles per 10-min period. This inconsistency applies to all methods, including the base methods. The highest average delay, 120 sec per vehicle, occurred at a volume of 105 vehicles per 10-min period. Above this volume, the average delay per vehicle dropped. At 115 vehicles per 10-min period, the average delay per vehicle was 73 sec.

3. The average queue length-delay relation showed some trend, although not consistently (Tables 3 and 4). As the average queue length per 10-min period increased,

Table 2. Traffic characteristics and delays (lane 2).

Period Number	Cycle	Beginning of Period	Total Volume	Trucks and Buses (percent)	Right Turns (percent)	Average Queue Length (vehicle per cycle)	Delay per Vehicle (sec) Flow Analyzer	Sagi-Campbell	Berry-Van Til Sampling	Photo Stopped	Photo Aggregate	Comment
1	1 to 8	10:53 a.m.	74	10.8	15.0	3.2	10.7	6.8	8.6	6.8	8.4	Bus loading (cycle 3)
2	9 to 16	11:03 a.m.	71	18.3	18.3	4.9	16.5	10.8	—	10.3	14.7	
3	17 to 24	11:13 a.m.	69	18.8	21.8	5.2	17.0	15.1	15.7	11.1	15.5	
4	25 to 32	11:23 a.m.	75	14.7	21.4	5.7	20.4	12.4	—	14.5	19.1	
5	33 to 40	11:33 a.m.	80	10.0	21.2	6.9	19.4	12.4	15.8	—	—	
6	41 to 48	11:43 a.m.	69	21.2	14.5	4.9	18.1	10.7	—	—	—	
7	1 to 8	12:03 p.m.	79	13.9	21.6	9.5	34.9	16.3	29.4	19.8	32.0	Bus loading (cycle 3)
8	9 to 16	12:13 p.m.	78	12.8	14.1	7.1	19.2	20.6	—	12.1	17.2	
9	17 to 24	12:23 p.m.	82	11.0	20.8	7.5	21.0	18.2	15.4	12.0	17.8	
10	25 to 32	12:33 p.m.	87	10.3	23.0	9.4	26.8	17.4	—	14.4	21.7	
11	33 to 40	12:43 p.m.	66	13.6	18.2	5.5	18.2	11.3	17.1	13.0	18.3	
12	41 to 48	12:53 p.m.	76	13.2	14.5	8.5	25.6	22.4	—	16.6	22.4	
13	1 to 8	3:01 p.m.	90	12.2	17.8	8.4	23.0	13.7	19.3	14.9	22.7	
14	9 to 16	3:11 p.m.	76	7.9	17.1	6.1	21.4	13.2	15.7	12.2	17.8	
15	17 to 24	3:21 p.m.	75	10.6	16.0	6.2	17.5	11.8	16.4	11.2	15.5	Bus loading (cycle 21)
16	25 to 32	3:31 p.m.	83	9.6	15.7	13.0	39.0	44.2	38.7	25.5	33.2	Bus loading (cycle 27)
17	33 to 40	3:41 p.m.	89	5.6	12.4	8.0	19.4	15.4	18.4	12.8	16.3	
18	41 to 48	3:51 p.m.	80	11.2	13.7	9.2	26.5	16.2	22.6	16.5	23.8	
19	1 to 8	4:16 p.m.	92	4.3	10.9	18.4	65.3	65.5	56.0	—	—	Bus loading (cycle 4)
20	9 to 16	4:26 p.m.	77	6.5	11.7	16.9	70.8	66.6	59.3	—	—	Right turn interference
21	17 to 24	4:36 p.m.	94	7.5	7.5	25.1	115.2	105.1	97.8	—	—	(road construction
22	25 to 32	4:46 p.m.	100	3.0	14.0	26.7	119.2	107.4	76.3	—	—	equipment)
23	33 to 40	4:56 p.m.	105	3.8	14.3	22.7	102.0	78.4	75.5	—	—	
24	41 to 48	5:06 p.m.	95	2.1	12.6	26.8	112.0	109.3	83.3	—	—	Bus loading (cycle 47)

Figure 2. Delays during off-peak periods (lane 1).

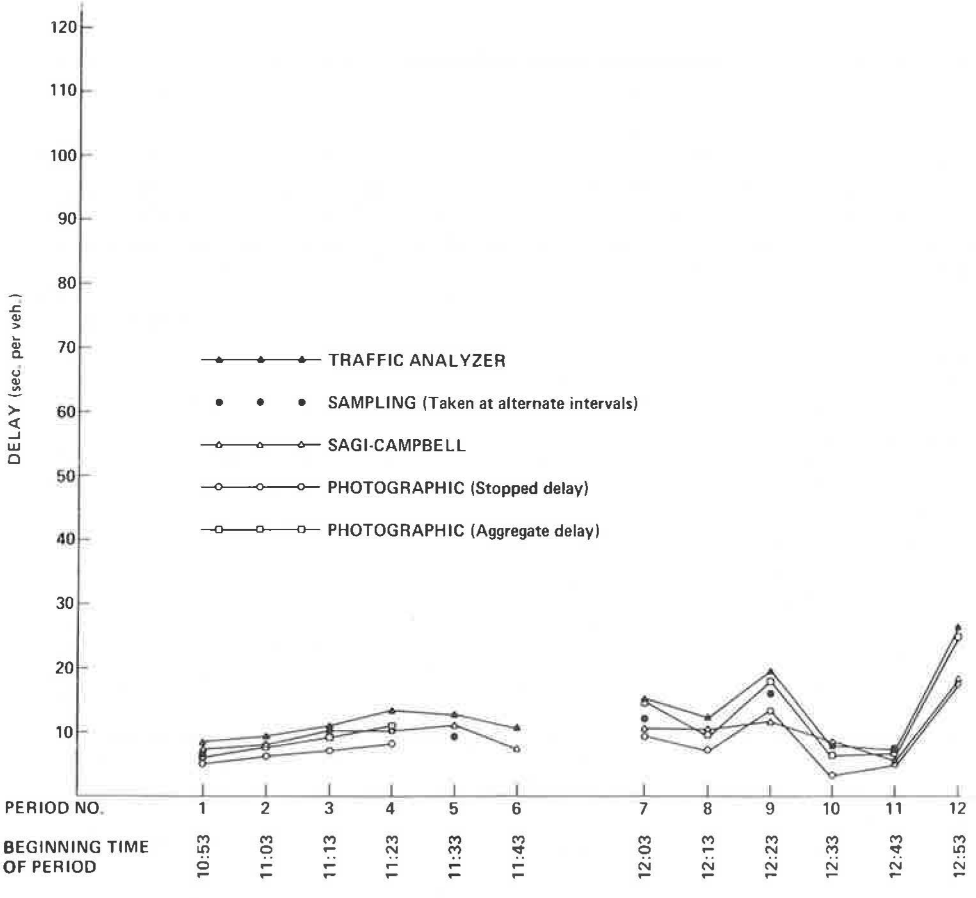

Figure 3. Delays during off-peak and peak periods (lane 1).

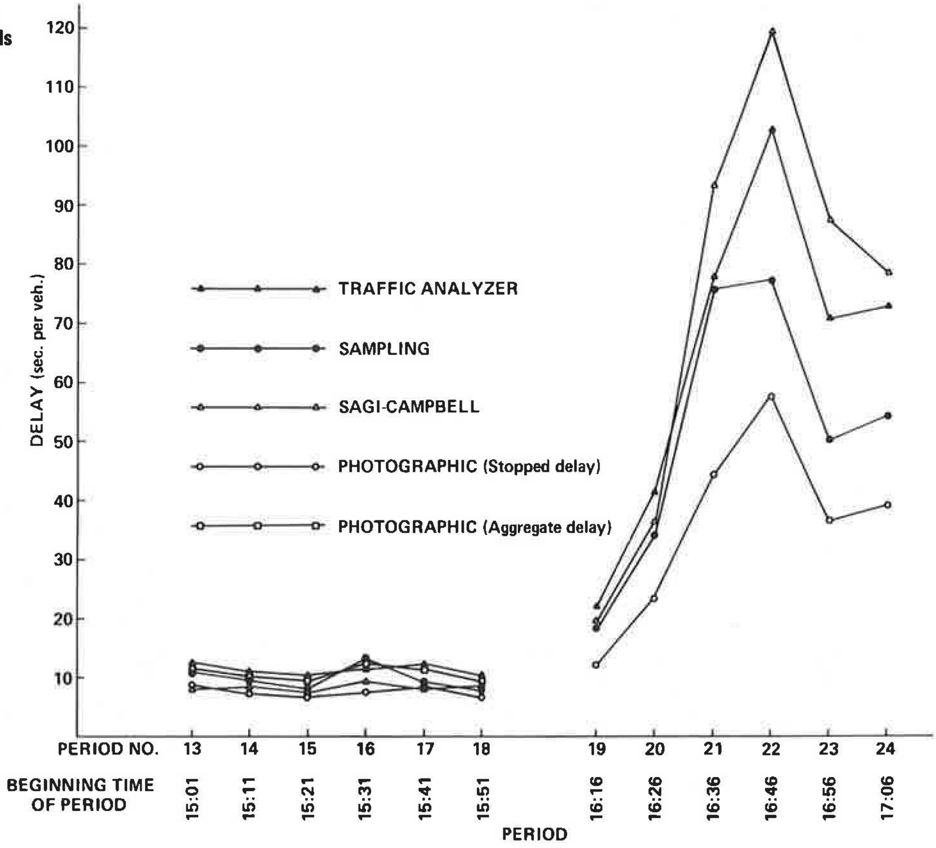

Figure 4. Delays during off-peak and peak periods (lane 2).

