Starting Delay and Time Spacing of Vehicles Entering Signalized Intersection

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● TWO parameters of traffic performance can be used to represent several important characteristics of intersection operation. The two parameters, investigated in this study, are starting delay and time spacing of vehicles entering a signalized intersection. Information on the variability of these parameters at an intersection and among intersections may have useful applications in studies of intersection capacity and signal timing.

Starting delay, designated D, is defined as the time in seconds required for the first vehicle to enter the intersection after the display of the green signal. This corresponds to the "entrance time for the first-in-line vehicle" used by Greenshields, Schapiro, and Ericksen (1).

Time spacing, designated S, is the average time headway in seconds between successive vehicles in an entering platoon. In the measurement of S the number of lanes in which traffic is moving is disregarded, and the entire intersection approach is considered a unit. Time spacing as used here agrees with the definition presented in the "Traffic Engineering Handbook" (2) in its discussion of traffic signal timing formulas. S applies only to platoon movement and is computed by dividing total time in a signal cycle used for platoon movement by one less than the number of cars in the platoon.

INTERSECTION CAPACITY

Maximum capacities for signalized intersections can be expressed in terms of these parameters S, time spacing, and D, starting delay. For each cycle the time used for actual traffic movement is nS where n is the number of vehicles entering and S is time spacing as defined above. This assumes that a time S follows the entrance of the last vehicle in each cycle. Since an allowance must be made for a platoon of vehicles to start, a time D must be added to each cycle. The total time t required for n cars to enter an intersection is then $t = D + nS$.

This equation may be written $n = \frac{t - D}{S}$, and if the green and amber time is considered available for movement, this becomes $n = \frac{g + a - D}{S}$ for the number which can enter in one cycle with green time g seconds and amber time a seconds. Use of green plus amber appears appropriate here since the formula allows S seconds to follow the last car before the green begins on the opposing phase.

To find the total volume N which can enter during an hour, n is multiplied by the number of cycles per hour, m.

$$N = \frac{m\,(g + a - D)}{S} \qquad \text{(I)}$$

Equation (I) expresses maximum hourly capacity under fixed time control. The general expression for hourly capacity, which is applicable to any kind of signal control, uses G and A as total hourly green and amber time, thus:

$$N = \frac{G + A - mD}{S} \qquad \text{(II)}$$

In the "Highway Capacity Manual" (3) basic capacity is established as 1500 vehicles per 12-ft. lane per hour of green or 1250 per 10-ft. lane per hour of green. Basic capacity assumes a continuous stream of cars entering the intersection at about 2.4-sec. intervals in 12-ft. lanes and 2.9 sec. intervals in 10-ft. lanes for the entire period considered and in this way uses a factor closely related to the S used here. Basic capacity does not allow for traffic delays. Neither does it allow for the effect of parked cars, turning movements, trucks and buses, or other traffic conditions, nor for normal fluctuations in traffic volumes.

The formulas presented previously are based on values of S and D applicable to an individual intersection approach and, when applied to that approach, take into account the effect of all traffic conditions peculiar to that intersection. No allowance is made for fluctuations in volume. In the formulas presented here it is assumed that all available time in every cycle will be used for movement.

Greenshields et al (1) have approached the capacity problem in terms of the capacity of a traffic lane. They report that the first car enters the intersection 3.8 sec. after the beginning of the green (a value corresponding to D as used here) and that successive cars enter at 3.1, 2.7, 2.4, and 2.2-sec. intervals until the sixth-in-line and all following cars enter at 2.1-sec. headways. Use of these headway figures permits computation of the number of cars which can enter in one lane during a given green signal period and, from this, hourly capacity per lane.

Greenshields' method for computing capacity is similar in many ways to that suggested in this paper. It differs in that it is based on single-lane capacity rather than that of the entire intersection approach as used in this paper. In addition, Greenshields' method utilizes, in effect, single values of S and D and suggests no method for adjusting headways derived from them to conditions at other intersections. One of the purposes of this paper is to determine whether S and D are constants for all intersections.

TRAFFIC SIGNAL TIMING

Several formulas have been proposed for optimum timing of traffic signals. The formula used by Earl J. Reeder in reference (2) above utilizes S as defined here, V, the average velocity at the intersection, and q*, the number of vehicles arriving at the intersection in fifteen minutes, for computation of cycle length.

The National Safety Council (4) and the Institute of Transportation and Traffic Engineering (5) have published another formula, similar in some respects. The NSC-ITTE formula is:

$$T = \frac{y_1 + y_2 + D_1 + D_2 - S_1 - S_2}{1 - .0011\,(q_1S_1 + q_2S_2)}^{*} \qquad \text{(III)}$$

where quantities are defined as follows:

T = cycle length in seconds
y = amber clearance period (sec.)
D = starting delay (sec.) as previously defined
S = time spacing (sec.) as previously defined
q = number of vehicles entering in 15 minutes*

The subscripts apply to values for the opposing signal phases, and data are used for the heavier approach on each phase.

The signal timing formula is extremely sensitive to small changes in the value of S and somewhat less sensitive to changes in D. This sensitivity of T to changes in S and D indicates that formula (III) will be of relatively little use if either quantity is extremely variable from cycle to cycle or if their values do not remain reasonably constant from day to day. The cycle length T computed from formula (III) is usually somewhat shorter than most traffic engineers prefer to use in practice.

The above discussion indicate that these parameters, S and D, may have useful applications to traffic problems. The usefulness of these parameters, however, appears to be dependent on the variability of S and D values—both at any one intersection and among different intersections and intersection approaches.

INVESTIGATION OF S AND D

This experiment was intended to measure some typical values of S and D, to examine the variability of these parameters both from intersection to intersection and from day to day at the same intersection, and, if possible, to relate these values and their variabilities to physical and traffic conditions.

Starting delays and time spacings were observed and the results analyzed at thirteen

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* n is used instead of q in the references cited. The notation has been changed in order to be consistent in this paper.

heavily travelled intersection approaches in the Los Angeles area. The following conditions prevailed at these intersections:

1. All intersections were signalized. Eight approaches were controlled by fixed-time and five by full traffic-actuated signals.
2. Data were collected only on heavily loaded approaches. Each platoon of vehicles entering the intersection from the studied approach during the period of observation started from a stop and usually contained at least ten vehicles.
3. There were no streetcars and very few buses on the approaches studied.
4. The intersection approaches studied carried at least two lanes of travel in each direction. The range of street widths studied was 50 ft. to 76 ft.

Observations were made between 4:15 and 5:45 p.m. on warm, dry days in the spring and summer of 1952, and, where possible, observations at a given intersection were made on five consecutive week days. Data were taken by five different individuals during the course of the study. The same observer took all data for a given intersection approach.

During each study period the observer recorded data for thirty-one signal cycles. Starting delay, D, was recorded as the time from the first display of green to the entrance of the first vehicle into the intersection. A vehicle was considered to have entered when its rear wheels crossed the pedestrian crosswalk line nearer the center of the intersection. The first vehicle could enter from any lane. Negative values for starting delay are possible, but at the intersections studied cross traffic was always heavy enough during periods when data were being taken to prevent cars from entering before receiving the green signal.

The time for platoon movement was recorded as the time from the entrance of the first vehicle into the intersection until the entrance of the last car of the platoon. The observer also recorded the number of vehicles entering during this time. Average time spacing, S, for a given cycle was determined by dividing the time for platoon movement by one less than the number of vehicles entering during that time.

If a cycle occurred in which at the beginning of the green the front rank of the approach was not fully occupied by stopped vehicles, data were not taken for that cycle. Under prevailing traffic conditions, however, a large reservoir of waiting vehicles were usually assured during the study periods. All data were collected during peak traffic periods.

Determining the end of a platoon was a judgment on the part of the observer. Observers were instructed to consider a platoon ended whenever any one lane was empty or whenever traffic entered the intersection without being restricted in any way by cars immediately ahead. Observers were urged, if necessary, to cut off platoons early in order to be certain that all cars counted were actually traveling in platoons. Data were not recorded for individual lanes; data were based on all cars entering from all lanes in one direction.

Possible differences in results among observers were studied. Three observers independently recorded data for time spacing for the same intersection approach. An analysis of variance [1] was made among the three individuals' data and differences among observers were found not significant at this particular intersection.