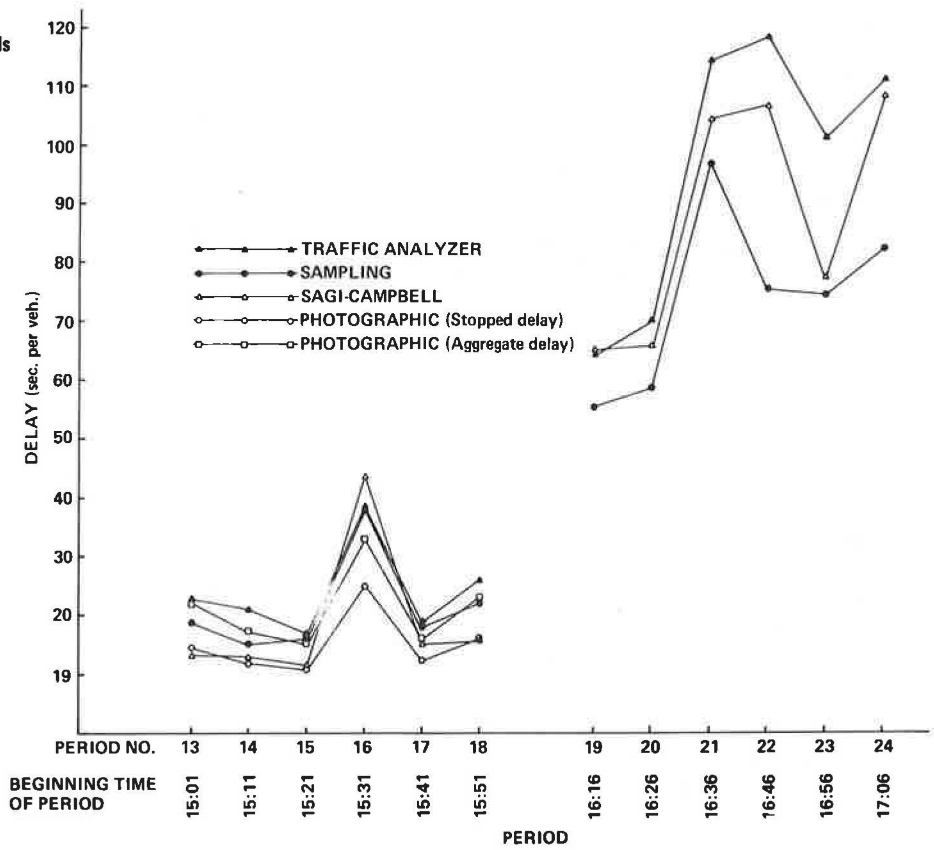

Figure 5. Volume-delay relation (lane 1).

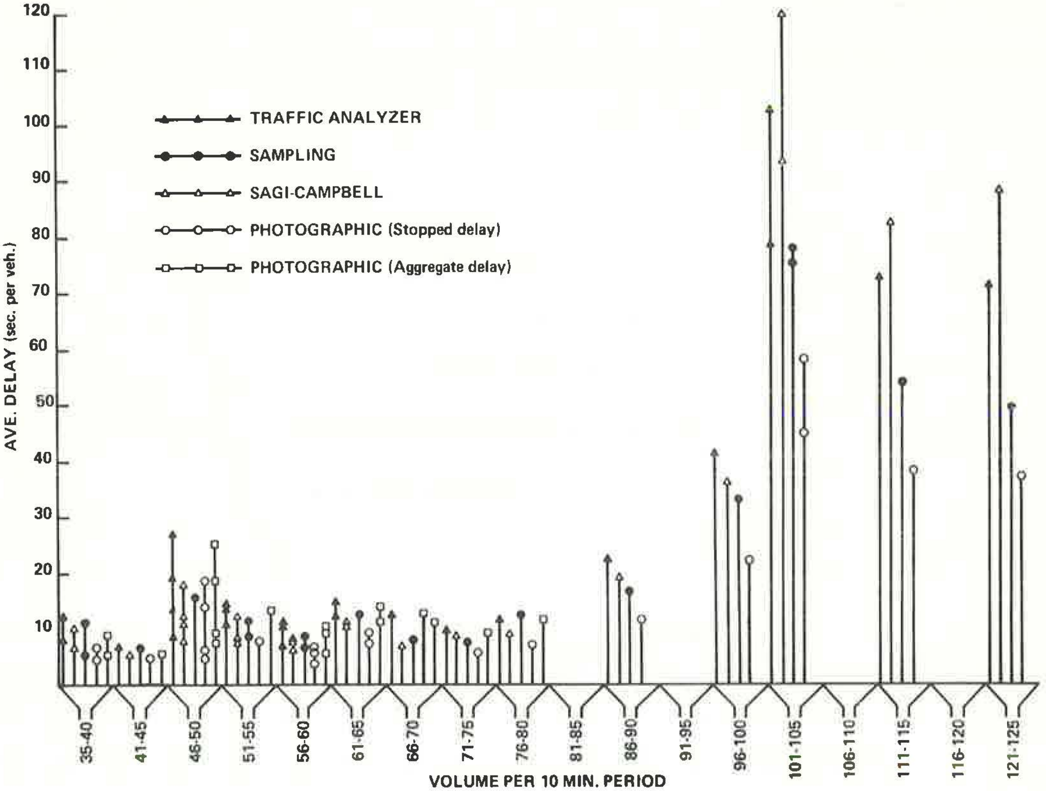

Figure 6. Volume-delay relation (lane 2).

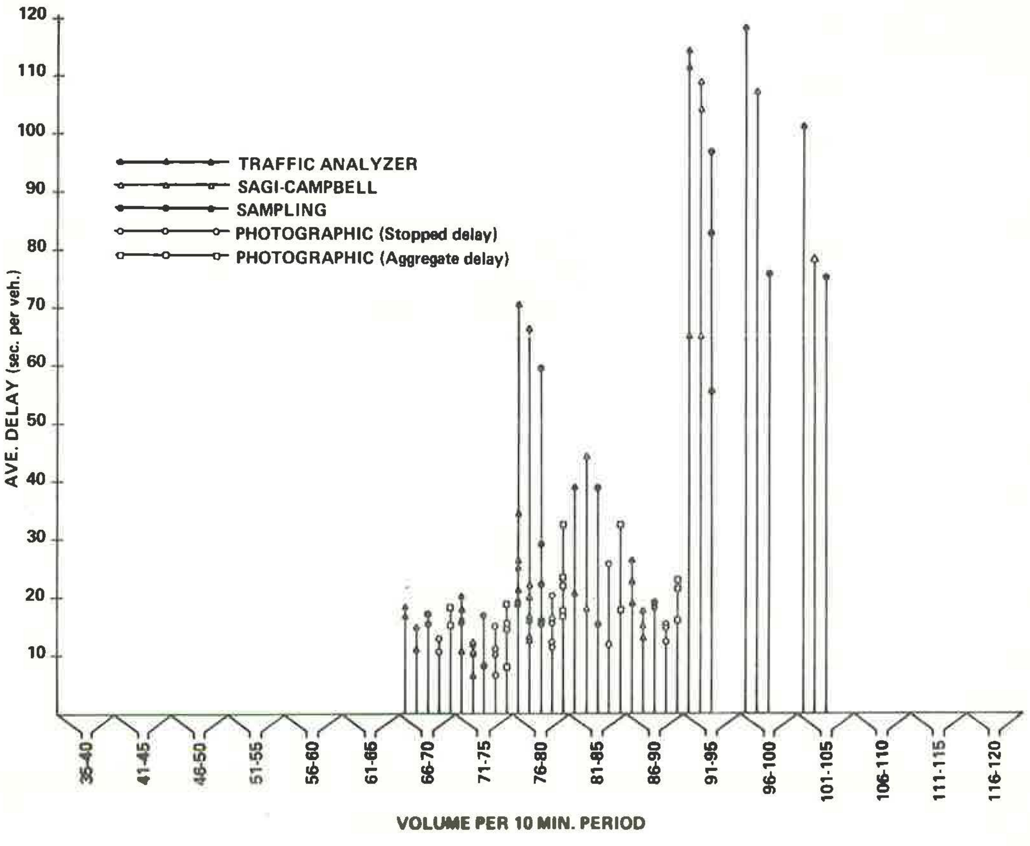

Table 3. Average queue length-delay relation (lane 1).

Period	Average Queue (vehicle per cycle)	Delay per Vehicle (sec) Flow Meter	Sagi-Campbell	Berry-Van Til Sampling	Photo Stopped	Photo Aggregate
11	1.6	7.4	5.9	7.5	5.1	6.9
1	1.8	8.4	7.2	6.5	5.3	6.3
3	2.1	10.9	10.3	10.8	7.1	9.2
2	2.5	9.2	8.1	–	6.3	8.0
6	2.8	10.8	7.5	–	–	–
13	2.8	12.1	8.2	11.3	8.7	11.7
15	3.0	10.1	7.5	8.1	6.9	9.6
14	3.1	10.7	8.5	9.8	7.3	10.2
8	3.2	12.3	10.5	–	7.2	9.8
10	3.5	8.0	8.7	–	4.6	6.7
17	3.5	11.9	8.1	9.0	8.3	11.3
5	3.8	12.7	11.2	9.4	–	–
9	4.1	19.7	11.9	16.2	13.5	18.2
18	4.1	10.0	8.8	8.4	6.8	9.1
4	4.2	13.2	10.3	–	8.2	11.0
7	4.6	15.3	10.7	12.1	9.6	14.8
16	4.9	12.0	9.5	12.7	7.5	12.1
12	5.2	26.7	18.6	–	18.1	25.2
19	9.1	22.0	19.8	18.4	12.1	–
20	14.6	41.3	36.4	34.2	23.4	–
23	26.5	70.6	87.2	50.2	36.9	–
21	27.0	77.9	93.4	75.7	44.4	–
24	27.0	72.8	78.4	54.2	39.3	–
22	30.6	102.8	119.4	77.2	57.7	–

Table 4. Average queue length-delay relation (lane 2).