The observed data described above permitted computation of mean values and standard deviations for D, the starting delay, and S, the average time spacing between vehicles. A tabulation of physical and traffic characteristics for the thirteen intersection approaches is presented in Table 1 and a summary of the values for starting delay and time spacing is presented in Table 2.

Values of both parameters, S and D, were assumed to be normally distributed. The chi-square test of goodness of fit was applied to three intersections for each parameter, and for each parameter the hypothesis of normality was rejected of the .05 level for one of three arbitrarily selected intersection approaches. Principal departures from normal distribution are noted in a greater number of relatively large S and D values under actual conditions than would be expected in a normal distribution.

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[1] For this and other statistical techniques, Dixon and Massey (6) was used as the principal reference.

TABLE 1
PHYSICAL AND TRAFFIC CONDITIONS AT THIRTEEN INTERSECTION APPROACHES STUDIED

Intersection	Direction	Widths Study Street	Widths Cross Street	Estimated ADT[a] Study Street	Estimated ADT[a] Cross Street	Parking	Prohibited Turns	District
Beverly at Fairfax	West	57	60	14.5[b]	21.5[c]	None	Left	Business
LaCienega at Pico	South	71	69	6.5[c]	26.0[b]	Yes	None	Business
LaCienega at Third	North	70	56		18.5[b]	Yes	None	Lt Bus
Melrose at LaBrea	West	57	70	13.5[b]		Yes	None	Lt Bus.
Santa Monica at Beverly Dr.	West	60	60	17.5[b]	12.0[b]	None	None	Residential
Sepulveda at Olympic	North	60	86	14.5[b]	27.0[b]	None	None	Intermediate
Sepulveda at Slauson	South	62		17.5[b]	11.0[b]	None	None	Rural
Sepulveda at Sunset	North	56	50	13.0[b]	17.0[b]	None	None	Open
Sepulveda at Sunset	South	56	50	15.5[b]	17.0[b]	None	None	Open
Sunset at Sepulveda	East	50	56	9.5[b]	28.5[b]	None	None	Open
Sunset at Sepulveda	West	50	56	7.5[b]	28.5[b]	None	None	Open
Westwood at Pico	South	50	60	8.0[b]	24.5[b]	Yes	None	Lt. Bus
Wilshire at Sepulveda	West	76	52	18.0[b]	27.0[b]	Yes	None	Vets. Home

[a] Average Daily Traffic is given in thousands for one entering approach for study street and for total cross street traffic in both directions. Sources are as indicated. Six-hour counts have been multiplied by 2.5. Peak counts have been multiplied by 10.
[b] Division of Highways
[c] City of Los Angeles

STARTING DELAY D

The mean starting delays at the thirteen intersection approaches studied ranged from 2.91 sec. on Sepulveda at Slauson to 4.40 sec. on La Cienega at Pico. The effect of location on mean starting delay was tested for significance by analysis of variance. The hypothesis tested was that all thirteen mean starting delays were equal. The analysis is presented in Table 3. The hypothesis of equal means can be rejected at the .005 level of significance, and the effect of location is thus found to be significant.

Statistical tests were applied to the effect of day of data-taking on the mean value of D for each intersection approach. Where variances of D on the different days were homogeneous, the analysis of variance was used; where variances were not homogenous, a modified t-test was used. The mean value of D for any one of the five days was significantly different from the five-day mean on only one of the thirteen intersection approaches studied (Sepulveda N at Olympic). At the twelve other intersections the effect of day of data-taking on D was not significant at the 5 percent significance level.

Reference to Table 2 shows that the highest five-day-average standard deviation, 1.52 sec., was observed on Sepulveda southbound at Sunset while the lowest five-day-average standard deviation, 1.04 sec., was on Sepulveda northbound at Sunset. The extreme values of variability were found on opposite approaches to the same intersection. Except for the four approaches to the Sunset-Sepulveda intersection, low mean starting delay is associated with low mean standard deviation.

The lowest standard deviation is 26 percent of the mean and the highest 39 percent. Seven of the 13 intersections have std. dev./mean values between 30 and 33 percent.

The mean value for starting delay, D, for the thirteen approaches studied is 3.83 sec. with an average standard deviation of 1.27 sec. within each day. The average standard deviation among five means for different days is 0.27 sec.

Comparison of mean delay values with various intersection characteristics does not reveal any one factor which appears to have a consistently important effect in increasing or decreasing the value of D. Volume of vehicle and pedestrian cross traffic, gradient of the approach, and the extent of right turning movements may be important factors in determining starting delay. Width of the pedestrian crosswalk may have some effect for very wide and very narrow widths. There are insufficient data in this study to permit careful evaluation of the effects of all variables.

Examination of the data indicates that starting delay is independent of street width and the type of signal installation at the intersection. The visibility of signal faces may be a factor, but this could not be evaluated from these data.

TIME SPACING S

Average time spacing for the approaches studied ranges from 0.95 to 1.63 sec., and mean values are significantly different among different approaches studied. The hypothesis

TABLE 2
OBSERVED STARTING DELAY AND TIME SPACING VALUES

Intersection	Dir.	Day	Starting Delay Mean	Starting Delay Std. Div	Time Spacing Mean	Time Spacing Std. Div	5-Day Averages D Mean	5-Day Averages D Std Div.	5-Day Averages S Mean	5-Day Averages S Std. Div.	Grand Std. Dev. D	Grand Std. Dev. S
Streets with Parking												
LaCienega at Pico	(71')S	M	4.14	1.30	1.27	0.26	4.40	1.38	1.15	0.23	1.377	0.237
		T	4.56	1.21	1.12	0.25						
		W	4.38	1.42	1.15	0.23						
		Th	4.49	1.48	1.09	0.21						
		F	4.45	1.50	1.10	0.20						
LaCienega at Third	(70')N	M	3.41	1.29	1.13	0.21	3.87	1.28	1.14	0.22	1.298	0.217
		M	3.13	1.39	1.16	0.23						
		T	3.97	1.26	1.10	0.17						
		T	3.60	1.27	1.13	0.23						
		W	3.49	1.21	1.16	0.26						
Melrose at LaBrea	(57')W	M	3.71	1.17	1.53	0.25	3.96	1.40	1.53	0.26	1.439	0.252
		T	4.32	1.03	1.57	0.27						
		W	3.91	1.67	1.53	0.26						
		Th	4.27	1.67	1.52	0.26						
		F	3.60	1.45	1.49	0.24						
Westwood at Pico	(50')S	T	3.51	0.94	1.58	0.27	3.69	1.13	1.63	0.32	1.255	0.306
		W	3.65	1.14	1.62	0.34						
		Th	3.70	1.31	1.61	0.32						
		F	3.71	1.01	1.66	0.29						
		F	3.89	1.24	1.68	0.31						
Wilshire at Sepulveda	(76')W	M	3.74	0.94	0.94	0.17	3.77	1.19	0.95	0.14	1.266	0.166
		T	4.52	1.34	0.99	0.11						
		W	3.19	0.96	0.95	0.18						
		Th	3.56	1.27	0.98	0.10						
		F	3.85	1.43	0.91	0.16						
Streets with No Parking												
Beverly at Fairfax	(57')W	M	3.56	1.26	1.23	0.19	3.79	1.23	1.26	0.24	1.247	0.236
		T	3.69	1.29	1.28	0.23						
		W	4.21	1.55	1.30	0.35						
		Th	3.43	1.12	1.26	0.17						
		F	4.08	0.91	1.25	0.18						
Santa Monica at Beverly Dr	(60')W	T	2.94	0.94	1.21	0.17	3.22	1.16	1.29	0.20	1.242	0.210
		W	3.39	1.30	1.34	0.21						
		Th	3.13	1.13	1.27	0.20						
		F	3.24	1.11	1.34	0.24						
		F	3.38	1.31	1.30	0.20						
Sepulveda at Olympic	(60')N	M	4.08	1.56	1.30	0.28	4.35	1.34	1.29	0.25	1.485	0.257
		M	4.42	1.30	1.36	0.24						
		T	4.90	1.59	1.22	0.24						
		T	3.73	0.95	1.20	0.27						
		Th	4.42	1.30	1.38	0.23						
Sepulveda at Slauson	(62')S	T	2.74	0.92	1.30	0.23	2.91	1.09	1.27	0.23	1.093	0.229
		T	2.99	0.90	1.26	0.24						
		W	2.86	1.29	1.24	0.19						
		Th	2.94	1.33	1.30	0.25						
		F	3.01	1.00	1.23	0.25						
Sepulveda at Sunset	(56')N	M	4.26	1.27	1.28	0.15	4.01	1.04	1.31	0.17	1.050	0.171
		T	3.70	1.05	1.34	0.25						
		W	4.02	0.94	1.30	0.16						
		Th	4.09	0.90	1.29	0.14						
		F	3.97	1.05	1.34	0.13						
Sepulveda at Sunset	(56')S	M	3.97	1.74	1.41	0.25	3.92	1.52	1.36	0.24	1.532	0.249
		T	4.29	1.69	1.31	0.21						
		W	4.04	1.59	1.27	0.22						
		Th	3.72	1.26	1.40	0.27						
		F	3.60	1.33	1.40	0.28						
Sunset at Sepulveda	(50')E	M	4.04	1.32	1.71	0.32	4.20	1.27	1.63	0.24	1.255	0.249
		T	4.14	1.10	1.53	0.18						
		W	4.29	1.42	1.66	0.22						
		Th	4.56	1.14	1.66	0.23						
		F	3.99	1.35	1.57	0.25						
Sunset at Sepulveda	(50')W	M	3.45	0.87	1.36	0.29	3.63	1.42	1.43	0.25	1.340	0.263
		T	3.75	1.35	1.34	0.27						
		W	3.74	1.74	1.53	0.28						
		Th	3.29	1.19	1.38	0.18						
		F	3.93	1.95	1.52	0.24						