Period	Average Queue (vehicle per cycle)	Delay per Vehicle (sec) Flow Meter	Sagi-Campbell	Berry-Van Til Sampling	Photo Stopped	Photo Aggregate
1	3.2	10.7	6.8	8.6	6.8	8.4
2	4.8	16.5	10.8	–	10.3	14.7
6	4.9	18.1	10.7	–	–	–
3	5.2	17.0	15.1	15.7	11.1	15.5
11	5.5	18.2	11.3	17.1	13.0	18.3
4	5.7	20.4	12.4	–	14.5	19.1
14	6.1	21.4	13.2	15.7	12.2	17.8
15	6.2	17.5	11.8	16.4	11.2	15.5
5	6.9	19.4	12.4	15.8	–	–
8	7.1	19.2	20.6	–	12.1	17.2
9	7.5	21.0	18.2	15.4	12.0	17.8
17	8.0	19.4	15.4	18.4	12.8	16.3
13	8.4	23.0	13.7	19.3	14.9	22.7
12	8.5	25.6	22.4	–	16.6	22.4
18	9.2	26.5	16.2	22.6	16.5	23.8
10	9.4	26.8	17.4	–	14.4	21.7
7	9.5	34.9	16.3	29.4	19.3	32.0
16	13.0	39.0	44.2	33.7	25.5	33.2
20	16.9	70.8	66.6	59.3	–	–
19	18.4	65.3	65.5	56.0	–	–
23	22.7	102.0	78.4	75.5	–	–
21	25.1	115.2	105.1	97.8	–	–
22	26.7	119.2	107.4	76.3	–	–
24	26.8	112.0	109.3	83.3	–	–

the average delay per vehicle increased with some exceptions. At an average queue length of 5.5 vehicles in lane 2, the average delay per vehicle was 18 sec by the traffic flow meter method and 11 sec by the Sagi-Campbell method. At a queue length of 7.1 vehicles, the average delays were 19 sec by the traffic flow meter method and 20 sec by the Sagi-Campbell method. At an average queue length of 8.4 vehicles, the respective average delays were 23 and 14 sec.

In lane 1 the Sagi-Campbell method produced the highest average delay of 119 sec per vehicle at a queue length of 30.6 vehicles; in lane 2 the traffic flow meter method produced the highest average delay. Similar inconsistencies occurred in delays computed by the other methods.

4. The average delays by the Sagi-Campbell method and the Berry-Van Til sampling method were expressed as percentages of the delays determined by the traffic flow meter and were plotted against volume per 10-min periods. The points did not follow any particular trend for either of the methods. The delays by the Sagi-Campbell method varied from 60 percent of the flow meter delay at a volume of 47 vehicles per 10-min period to 123 percent at a volume of 122 vehicles. Delays by the Berry-Van Til method varied from 71 percent at the flow meter delay at a volume of 120 vehicles to 101 percent at a volume of 42 vehicles per 10-min period.

5. When the average delays by the Sagi-Campbell and the Berry-Van Til sampling methods were expressed as percentages of the photographic stopped delay and the photographic aggregate delay and were plotted against the corresponding volumes, again no trends could be detected.

Although the principal objective of the study was a practical evaluation of the field application of the two delay measurement techniques, a statistical analysis of the data was performed to investigate validity of the methods. The analysis was aimed to determine if the true mean values of any two methods differ significantly at the 5 percent level of significance under the standard analysis-of-variance assumptions. The results of this analysis indicate that in every case the two techniques under comparison do not measure the same traffic delay information.

Another objective of this phase of the study was to compare the ease of conducting the Sagi-Campbell method and the Berry-Van Til sampling method of delay study in the field.

The Sagi-Campbell method of observation is more suitable when conducted in conjunction with capacity measurements. Data are collected on a cyclic basis, the same observers can be used to collect capacity data, and some of the data can also be used for capacity analyses. This method, however, is more difficult than the sampling method to explain to field personnel, and it requires considerable subjective judgment on the part of field personnel. The summarization and analysis of the data are also more time-consuming. The ability of personnel to observe accurately when queue lengths increase to about 20 vehicles drops considerably; i.e., in saturated conditions the accuracy of the method becomes questionable.

The Berry-Van Til sampling method is easy to explain to field personnel, and it requires no subjective judgment on their part. The data summarization and analysis are simple and easy to perform. This method, however, requires more field personnel to conduct because very little of the information can be used in capacity considerations. Most of the capacity data have to be collected on a cyclic basis. This method of observation also fails to produce reliable results during saturated flow periods. When the queue lengths exceed 20 vehicles, observers have difficulty counting the number of stopped vehicles in the queue.

CONCLUSIONS AND RECOMMENDATIONS

The study failed to produce results that would identify either the Sagi-Campbell method or the Berry-Van Til sampling method of field observation as obviously better than the other. Neither of the two methods produced consistent trends in measuring delay as compared to the base methods. Neither of the methods proved to be consistently simpler or easier to conduct in the field than the other under all traffic conditions.

The study cast some serious doubts as to the suitability of using delay as a level-of-service measure for individual intersections, at least given its current development. The subject of delay, as related to intersection performance, appears to require considerable research before it can be used as an operational tool in a nationwide intersection capacity data-gathering study.

Factors that appear to require research include the following:

1. The relations between, and differing influences of, the two fundamental types of delay found on urban intersection approaches (delay caused specifically by conditions at the intersection under study and delay caused by lack of coordination with upstream signals); and
2. The influence of differing driver populations at low volumes as compared with high volumes or mixes of populations at a given time.

There is increasing evidence—in the literature, in the results of FHWA's own research findings from simulation in connection with the urban traffic control system project, and in the preliminary findings herein reported—that research would show delay to be a more accurate and suitable indicator of overall system level of service (a long section of an arterial or a complete street network) than it is of service provided by an individual intersection within that system. This would be in keeping with the caution already expressed in the HCM that level of service for an individual intersection is a rather artificial thing because by definition level of service is an "over-a-distance," not "point," phenomenon.

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DISCUSSION

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Users of chapter six of the HCM have expressed various degrees of dissatisfaction with a number of the adjustment factors utilized in the analysis technique, with specific reservations regarding the concept of level of service, load factor, and peak-hour factor. Although some of these difficulties with the application of the HCM are real, many can be resolved if the user would apply engineering judgment in the selection of factors and the application of the analysis techniques to best fit the characteristics of the intersection under study. A number of critics have maintained that other methods utilizing measures such as lane headway, saturation flow, or delay produce more accurate results than the HCM procedures; however, most of these methods are more complex and unwieldy to use. What is needed is the simplest and most accurate method to provide a useful analysis tool for the practitioner. Furthermore, some of these techniques are only valid at capacity or saturation flow and do not provide a measure of level of service or performance.

The authors of this paper started with the premise that delay was potentially the best general measure of level of service at intersections. The first step was to select the method of observation and data-collection techniques, and four study methods were singled out for evaluation. The second step was to determine the variance of delay and service volumes under different conditions. Their conclusions would tend to indicate that the project was a failure because it produced no significant relations between volume and delay. To the contrary, I feel that their work has produced significant information that may lead to a breakthrough in refining and simplifying intersection capacity analysis techniques. The following observations are made with regard to the authors' work:

1. Sophisticated equipment and techniques and extensive manpower requirements are necessary to conduct pilot studies that will evaluate delay as a measure of level of service. It is questionable whether nationwide studies could be initiated within the limitation of the manpower and equipment available to city and state agencies throughout the country.
2. There is a need to further identify and describe what is meant by the term delay and what is the most accurate and simplest measure of delay. This paper showed poor correlation among all measurement techniques in their ability to predict or measure delay, which emphasizes the need to define delay in an accurate, reliable manner.
3. The results of this paper would cause one to question the validity of delay as a consistent measure because all techniques fail to produce a direct relation between delay at any given level of volume input or output. Logic might seem to explain why a variety of delays can be achieved for any given level-of-service volume. Not only are we dealing with driver populations in different locations, but also there is a high variability of types of drivers, habits, decisions, and characteristics exhibited during various periods of the day at the same location.
4. This paper did not fully describe the influence of the mix of straight and left-turn movements in the left lane with the interference of the opposing flow and the effect of the mix of straight and right-turn movements in the right lane. Although these factors may have been identified, it is not clear whether the statistical techniques employed or considered were successful in isolating these and other factors that can influence or produce significant variations. Seemingly, the raw data collected would provide some basis for analysis to explain, at least partially, the obvious differences.

The results of this paper point out the need for a series of carefully designed and controlled pilot studies to determine a relatively consistent measure or measures of level of service, be it load factor, delay, headway, or "green bananas." The factor of delay or other potential measures of level of service should not be discarded as a subject of research until we are able to filter out the extraneous "noise" and gain a better understanding of the variability of driver populations by location and time. It is essential that these series of pilot studies be made and that the study techniques and procedures be carefully defined prior to the initiation of extensive nationwide studies. The results of conclusive pilot studies must serve as a basis for developing the more extensive data-collection projects to ensure that these efforts will utilize consistent measuring techniques, collect compatible data, and produce a valid interpretation and evaluation of the results. Finally, the factors that are selected as a measure of level of service must lend themselves to the limitations of the equipment and manpower available to the practitioners who must collect the data and eventually to the capabilities of the users who will need to apply the analysis techniques devised.

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The results of the study addressed in this paper are most interesting and most beneficial to the profession. Certainly, it was amply demonstrated that the techniques by which delay is measured are too ambiguous in their definition. Perhaps what is needed is a single, operational definition of delay so that a base line would exist. Other types of delay could then be related to this measure, and other measurement techniques could then be evaluated on the basis of how close they came to the base line.

In discussion with the authors, they have correctly pointed out that the techniques considered defy clear and direct correlation. Moreover, they point to a significant research role to resolve questions raised by their study. This difficulty and these recommendations are in themselves a major contribution.

It is appropriate to review the data available from this study for insights that go beyond the intended and defined task of the original work. If this is done, it is discovered that the various delays are very highly correlated, the several techniques are dependent on the total volume and other parameters, but with differing sensitivities for each technique, and the various delays can be systematically interrelated. This is in conflict with the conclusion that "... the data showed neither uniformity nor consistent relations between the delays computed by the various methods," although the simple and desirable "uniformity" was certainly lacking.

These conclusions can be illustrated by first considering the following definitions:

Method	Lane 1	Lane 2
D_1 = flow analyzer delay	X_1 = 10 min, total volume	Y_1 = 10 min, total volume
D_2 = Sagi-Campbell delay	X_2 = 10 min, trucks and buses	Y_2 = 10 min, trucks and buses
D_3 = Berry-Van Til delay	X_3 = 10 min, left turns	Y_3 = 10 min, right turns
D_4 = photo stopped delay		
D_5 = photo aggregate delay		

The delays are in seconds per vehicle and the volumes in 10-min counts. Percentages for trucks and buses and for turns are not used. Queue data were not listed in the draft on which this analysis is based.

The data were considered in two groups, group 1 (periods 1 to 18) and group 2 (periods 19 to 24), that correspond to flow levels.

For the group 1, lane 1 data, typical sample correlation coefficients are D_1, D_2 = 0.92, D_1, D_4 = 0.99, D_2, D_4 = 0.90, and D_3, D_4 = 0.89. This illustrates the strong interrelations among the various delay measures. Figure 7 shows one of the stronger correlations. As indicated by the sample correlation coefficients, the correlations of the Sagi-Campbell and Berry-Van Til delays to the photo stopped delay are not as extreme but are rather strong.

Consideration was also given to relating each delay to the three available traffic variables by regression analysis: $D_i = \hat{\alpha}_{0i} + \hat{\alpha}_{1i}X_1 + \hat{\alpha}_{2i}X_2 + \hat{\alpha}_{3i}X_3$ for lane 1 and similarly for the Y_j in lane 2. Any term whose coefficient could not cause the hypothesis $\alpha_{ik} = 0$ to be rejected at a significance level of 0.05 was dropped.

Table 5 gives the results of the analysis for the group 1 data for four delay measures. Runs were not made for D_4 (photo stopped delay) in this particular analysis.

Note that, in the lane 1 data, the flow analyzer and Sagi-Campbell delays are sensitive only to the left-turn volume. This factor alone accounts for 50 percent or more of the variance. However, they have different sensitivities: Every left-turning vehicle adds 0.73-sec-per-vehicle delay to the flow analyzer measure but only 0.46-sec-per-vehicle delay to the Sagi-Campbell measure.

The Berry-Van Til measure is statistically insensitive to any traffic variable over the range of data. (Recall that all of these are per-vehicle delays, so that total delay is increasing in all cases as volume increases.) Based on the formulas developed, one would expect the Sagi-Campbell delay to be less than the Berry-Van Til delay for left-turn volumes of 6 or less per 10 min.

The photo aggregate delay is more complex, depending on both the left-turn volume and the trucks-and-buses volume.

The lane 2 situation is interesting in that not one of the four delay measures is discernibly dependent on the variables considered.

Based on this analysis, it is apparent that the several delay measures considered do indeed measure the same sort of thing (that is, delay) but that each emphasizes a different aspect of it. The situation is thus ambiguous, and the authors sensibly question this ambiguity from a practitioner's point of view. Certainly it is not safe to proceed with a major data-collection effort until a single, relevant definition of delay (and the several measures related to it) is accepted. This may require collection of supplemental data. If, for instance, photo aggregate delay were the standard and the data were collected in terms of photo stopped delay (an unrealistic but illustrative pair), the best linear conversion, $D_5 = -2.52 + 1.42D_4 + 0.04X_1$, for group 1, lane 1 would also require information on total volume (which would be collected routinely in any case, but a more subtle variable might also enter and would require additional collection for maximum precision).

It is appropriate to consider the disquieting conclusions of the authors as to current utility with the positive emphasis that their data have provided for directions in the research component they recommend: The path for this analysis of functional relations among the measures and the traffic variables is clear, and the prospect for success is heartening. The negative conclusions on existence of "consistent, direct relations" would seem to be too strongly stated.

It is important to systematically investigate all reasonable potential determining variables before proceeding with a major data-collection effort. These variables should include opposing volumes, upstream offset and cycle length, downstream queue extent (in heavy-flow situations), and component volumes and compositions. It would be most advantageous to study the basic relations in a controlled experiment—a simulator such as UTCS-1 would be appropriate—eliminate variables if possible, and return to a field validation of the type conducted by the authors.

The opportunity to present this discussion is greatly appreciated, and the authors are thanked for the discussions we have had on this subject. Robert L. Siegel is also thanked for the execution of the regression analysis programs.

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One of the most important contributions of the HCM was the introduction of the level-of-service concept to capacity analysis. However, the implementation of new concepts is often difficult and encounters diversity of opinion. This has been the case when the level-of-service concept was applied to signalized intersections. Load factor was selected as the measure of level of service at signalized intersections because it was available in the previous intersection capacity studies and it is relatively easy to measure. However, its use has been criticized because results at high flow-capacity ratios are not consistent, results are dependent on the arrival distribution of vehicles as well as on the intersection itself, and the driver does not consider load factor as the measure of his level of service.

As the authors of this paper have indicated, some form of delay appears to be the single most important measure of level of service from the viewpoints of the driver and those undertaking intersection capacity analysis. Therefore, the authors are to be commended for undertaking this study of evaluating various techniques for measuring intersection delay before embarking on a new nationwide data-gathering and analysis effort to update the intersections chapter of the HCM.

A summary table of all measurements obtained for the twenty-four 10-min data-collection periods including individual vehicular delays (seconds) calculated by five different methods is contained in the paper. It was unfortunate that measurements and calculations for all time intervals and for all methods were not obtained and a complete comparative analysis was possible. The use of television video tapes combined with time-lapse photography was a very excellent method of collecting data. One of its advantages is the ability of replaying the films to collect data.

My review of the paper has resulted in several questions that the authors may wish to answer in their closure:

1. The percentage of trucks and buses as well as the percentage of vehicles turning left or right was measured for each time interval and for each lane. Were these data included in the analysis?
2. In the Berry-Van Til method, the number of stopped vehicles was recorded at 20-sec intervals. How was this sampling interval selected, and how does it affect the accuracy of the method?
3. The paper reports that statistical analyses were undertaken to test for significant differences among the mean delays obtained by the various methods. Was the analysis conducted for each individual lane and for each hour of measurements? How were missing data handled in this analysis?
4. Did the authors consider combining the lane data and performing the analysis on an approach basis? In this connection, what was the capacity of this approach according to the HCM?

The paper has stimulated me to undertake some additional analyses that may be of interest and perhaps will suggest possible directions for future research. The results presented should not be considered as complete or final because this is not the intent, and the complete data base was not available to this author at this time. Perhaps the authors of the original paper, who have the original data either on film or in tabular form, may wish to extend this analysis further.

First, statistical analysis was undertaken to test for significant differences between mean delays as calculated for different hourly periods and different methodologies. Table 6 gives a summary of the results of the significant tests. Although there are differences between the significance levels, there appear to be patterns. The mean delays as calculated by one method are consistently larger (or smaller) than the corresponding mean delays as calculated by another method for the various hours of observations. For example, the stopped delay (as obtained by the photographic technique) was always numerically less than the aggregate delay (also obtained by the photographic technique). This led the author to suspect that, although there were some comparisons that showed significant differences in mean delays by the various methods, there still might be significant relations between mean delays so that one method could be used to estimate the results of another. Consequently, a series of linear regression analyses between selected mean delays was undertaken.

Table 7 gives a summary of the results of the linear regression analysis investigations. Four comparisons were made among mean delays for lanes 1 and 2 and the combined observations of lanes 1 and 2. The resulting linear equations, the correlation coefficients, and the sample size are given in the table for each investigation. The results for the most part are very encouraging and give some evidence that one method might be used in the field to estimate the mean delay of another method. The linear relations are shown in Figures 8, 9, 10, and 11.

The authors are encouraged to continue their efforts in this two-stage pilot study leading toward a nationwide data-gathering and analysis effort to update the intersections chapter of the HCM. Much has been done, but much is left to be done. Through the continued efforts of such researchers as the authors, we can look forward to improved methods for capacity analyses.

I wish to acknowledge the assistance of Maxence Orthlieb in making the analyses for this discussion paper.

Donald S. Berry, Northwestern University

Average delay per vehicle obtained by sampling the number of stopped vehicles every 20 sec was found by the authors to be approximately 40 percent higher than the corresponding stopped-time delays obtained from time-lapse photography. This is rather surprising because a similar comparison reported for two intersection approaches in 1954 (6) and 1956 (9) yielded stopped-time delays via manual sampling that were

Figure 7. Group 1, lane 1 data for two delay measures.

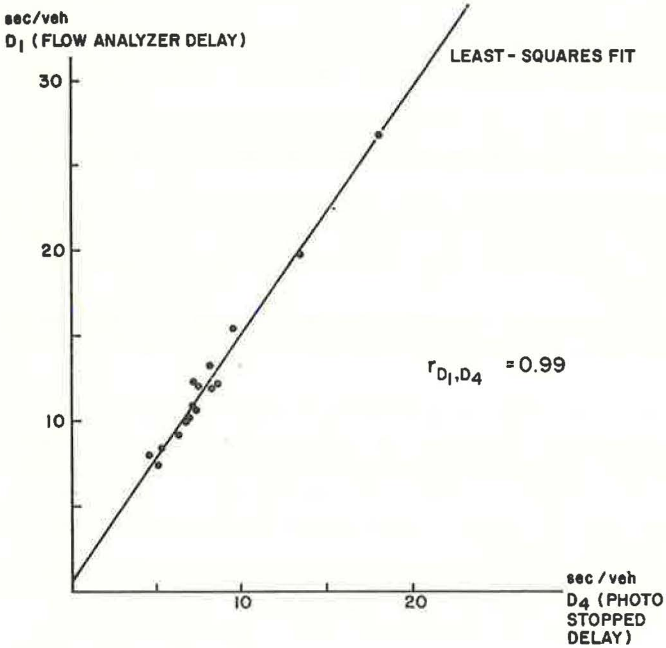

Table 5. Regression analysis for group 1 data.

Variable	Regression Line	Number of Data Points	Multiple Correlation Coefficient	Reduction of Sum of Squares (percent)
Lane 1				
D_1	$8.36 + 0.73X_3$	18	0.71	50
D_2	$7.04 + 0.46X_3$	18	0.74	55
D_3	10.1	12	—	—
D_5	$-2.52 + 0.72X_3 + 0.56X_2$	16	0.75	57
Lane 2				
D_1	21.9	18	—	—
D_2	16.0	18	—	—
D_3	19.4	12	—	—
D_5	19.8	16	—	—

Table 6. Significant differences between mean delays.

	Observation							
	First Hour		Second Hour		Third Hour		Fourth Hour	
Methods Compared	Value	Significant Difference Level	Value	Significant Difference Level	Value	Significant Difference Level	Value	Significant Difference Level
Photo stopped and photo aggregate	$X_1 < X_2$	90 percent	$X_1 < X_2$	None	$X_1 < X_2$	99 percent	—	—
Flow analyzer and photo aggregate	$X_1 > X_2$	95 percent	$X_1 > X_2$	None	$X_1 > X_2$	None	—	—
Sagi-Campbell and photo aggregate	$X_1 > X_2$	None	$X_1 < X_2$	None	$X_1 < X_2$	99 percent	—	—
Berry-Van Til and photo stopped	$X_1 > X_2$	90 percent	$X_1 > X_2$	None	$X_1 > X_2$	99 percent	$X_1 > X_2$	90 percent

Note: A one-sided test was used with the assumption that population variances are unknown.