TABLE 3
SUMMARY OF ANALYSIS OF VARIANCE TEST OF EQUALITY OF MEAN D'S FOR ALL INTERSECTIONS

$H_0: \mu_1 = \mu_2 \ldots = \mu_{13} \quad \alpha = .005$

	Sum of Squares	d. o. f.[a]	Mean Square	F-Ratio
Among means	389.82	12	32.48	$F = \frac{32.48}{1.68} = 18.3$
Within groups	3353.78	1998	1.68	

For $\alpha = .005$, $F_{.995}(12, 1998) = 2.36$. Reject H_0 if F greater than 2.36. Therefore, H_0 may be rejected at $\alpha = .005$. Mean starting delay is significantly different for different intersection approaches.

[a] A total of 2011 individual valid observations were made. Four cycles of data were omitted.

of independence of S values taken on different days was tested, and there was significant difference among days at three of the 13 intersection approaches (La Cienega-Pico, Sepulveda-Olympic, Sunset West at Sepulveda). As in the case of D values, analysis of variance was used where variances were homogeneous and a modified t-test where they were not.

The standard deviations of S for the 13 approaches fall in the range between 0.16 and 0.31 sec. The mean standard deviation is 0.234 sec., and eight of the 13 fall between 0.225 and 0.265 sec.

The standard deviations of five daily means ranged from .03 sec. for Sepulveda at Olympic to .09 sec. on Sunset W. at Sepulveda. The hypothesis of equal mean values for the five days was accepted for all approaches with standard deviations of S less than .07 among days.

The data show that S is a function of intersection characteristics and that significant differences in time spacing values exist among different intersection approaches. Examination of the data indicates that the two factors having the greatest effect on time spacing S for the intersection approaches studied are (1) street width and (2) parking conditions. The "Highway Capacity Manual" (3) reports studies which suggest that intersection capacity is increased almost linearly by additional feet of width and that capacity is a function of street width rather than simply of number of lanes. The studies conducted here appear to support these findings.

For the purpose of analysis the intersections studied were divided into two groups depending on whether or not parking was present on the approaches. Parking is permitted on La Cienega, Melrose, Westwood, and Wilshire. Parking is also permitted on Sepulveda at Slauson, but there was no parking here during any periods of data collection. Parking is prohibited at least during peak hours on the other streets studied.

Average time spacing S is plotted against street width in Figure 1. The data in squares are for streets with no parking, and Line A is the linear approximation for these data. The data in circles are for streets with parking permitted, and Line B is the linear regression line for these data. Both relationships were approximately linear within the range of street widths studied.

For the entire range of possible street widths, however, the relationships can be expected to be curves approaching the axes asymptotically, and Lines A and B are approximations of portions of these curves.

The regression lines for the assumed linear relationship of starting delay to street width are as follows:

Parking: $S = 2.98 - 0.026\,w$
No Parking: $S = 2.63 - 0.023\,w$

where w is the curb-to-curb street width.

Left turns were permitted at all intersections having no parking except on Beverly at

Fairfax. Beverly Boulevard is 57 feet wide here, and the S value falls below the assumed linear relationship of Line A.

Two instances of very good agreement between S values for intersection approaches of the same width are noted in the data. The two approaches studied on La Cienega have mean values for S of 1.14 and 1.15 sec. for widths of 71 and 70 ft., respectively. For streets without parking, Sepulveda at Olympic and Santa Monica at Beverly Drive are both 60 ft. wide, and the mean S value was 1.29 sec. for both approaches studied. On the other hand, mean values of S for eastbound and westbound approaches of Sunset Boulevard at Sepulveda are markedly different, apparently due to turning movements.

Within the range of street widths studied, smaller values of S were found for streets with no parking than for streets of the same curb-to-curb width but with parking. The difference between S values for the two parking conditions appears to decrease as the width increases. Better curves presumably can be drawn when data have been collected for more intersections and on a wider range of street widths.

CONCLUSIONS REGARDING S AND D

Several conclusions with respect to starting delay, D, can be drawn from examination of the data presented here.

1. There is a significant difference in starting delay among approaches at different intersections and among different approaches to the same intersection.

2. In general starting delay on one weekday can be considered equal to starting delay for all weekdays since no significant difference among mean D's on five different days was found at twelve of the thirteen intersections.

3. Factors which influence starting delay and are responsible for the difference in D among various locations have not been isolated and identified in this study. Volume of vehicle and pedestrian cross traffic, gradient, and percent of right turns are factors believed to have some effect. Type of signal control, width of the street, width of the cross street, and width of the pedestrian crosswalk appear to have little or no effect in the cases studied.

4. Starting delays are in most cases normally distributed. Departure from normality, where it exists, is in the form of positive skewing with a long tail of high values of D. In this study standard deviations of delays on a given day were found to be of the order of 1¼ sec., about 30 percent of the mean. A standard deviation of about ¼ sec. was found among mean values for five different days on the same intersection approach.

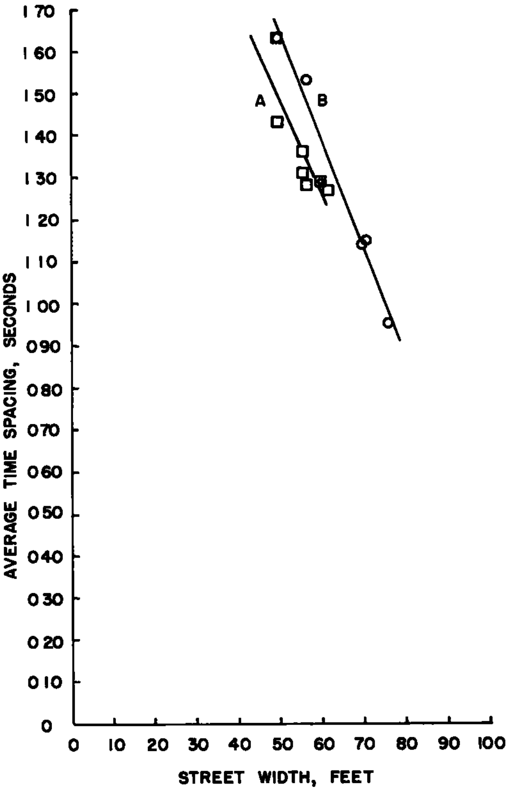

Figure 1. Time Spacing, S, as a function of street width. Curve A (points marked by squares): Approaches without parking. Curve B (points marked by circles): Approaches with parking.

Conclusions which may be drawn about time spacing, S, are the following;

1. Average time spacing is significantly different for different intersections, but the mean value obtained for one weekday will not usually differ significantly from that for other weekdays.

2. Although normality was not tested for all cases, time spacing appears generally to be normally distributed. The average standard deviation for an intersection approach is 0.23 sec., and this does not vary greatly from intersection to intersection. The dis-

tribution of mean S values for intersection approaches shows that all standard deviations of means among days are less than 0.1 sec. For six of the 13 approaches the standard deviation of daily means was less than .04 sec.