Table 7. Linear regression analysis of mean delays.

	Observation								
	Lane 1			Lane 2			Lanes 1 and 2		
Methods Compared	Linear Equation	Correlation Coefficient	Sample Size	Linear Equation	Correlation Coefficient	Sample Size	Linear Equation	Correlation Coefficient	Sample Size
Photo stopped (y) and photo aggregate (x)	$y = 0.693x + 0.355$	$r^2 = 0.989$	16	$y = 0.663x + 0.874$	$r^2 = 0.967$	16	$y = 0.678x + 0.553$	$r^2 = 0.985$	32
Flow analyzer (y) and photo aggregate (x)	$y = 0.999x + 1.119$	$r^2 = 0.988$	16	$y = 1.107x + 0.419$	$r^2 = 0.981$	16	$y = 1.106x + 0.184$	$r^2 = 0.989$	32
Sagi-Campbell (y) and photo aggregate (x)	$y = 0.535x + 3.530$	$r^2 = 0.900$	16	$y = 0.296x + 9.005$	$r^2 = 0.476$	16	$y = 0.477x + 4.813$	$r^2 = 0.773$	32
Berry-Van Til (y) and photo stopped (x)	$y = 1.455x - 0.817$	$r^2 = 0.988$	17	$y = 1.606x - 2.997$	$r^2 = 0.992$	11	$y = 1.463x - 0.964$	$r^2 = 0.989$	28

Figure 8. Relation of stopped delay and aggregate delay.

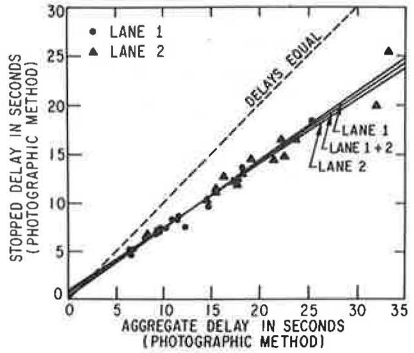

Figure 9. Relation of flow analyzer delay and and photo aggregate delay.

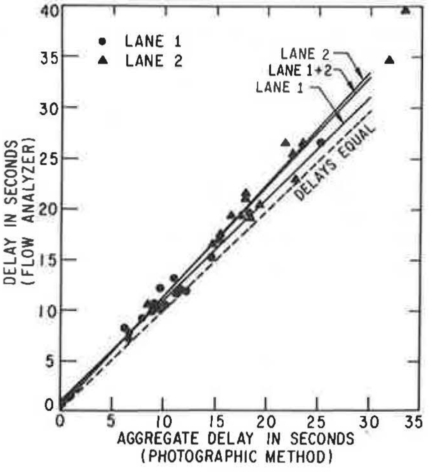

Figure 10. Relation of Sagi-Campbell delay and photo aggregate delay.

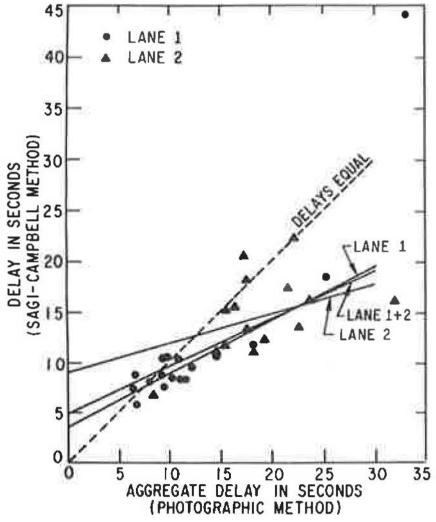

Figure 11. Relation of Berry-Van Til delay and photo stopped delay.

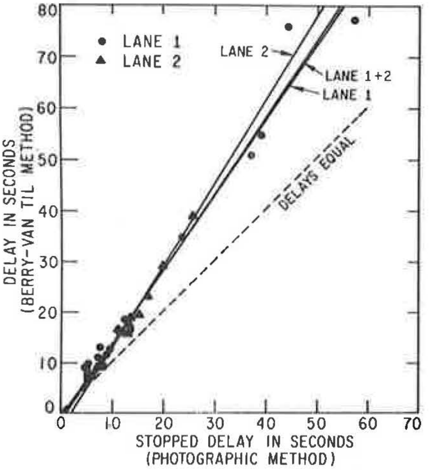

Figure 12. Intersection performance (75 percent stopping for signal).

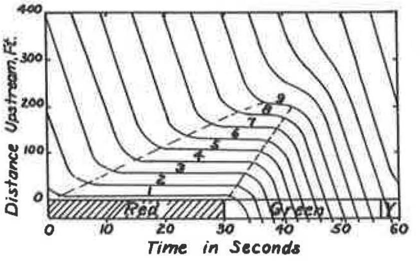

Figure 13. Intersection performance (100 percent stopping for signal).

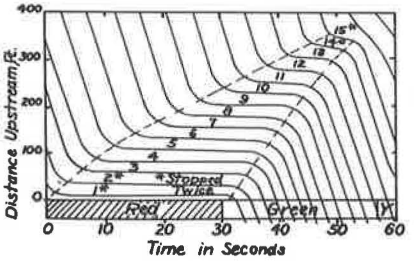

Figure 14. Intersection performance (platooned arrivals).

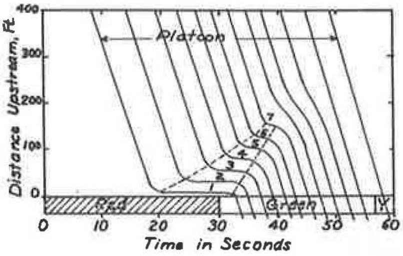

Table 8. Delay as scaled from Figures 12, 13, and 14.

Delay Method	Delay in Seconds per Cycle: 75 Percent Stopping	100 Percent Stopping	Platooned Arrivals
Travel-time delay (scaled)	210	315	41
Stopped-time delay (scaled)	157	261	35
Stopped delay (10-sec sampling)	170	260	40
Sagi-Campbell	135	315	105
Flow meter (scaled)	199	365	62

usually within 10 percent of values obtained by camera. Further study should be made to determine whether these differences are due to differences in procedures used in gathering the data for the two studies.

Some of the reasons for differences in results from the different methods can be identified by applying the methods to time-space diagrams of intersection performance (Figs. 12, 13, and 14). Stopped-time delays (between broken lines) were scaled for vehicles discharged during the cycles that are shown and are compared with other delay measures given in Table 8. Sampling of stopped-time delay was done at 10-sec intervals for all vehicles stopped during these cycles. Travel-time delay, used as the base method in earlier studies (6, 9), was also scaled from the diagrams.

Although results are not strictly comparable because the sampling method also includes delay for vehicles discharged in the following cycle, the study reveals the following:

1. The flow meter method includes in its delay values the travel times for vehicles after they leave the queue. For example, delay values scaled for vehicle 13 in Figure 13 are as follows: 16.5 sec via flow meter, 7.5 sec for stopped-time delay, and 10.0 sec for travel-time delay.
2. When a vehicle must stop for two red intervals (vehicles 1 and 2 in Fig. 13), the travel time between stops would normally not be included in stopped-time delay but is included in the other methods.
3. When vehicle arrivals are platooned as in Figure 14, the Sagi-Campbell method will overestimate delays (Table 8). Sagi and Campbell, in their original paper (5), suggest a method for correcting for such platooning.

If stopped-time delay is to be sampled in conjunction with studies of intersection capacity, effects of sampling rates should be investigated. A short sampling interval is needed for short cycle lengths and when distributions of vehicle arrivals are affected by upstream signals. Trials of sampling at 10-sec intervals are suggested, starting the sampling for each cycle at the beginning of the red.

If films for the Virginia intersection are still available, such sampling intervals can be tried along with investigating variations in instructions on how to sample stopped-time delay.

REFERENCE

9. Berry, D. S. Field Measurement of Delay at Signalized Intersections. HRB Proc., Vol. 35, 1956, pp. 505-522.

AUTHORS' CLOSURE

The authors would like to thank Hutter, McShane, May, and Berry for taking the time to comment on our paper. We agree in general with the discussions and have a few comments to offer.

In answer to May's four questions, we have the following remarks:

1. Neither the Berry-Van Til method nor the Sagi-Campbell method, at their present stages of development, provides any distinction or special treatment for trucks and buses and right- and left-turning vehicles. These data were included in our measurements to see if some of the variation in the computed delays could be due to high turning movements or high rates of commercial vehicles in the traffic stream. From simple observations, we could not detect any such relations that were consistent. McShane's statistical analysis indicates that there are such relations.
2. The 20-sec sampling interval in the Berry-Van Til method was selected so that repetitive sampling in the same parts of the signal cycle could be avoided. With a 75-sec cycle, the 20-sec interval appeared to be the most practical choice. (Berry and Van Til recommended an interval of 15 to 20 sec in their paper.) A shorter sampling

interval (10 sec), as Berry recommends in his discussion, might have produced more accurate results under normal traffic conditions. During congestion periods, however, where long queues develop, the accuracy of the method would become questionable as observers would be hard pressed to count and record the stopped vehicles in such short intervals.

3. The statistical analyses to test for significant differences among the mean delays obtained by the various methods were conducted for each individual lane for each set of hourly measurements.

4. We did not combine the lane data on an approach basis because we considered that that would not be in keeping with the committee's recommendation that, for any future revision of the HCM, intersection capacity and delay be expressed on a "by-lane" basis. Furthermore, combining the data might have a diluting effect on the influence of turns and commercial vehicles.

McShane grouped the data into two categories: group 1 consisting of the three off-peak periods and group 2 of the 1-hour peak period. His discussion is limited to the first group. He has found that the various delays are highly correlated, but each method has differing sensitivities to traffic parameters. We agree with him, and we so indicated in our report that "during the low traffic volume periods the delays computed by any of the five methods were generally uniform. . . ." It was during the high traffic-flow period that the data showed inconsistent relations among the delays computed by the various methods. We do not doubt that the methods tested "do measure the same sort of thing (that is, delay)." But they measure delay with varying sensitivities to volume, turns, and trucks and buses. Until these degrees of sensitivities are evaluated and factored, neither of the methods is ready for nationwide application. Even more important than this is the need, as Hutter points out, to describe what is meant by intersection delay so that we will all be talking in the same terms.