3. Time spacing on a given approach appears to be primarily a function of street width and parking conditions for the intersections studied. For streets with parking and without parking, time spacing S is approximately a linear function of street width over the range of street widths studied.

The above discussion of the experimental results indicates clearly that starting delay D and time spacing S are functions of conditions at individual intersections. Values for these parameters do not vary significantly from day to day in most cases. These statements are made for periods when traffic enters the intersection in platoon movement and there is a reservoir of waiting vehicles at the start of each green period, and are based on observations made only on week days.

These data indicate that while values of both D and S vary considerably from cycle to cycle, mean values vary somewhat less from day to day. This variability must be considered in applying these parameters to traffic problems and may restrict their usefulness

Additional studies of S and D for a wider range of intersection types are necessary to define in more detail the relationship of these parameters to physical and traffic conditions existing at intersections. A wider range of street widths should be studied. Turning movements should be studied for effect. No high volume downtown intersections were included among those studied here, but data for such locations are necessary for complete understanding of S and D.

APPLICATIONS TO CAPACITY

The maximum capacity of an intersection approach is related to D and S as discussed above and can be calculated using formula (II). Maximum capacity N can be estimated for a given intersection if S is measured for that approached or estimated from curves similar to those presented in this paper. S is a function primarily of street width and parking conditions, and a typical D may be assumed since N is not affected greatly by small changes in D.

Formula (II) shows that while intersection capacity can be increased by decreasing either S or D, a reduction in S will give the greater proportional increase in N. The results of this study indicate that S can be decreased by widening or by prohibiting parking. Only one intersection was studied at which left turns were prohibited, and the data, though inconclusive, indicate that time spacing was thus reduced. The "Highway Capacity Manual" (3) suggests several methods for increasing what it calls practical and possible capacity. Any of these operational measures except those which alter signal cycles can be effective if and only if they act to reduce either average time spacing of vehicles in platoon movement or the starting delay.

SIGNAL TIMING

Because of relatively large percentage variation in values from cycle to cycle, D and S are of limited value in determination of optimum signal timing. Formula (III), which is highly sensitive to changes in S and D, appears to require modification to allow for this variation if it is to accomplish the objective for which it was developed. To accommodate the traffic in 95 percent of all cycles in the peak period, Formula (III) might be improved by adding two standard deviations each to S and D in order to produce a larger value of T. The variability of S and D is so great, however, that their use in computing optimum cycle length is of doubtful value.

Use of these parameters is far more appropriate in determining the most effective cycle division when a total cycle length has been fixed by other considerations. The green time on any phase at a signalized intersection is composed of time for starting, time of actual vehicle movement, and extra time during which there is no movement. The last-named component should be divided between or among phases in proportion to the times actually required by each phase for the other components.

Expressed in terms of S and D, the time required for one phase per cycle is D + nS, where n is the number of vehicles entering that cycle. A reasonable basis for assigning a proportion of total green time to the two phases is:

$$g_1 : g_2 :: (D_1 + n_1S_1) : (D_2 + n_2S_2) \qquad \text{(IV)}$$

The above formula may be modified in one of several ways to meet individual conditions. Where the amber periods are used extensively for movement of vehicles, g can be replaced by (g + a). Where nS/D is very large, D can be omitted from consideration. Since extreme precision in cycle division is seldom required, the latter approximation can usually be used. This would divide the green time in direct proportion to the ratio of nS values for the heaviest legs of the opposing phases.

References

1. Greenshields, Bruce D., Schapiro, Donald, and Ericksen, Elroy L., "Traffic Performance at Urban Street Intersections", Eno Foundation for Highway Traffic Control, 1947. (Technical Report No. 1, Yale Bureau of Highway Traffic).
2. "Traffic Engineering Handbook", 2nd Edition, Institute of Traffic Engineers, 1950.
3. "Highway Capacity Manual", U. S. Department of Commerce, Bureau of Public Roads, 1950.
4. National Safety Council, Public Safety Memorandum No. 84.
5. "Timing Fixed-Time Signals", Institute of Transportation and Traffic Engineering Bulletin T-12, 1950 (mimeo).
6. Dixon, Wilfred J. and Massey, Frank J., Jr., "Introduction to Statistical Analysis", McGraw-Hill Book Co., New York, 1951.