INVESTIGATION TO DETERMINE THE CAPACITY OF PROTECTED LEFT-TURN MOVEMENTS

Ronald R. Johnsen, City of Des Moines, Iowa; and
Judson S. Matthias, Arizona State University

This research evaluated the capacity of protected left-turn movements at signalized intersections where both separate turning lanes and separate signals were provided. Capacities were determined by field observations at selected urban intersections where signals had varying cycle lengths. Observed capacities were found to be significantly larger than capacities determined by the Highway Capacity Manual. The validity of observed results was checked by studying different intersections; no significant differences were observed. This research shows that protected left-turn capacities can be as much as one-third greater than those calculated by current procedures.

•OF all the problems of interest to the traffic engineer, the urban intersection at grade is one of the most important. If one considers that approximately one-half of all urban accidents and more than three-quarters of all urban delays are caused by, or are related to, urban intersections, the range and far-reaching consequences of the problem are more fully understood (1).

Efficiency and safety of movement through intersections is provided by regulating vehicles and pedestrians through the use of various types of traffic control devices. In many cases, the actual warrants used for the application of these control devices need much more refinement and development and are often subjects of controversy. The application of the protected left-turn type of signal control has been controversial, and a uniform acceptance has not yet been attained. Protected left-turn phases have been used either with or without a permissive left-turn phase following the protected movement.

Little factual study has been conducted in the past to establish uniform warrants for the installation of protected left turns. The existing warrants are not specific and are usually based on an expected increase in left-turn capacity or accident-reduction potential.

STATEMENT OF THE PROBLEM

The number of vehicles that can execute a left turn at a signalized intersection with a protected left-turn phase during a given period of time depends basically on two factors. The first is how soon the vehicles begin to move after the signal indication changes to green. The second is how fast each individual vehicle in the queue reacts to the vehicle immediately ahead. This process continues until all cars in the queue have entered or have progressed through the intersection or until the flow is stopped by a red signal indication. The amount of time required to dissipate a queue of vehicles after the signal changes to green depends on the reaction time and acceleration characteristics of each individual driver and vehicle. The total time for a group of vehicles to negotiate a left-turn movement at a signalized intersection can vary considerably.

Other factors affecting left-turn capacity at intersections with protected left-turn movements are physical and operating conditions such as approach width and parking conditions; load factor, peak-hour factor; metropolitan area population and location within the metropolitan area; traffic characteristics such as trucks, through buses, and local transit buses; and control measures such as type of traffic signals and marking of approach lanes (2).

PURPOSE OF THE STUDY

The primary purpose of this study was to determine the capacity of the left-turn movements at signalized intersections where both separate turning lanes and separate signal controls were provided for the left-turn movement and compare these capacities to Highway Capacity Manual (HCM) estimates.

A secondary purpose of this study was to investigate the effect of cross-traffic left-turning vehicles on the capacity of protected left-turn movements. Protected left-turn control is often installed at intersections where the cross-traffic left-turning vehicles are not controlled but are given a permissive movement. These permissive left-turning vehicles often fail to clear the intersection prior to their red indication, which means that they are blocking the intersection during the initial portion of the protected left-turn green phase.

VEHICLE HEADWAY AS A MEASURE OF CAPACITY

Average vehicle headways have been found to be a very practical unit in calculating the capacity of signalized intersections. Any given phase time divided by the average headway will give the actual number of vehicles that can pass the intersection during that period of time. To determine the capacity of a protected left-turn movement, we must obtain the average vehicle headways for the left-turning vehicles under loaded conditions. The phase length of the separate signal indication divided by the average headway for loaded conditions yields the capacity of the movement per cycle. The service volume per hour at capacity can readily be obtained by multiplying the capacity per cycle by the number of cycles per hour.

It has been found that the following factors affect average headways for through traffic: length of green phase, percentage of trucks, percentage of turning traffic, lane width, and grade. These same factors, with the exception of the percentage of turning vehicles, apply to average headways for left-turning vehicles (3).

LEFT-TURN STORAGE LANES

Intersections equipped with separate left-turn signal indications should normally also have separate turning lanes to store vehicles waiting for the green arrow. It has been found that both the length and the width of these lanes have an effect on the capacity of the movement.

Separate turn signal phases are often red, whereas the other through movements have a green indication; therefore, it is desirable to have the storage lane long enough to prevent blockage of a through lane. To ensure that all or nearly all vehicles are accommodated during each through green phase, the HCM suggests that, where possible, the storage lane be long enough to accommodate twice the average number of turning vehicles arriving per cycle (4). George and Heroy found that approximately 25 ft were required for each stopped vehicle at an intersection approach (5).

Leisch (6) felt that another consideration should be investigated in the determination of turning-lane lengths with separate signal phasing. He proposed that the turning lane should be long enough to allow entry of turning vehicles past a line of stopped, through vehicles.

A minimum length, based on 1.5 times the average number of through vehicles arriving per cycle, is needed to meet the requirements for through-traffic storage. This aspect often calls for a longer storage lane than that required to store the turning vehicles.

INTERSECTIONS STUDIED

The existing capacities of protected left-turn movements in Tempe, Arizona, were determined by studying three intersections. All intersections studied were at-grade, 90-deg crossings of signal-controlled arterial streets. Each intersection had four legs, protected left-turn lanes, and two-way traffic. Approaches studied were three lanes including the protected left-turn lane. The intersections studied were selected because of high volumes during peak hours. Curb parking is prohibited on all approaches.

DATA COLLECTION

The volumes of vehicles turning left were observed and recorded for the intersection approaches studied. Because the study was conducted with the cooperation of the Tempe traffic engineering department, the data collection was restricted to the afternoon peak so that a city employee could be present during data collection to adjust the traffic signal controllers. The controllers were adjusted to provide left-turn phase lengths both longer and shorter than the normal settings to permit data collection over a range of phase lengths. The adjustments made were in 1-sec increments so that adverse traffic conditions would not be generated.

Approximately 5 hours of data were collected at two of the intersections with 10 hours collected at the other—5 hours during a 70-sec cycle and 5 hours during an 80-sec cycle.

The left-turn movements at the intersections observed were not operating at capacity; therefore, it was deemed necessary to record the observed volumes on a per-cycle basis. Loaded cycles were recorded separately from partially loaded cycles. A loaded cycle was defined as the condition where the entire green phase was utilized by traffic with a backlog of at least one vehicle at all times. In other words, vehicles were continually present during the protected left-turn green phase, and at least one vehicle was restrained at the end of the phase by the amber or red signal indication.

The protected left turns studied were all located at intersections where the cross-traffic left-turn movements were permissive. It was observed that these permissive left-turning vehicles often remained in the intersections through their amber phase and into their red phase. The presence of vehicles in the intersection during their red phase (the green phase for the protected left-turn movement) interfered with the protected left-turn movements. In view of this situation, the recorded left-turn volumes were identified as having interference or no interference.

ANALYSIS OF FIELD OBSERVATIONS

The data obtained in the field for this study were reduced to a form that would permit the application of statistical tests. Statistical tests were used to determine the significance of the observations.

The observed protected left-turn volumes were recorded in one of four categories: loaded phases with interference from cross-traffic left-turning vehicles, loaded phases without interference, nonloaded phases with interference, and nonloaded phases without interference. Mean values that represent vehicles per protected left-turn phase were calculated for all loaded phases, loaded phases with interference, and loaded phases without interference. An adjusted mean was also calculated for the same three columns, which included the volumes from the partially loaded phases that had a greater number of vehicles per phase than the calculated means of the loaded phases. The adjusted means are the maximum average number of vehicles per protected left-turn phase that could pass the intersection, therefore representing the capacity condition. The standard deviations and the average vehicle headways were also calculated for the same data used to obtain the adjusted sample means.

Methods of Comparing Observed Capacity and HCM Capacity

To determine if a significant difference existed between the observed protected left-turn capacities and the capacities estimated by the HCM, we used a statistical test, the t-test. The hypothesis tested was that the adjusted mean number of vehicles per phase as calculated from field observations was equal to or less than the mean number of vehicles per phase as determined from the HCM.

Methods of Analyzing the Effect of Cross-Traffic Left-Turning Vehicles on Protected Left Turns

It was desired to test statistically the adjusted mean number of vehicles per loaded protected left-turn phase with interference from cross-traffic left-turning vehicles ($\bar{y}$) against the mean number of vehicles per loaded protected left-turn phase without interference ($\bar{x}$). It was considered important to test the hypothesis that $\bar{x}$ was less than or equal to $\bar{y}$, desiring to reject whenever $\bar{x}$ was larger than $\bar{y}$ (using $\alpha = 0.05$).

RESULTS OF THE STUDY

Field Observations Compared to HCM Estimates

The t-test was applied three times to each of the 31 approach conditions studied. For each approach condition, the test was used to determine if any significant difference existed between observed capacities and HCM estimates for testing all loaded phases, loaded phases with interference, and loaded phases without interference separately. (At one intersection data were not available for phase lengths of 14 and 16 sec without interference; at another, data were not available for the northbound approach during the 80-sec cycle with an 8.8-sec left-turn phase with interference.)

The hypothesis that the adjusted mean number of vehicles per phase from field observations was less than or equal to the mean number of vehicles per phase, as estimated from the HCM, was rejected 80 times out of 90 tests conducted. It was observed that of the 10 tests not rejected seven were for loaded phases with interference, two were for loaded phases without interference, and only one was unable to be rejected in the all-loaded category.

Effect of Cross-Traffic Left-Turning Vehicles on Protected Left-Turn Capacities

The t-test was used to determine if cross-traffic left-turning vehicles did significantly reduce the capacity of protected left-turn movements. The hypothesis that the adjusted mean number of vehicles per loaded phase without interference was equal to or less than the adjusted mean number of vehicles per loaded phase with interference was rejected 10 times out of 28.

DISCUSSION OF RESULTS

The analysis of data indicated that the capacity of the protected left-turn movements observed for all loaded phases was significantly greater than that estimated by the HCM for 30 of 31 approach conditions studied. The capacities determined from field observations for all loaded phases represented the existing capacity of the movements.

A plot of left-turn phase time versus left-turn vehicles per phase is shown in Figure 1, giving the difference between observed capacities and HCM estimates. Equations were produced that represent the best linear estimate of the data presented in the graphs. The equations

$$Y_o = 0.5066 + 0.3888X \tag{1}$$

and

$$Y_H = 0.0222 + 0.3388X \tag{2}$$

were produced for the observed capacities and HCM estimates respectively. The correlation coefficients of 0.9513 and 0.9999 for Eqs. 1 and 2 respectively indicated that the relations were a good linear fit.

The primary purpose of this study was to compare observed protected left-turn capacities with HCM estimates; however, it was felt that an explanation of the results should be proposed.

The HCM was developed from data collected throughout the United States with the stated purpose of creating a manual that could be used with confidence throughout the country. Variations in driver characteristics and composition among different regions would tend to indicate that one manual may not accurately represent all existing conditions.

The study location, in a city with a major university, probably contains a driver population somewhat younger than the national average.

The HCM does not specify when the data collections were conducted for the protected left-turn capacity analysis. It does indicate that data collections for the Manual began in 1954. The horsepower output of American-manufactured automobile engines has increased considerably since 1954 when less than 40 percent were V8's compared to 84 percent in 1969.

The results of the investigation into the effect of cross-traffic left-turning vehicles on protected left-turn movements were not as expected and warrant further study. Initially it was felt that a protected left-turn movement with interference would exhibit a capacity significantly less than the same movement without interference. This assumption was found to hold only 10 times in 30 tests.

The fact that the original assumption was not rejected ten times would tend to indicate that a more comprehensive analysis may be warranted to determine if the intensity of interference varies under different conditions. It was observed that, if a backlog of left-turning vehicles developed on the cross street, the drivers, after having waited through several cycles to reach the front of the queue, would often enter the intersection late in their amber phase or in the early seconds of their red phase, thus remaining in the intersection for several more seconds of the green arrow time than the usual permissive left-turn drivers.

Validity Check

To check the validity of the results, we conducted a limited study at three additional intersections in Tempe. The characteristics of these approaches are similar to those of the original intersections studied.

One hour of additional data was collected at each approach and then compared to the capacities expected as determined from this study. The expected capacities used were obtained from Figure 1. The t-test of hypothesis about a single parameter was used to determine if the capacities at the test approaches were statistically equivalent to the expected capacities from previous tests. The hypothesis of equality was not rejected at any test approach. The average percentage of variation from expected capacity was 8 percent.

Application of Results

The information obtained from this study has application to protected left-turn capacity analysis in Tempe, Arizona. Figure 1 could be used to predict the capacity service volume in vehicles per phase; however, because service volumes are usually expressed in vehicles per hour, an additional graph was constructed. Figure 2 provides a graphical solution for capacity service volumes in vehicles per hour for a variety of cycle lengths with protected left-turn phase lengths between 6 and 16 sec. Lines representing capacity service volumes (level of service E) estimated by the HCM were also placed on the graph.

Analytical solutions for the graph are provided through the following equations:

Cycle Length (sec)	Observed Capacity	HCM Capacity
50	$Y = 36.4752 + 27.9936X$	$Y = 1.5984 + 24.3936X$
60	$Y = 30.2761 + 23.3442X$	$Y = 1.3320 + 20.3280X$
70	$Y = 25.8366 + 19.8288X$	$Y = 1.1322 + 17.2788X$
80	$Y = 22.7970 + 17.4960X$	$Y = 0.9990 + 15.2460X$

In the preceding equations Y is the service volume at capacity in vehicles per hour, and X is the protected left-turn phase time in seconds.

Figure 1. Left-turn vehicles per phase versus left-turn phase time.

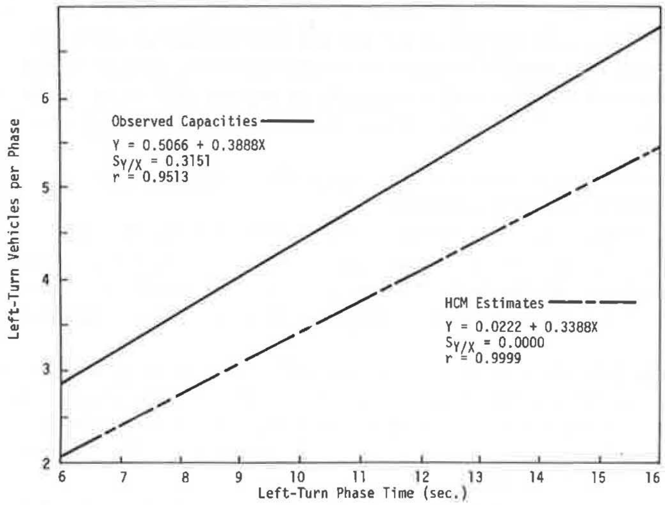

Figure 2. Left-turn volume versus left-turn phase time.

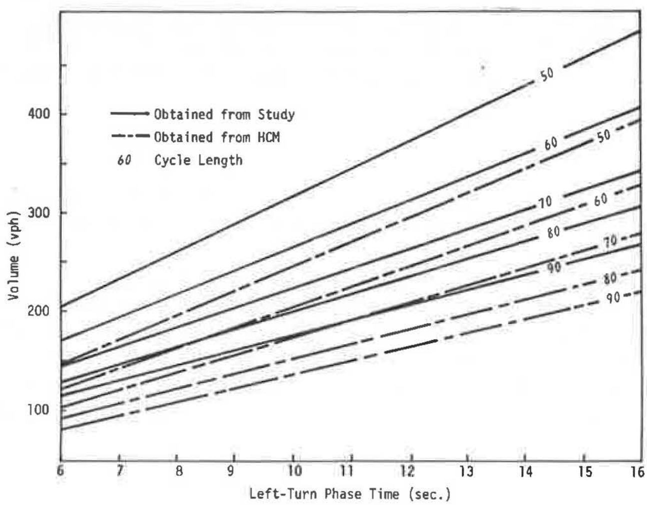

Table 1. Service volume per hour for 60-sec cycle.

Phase Length (sec)	Volume	
	HCM Estimate	Observed
6.3	129	177
7.0	143	194
7.2	147	198
7.7	157	210
8.0	163	217
8.4	171	226
8.8	179	236
9.0	184	240
9.1	186	243
9.6	196	254
10.0	204	264
10.4	212	273
10.5	214	275
11.0	224	287
12.0	245	310
13.0	265	334
14.0	286	357
15.0	306	380
16.0	326	404

Note: Average percentage of difference is 33 percent.

It should be pointed out that the scope of this study limited the investigation of these equations to left-turn phase lengths between 6 and 16 sec. The application of the equations for a 60-sec cycle length is given in Table 1. The observed capacities were on an average approximately 33 percent greater than HCM estimates.

CONCLUSIONS

The two main purposes of this investigation were to determine protected left-turn capacities at signalized intersections in Tempe, Arizona, and compare them with HCM estimates and investigate the effect that cross-traffic left-turning vehicles have on the capacity of protected left-turn movements.

By analyzing the volume of left-turn vehicles on four approaches at three different intersections, we made the following conclusions:

1. The protected left-turn capacities observed were significantly greater than those estimated by the HCM, and
2. The presence of permissive cross-traffic left-turning vehicles did not significantly reduce the capacity of protected left-turn movements in two-thirds of the observations.

AREAS OF FUTURE RESEARCH

In the course of collecting the data and preparing this report, several areas were encountered where further research could have been conducted.

One area of study would be an investigation into the effect of the time of day on intersection capacities. Other variables in capacity analysis that could be investigated include driver adherence to the laws related to protected left-turn movements, the presence of compact cars, and the composition of the driver population at various locations.

A study could be conducted to determine if regional variations in capacity exist and evaluate the feasibility of updating the HCM to adjust for regional variations if they do exist.

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HEADWAY APPROACH TO INTERSECTION CAPACITY

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The capacity of an approach to a signalized intersection is a function of approach headways of vehicles as the queue of waiting vehicles is discharged into the intersection and of lost time due to starting delay and to utilization of only part of the yellow light interval. This paper suggests a method for computing capacity of signalized intersections from measurements of headways, starting delays, and utilization of the yellow light and presents some preliminary data on its application.

•BARTLE, SKORO, AND GERLOUGH (1) measured starting delays and approach headways for loaded portions of cycles for many intersections in Los Angeles and proposed a formula for computing approach capacity of a fixed time signal. The formula for capacity of each cycle is $n = (g + a - d)/h$, where g is length of green in seconds, a is length of yellow, d is average starting time delay for first vehicle in peak hours, and h is average approach headway for loaded portions of cycles.

Webster (2) utilized the saturation flow of the approach (in vehicles per hour of effective green) as the basic measure of capacity. Saturation flow was measured by recording the number of vehicles that cross the stop line in saturated phases during successive 2-sec intervals after the start of the green and with information on commercial vehicles and turns (15). In computing saturation flow, data in the first few periods after the start of the green were excluded. Effective green time is chosen so that the product of saturation flow and effective green time equals the average number of vehicles passing during the combined green and yellow intervals. Effective green is green plus yellow minus lost time. Lost time averages about 2 sec per phase excluding all-red intervals.

Miller (3) also utilizes the concepts of saturation flow, lost time, and effective green but makes capacity computations by lane of approach. In the Australian Guide (14), saturation flow is defined as the reciprocal of the average time headway, in which the average headway for the loaded portion of a cycle is measured from the start of the green and 1 sec is subtracted because of starting time delay. Lost time is taken as the longer of intergreen time minus ½ sec or the travel time through the intersection plus 2½ sec. Effective green is defined such that the number of vehicles that cross the stop line in a fully saturated phase is equal to the product of the saturation flow and the proportion of effective green time.

In both the British and Australian methods, saturation flow values are given for ideal conditions. In defining capacity for prevailing conditions, through-car equivalents are utilized to correct for commercial vehicles and turns, and reduction factors are used to correct for effects of parking and grades.

METHOD

The method outlined in this paper differs from the Australian and British methods primarily in the procedures used for determining saturation flow and lost time. Effective green is determined from measurements of starting delay, utilization of the

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yellow, and length of the green as in the numerator of Eq. 1. This equation also takes into account the fact that the number of vehicles entering a loaded cycle is one more than the number of headways. Average values for headways, starting delays, and utilization of the yellow are used in Eq. 2 to compute the lane capacity or approach capacity in vehicles per hour.

$$n = \frac{g + \lambda y - D}{\bar{h}} + 1 \tag{1}$$

$$Cap = \frac{3,600\ (g + \bar{\lambda} y - \bar{D} + \bar{h})}{C\bar{h}} \tag{2}$$

where

n = number of vehicles discharged from one approach during one loaded cycle;
Cap = capacity of the signalized approach in vehicles per hour;
D = starting time delay in seconds elapsing from beginning of green to instant the rear wheels of the first vehicle cross the reference line (usually the stop line);
$\bar{h}$ = average headway time, in seconds, for all vehicles in a compact platoon that cross the reference line (in Eq. 2, $\bar{h}$ is the average for a large sample of cycles);
λ = proportion of length of yellow light, for a loaded cycle, which is utilized up to the time the last vehicle in a compact platoon crosses the reference line;
C = length of signal cycle in seconds;
g = length of green in seconds; and
y = length of yellow in seconds.

EXPERIMENTAL STUDY

Data were taken at one 18-ft approach to a three-phased signalized intersection to evaluate the proposed method and to study its application in evaluating effects of weather and visibility on capacity (4). The south approach of the intersection of Ridge and Church Streets, Evanston (Fig. 1), was selected because there were no left-turning traffic, no opposing flow, no vehicles parked, standing, or stopping, practically no commercial vehicles, and practically no pedestrians to interfere with right-turning vehicles. Those few cycles with buses and pedestrian interference were excluded from the study. All data were taken on weekdays during the evening peak period when about 95 percent of the cycles were loaded cycles. Cycle length was 60 sec with 17 sec green and 3 sec yellow.

Starting delays were measured with a stopwatch that makes one revolution in 10 sec. The stop line, which is 24 ft from the intersection as determined by a prolongation of the curblines, was used as the reference line. A second stopwatch was started as the rear wheels of the first vehicle crossed the stop line and was stopped when the last vehicle in the compact platoon crossed the stop line with its rear wheels. The elapsed time, T, shown on this second stopwatch, was then divided by the number of vehicles, less one, to determine the average headway, h, for the compact platoon. For a loaded cycle, the utilization of the yellow was $\lambda y = D + T - g$.

Results for 14 peak periods of data collection are given in Table 1, with data segregated according to weather and visibility conditions. Average queue discharge factors are shown for 60 loaded cycles on each of the days along with capacity values for each day as computed from Eq. 2 and as observed for 60 loaded cycles.

Statistical tests were performed to examine consistency among results for days having the same conditions. Tests were conducted at the 1 percent significance level under the null hypothesis that the mean headways come from the same population. For the 4 "dry-night" days, the mean values for each of the 4 days were not significantly different from the 4-day average. For the 5 "dry-daylight" days, 1 day had a mean headway significantly different from the 5-day average. Also, mean headway values for the two "wet-night" studies were significantly different from each other, perhaps because of the differences in intensity of rainfall. Such differences are being given further study by collecting additional data for other intersections.

Statistical tests were also performed using the null hypothesis that the mean headways for each set of adverse weather and visibility conditions were the same as for dry-daylight conditions. Comparisons were made between dry-night and dry-daylight conditions and between wet-night and dry-night conditions. The null hypothesis had to be rejected for all significance levels above 1 percent in both cases, indicating that adverse weather significantly increased headways.

Table 1 also shows that capacity values computed from Eq. 2 check closely with observed counts of 60 loaded cycles, as would be expected. Computed values for adverse weather and visibility are substantially lower than those for dry-daylight conditions. For dry pavement, capacity at night was 9.4 percent lower than that for daytime; wet-night capacities were 16.2 percent lower than those for dry-daytime conditions. Observed values for dry-daylight conditions are as much as 32 percent higher than those computed by Highway Capacity Manual (12) methods.

Othman and Rapino (5) studied utilization of the yellow for loaded cycles at the same intersection, for both the south and north approaches, and by lane for the south approach. Results given in Table 2 show values for λy for the south approach that are not significantly different from those for the north approach where the reference line was 3 ft closer to the intersection. Average values by lane for the south approach were similar (1.53 and 1.56 sec) and were somewhat higher than the 1.43 sec average per lane reported by Leong for Australian conditions (6).

The effects of position of the reference line on starting delay and utilization of the yellow were also examined by Othman and Rapino (5), for the south approach at Ridge and Church Streets, Evanston. A reference line 12 ft from the intersection would correspond to a position where the front of the average passenger car would be entering the intersection as its rear wheels cross the reference line. If the stop line remained 24 ft from the intersection (Fig. 1), use of a reference line 12 ft from the intersection instead of 24 ft would increase measured starting delay and increase utilization of the yellow. These effects tend to cancel each other when Eq. 2 is used to compute capacity. Further study is needed at other intersections to determine the location of the reference line best suited to varying positions of stop lines in relation to the intersecting curblines.

APPLICATION

Equation 2 can be applied to compute capacity for prevailing conditions at existing signalized intersections, as was done in the experimental study, without need for corrections for commercial vehicles, turns, parking, and so forth. Observers would measure headways, starting delays, and utilization of the yellow for loaded cycles and then utilize Eq. 2 to compute the capacity. In the event that there are no loaded cycles, typical values of λ can be used based on driver practice in the area. Corrections for turns, commercial vehicles, and so forth are not needed if the traffic conditions during field studies are typical of the average peak-hour conditions.

Utilization of this method for intersections not yet signalized, or where changes are expected in physical layout, controls, or traffic characteristics, will require development and use of standard values and adjustments as was done for the Australian method. This will involve compiling average values for starting delays and utilization of the yellow and lane headways (in through-car units). Also needed will be through-car equivalents for right turns and commercial vehicles, left-turn reduction factors as affected by opposing flow, and correction factors for parking, buses, grades, and lane width.

Some data on saturation flows and on reduction factors have already been collected in the United States. Passenger-car headways by lane and passenger-car equivalents for trucks and turning vehicles have been evaluated by Carstens for streets in Ames, Iowa (13). Effects of opposing flow on passenger-car equivalents for left turns have been studied in Australia (3) and at Northwestern University (7, 8).

Table 1. Capacity computed from queue discharge factors.

Weather Visibility and Date	Number of Loaded Cycles	Loaded Cycle Performance: Starting Delay, $\overline{D}$	Headways: $\overline{h}$	Headways: σ	Utilization of Yellow, $\overline{\lambda}y$	Computed Capacity (vph)	Observed Count for 60 Cycles
Dry-day							
3/22/71	60	2.379	1.107	0.047	0.967	905	904
3/23/71	60	2.607	1.086	0.049	1.574	941	942
3/25/71	60	2.490	1.074	0.039	1.746	968	968
3/29/71	60	2.485	1.071	0.047	1.293	944	946
4/15/71	60	2.457	1.089	0.054	1.460	941	944
Average		2.483	1.085	0.049	1.408	940	941
Dry-night							
11/17/70	60	2.434	1.167	0.098	0.300	823	835
11/18/70	60	2.483	1.178	0.083	1.301	865	869
11/21/70	60	2.555	1.176	0.040	1.816	890	896
11/22/70	60	2.458	1.178	0.084	1.089	855	858
Average		2.482	1.175	0.099	1.126	858	864
Wet-night							
11/16/70	60	2.670	1.256	0.070	1.408	810	813
2/4/71	60	2.762	1.318	0.135	1.193	762	772
Average		2.716	1.287	0.112	1.300	786	792
Snow-day							
3/18/71	60	2.714	1.282	0.059	1.696	808	799
3/19/71	60	2.683	1.255	0.063	2.088	844	846
Average		2.698	1.269	0.062	1.892	826	822
Snow-night							
2/12/71	60	2.638	1.283	0.064	2.073	829	821

Figure 1. Site of experimental study.

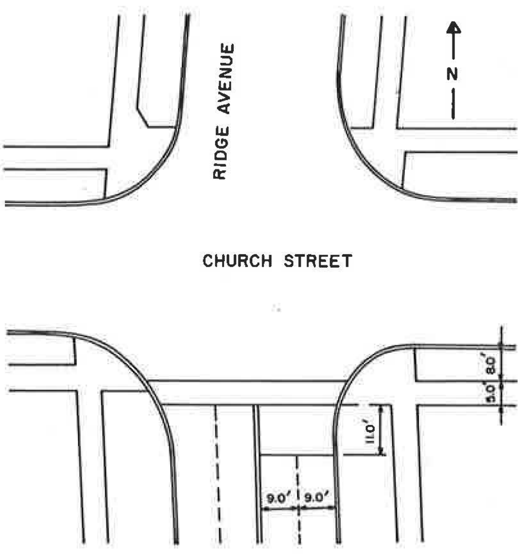

Table 2. Utilization of yellow light for loaded cycles.

Approach	Evening Peak: λy	Evening Peak: $\sigma_{\lambda y}$	Evening Peak: λ	Morning Peak: λy	Morning Peak: $\sigma_{\lambda y}$	Morning Peak: λ
South						
Both lanes	1.89	0.78	0.63	1.92	0.71	0.64
Curb lane	1.56	0.82	0.52			
Median lane	1.53	0.65	0.51			
North						
Both lanes	2.15	0.65	0.72	2.18	0.77	0.73

Note: Sample size varied from 38 to 99 loaded cycles (5).

Other studies by graduate students at Northwestern University have examined headways for double left turns on separate phasings (9) and on discharge rates by lane (10). Additional graduate students are extending these studies to additional intersections. We also are examining possible effects of reaction time in measuring starting time delay, as has been considered in Australia (3), and effects of grade of approach.

RECOMMENDATIONS

Additional data on discharge performance of typical signalized intersection approaches of different widths should be collected by lane for different types of streets and areas to determine average starting delays, through-car headways, and utilization of the yellow for loaded cycles. This type of data collection might be incorporated in the information-gathering procedure for updating the Highway Capacity Manual (12), which is in the planning stage. In addition, extensive sampling should be carried out at a few approaches to evaluate through-car equivalents for left turns as affected by opposing flow (8), for commercial vehicles, and for effects of pedestrians, parking, bus stops, and grades.

These procedures for capacity computations should be extended to determining service volumes for levels of service other than capacity, utilizing level-of-service criteria such as delay, queue length, percentage of loaded cycles, and percentage of vehicles stopping.

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