352

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Bulletin 352

Traffic Characteristics and Intersection Capacities

II. Intersection Capacity

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Contents

A Study of Peaking Characteristics of Signalized Urban Intersections as Related to Capacity and Design

DONALD R. DREW and CHARLES PINNELL, Respectively, Research Assistant and Assistant Research Engineer, Texas Transportation Institute, A & M College of Texas

The Texas Transportation Institute in cooperation with the Texas Highway Department conducted a research project to study the operational aspects of signalized diamond interchanges. This research revealed a need for specific studies of peak traffic demand of signalized intersections, and it was considered necessary to devote a phase of the interchange studies to an investigation of peaking characteristics of signalized urban intersection. This paper presents results of this investigation.

Data on vehicle arrivals were obtained from 60 intersection approaches located in eight major cities in Texas. Peak-hour studies were conducted during morning and afternoon peaks at selected intersections. Analysis of data indicated the existence of a peak period within the peak hour. The average arrival rates during this peak period greatly exceeded the average arrival rate for the peak hour.

The peak period was defined in terms of its duration and magnitude. The duration of the peak period was taken as that interval of time within the peak hour in which the equivalent hourly rate of flow for 5-min intervals exceeded the peak hourly rate. The magnitude of the peak period was interpreted as the ratio of average arrivals for the peak period to average arrivals for the peak hour.

Through a multiple regression analysis based on the sample of 60 intersection approaches, the magnitude of the peak period was expressed in terms of (a) the population of the city in which the intersection was located, (b) the location of the intersection with respect to the CBD and the city limits, and (c) the number of vehicles arriving at the intersection on a given approach within the peak hour. A similar multiple regression analysis showed no significant correlation between the duration of the peak period and the same variables of population, location, and volume. The mean duration of the peak period for all approaches sampled was approximately 26 min.

The distribution of vehicle arrivals during the peak period and peak hour were analyzed by the χ^2 test under the hypothesis of a Poisson distribution. It was shown that (a) arrivals during the peak period did conform to a Poisson distribution, and (b) arrivals throughout the entire peak hour did not conform to a Poisson distribution.

Finally, some of the aspects of the capacity-design analysis of an intersection are considered in light of these findings. New design and signalization procedures are developed based on vehicle arrivals during the peak period.

•THE AT-GRADE intersection is one of the most critical elements of an urban street system because it exerts the greatest single influence on traffic operation. If a high level of service is to be obtained on urban highways and major arterials, proper design and signalization of the intersections on these facilities is imperative.

The systematic assignment of right-of-way between conflicting flows is accomplished most efficiently by the traffic signal. However, the signalized intersection creates a capacity-reducing effect on the roadways concerned. This reduction in capacity can be minimized only by the application of sound principles in the design and operation of the intersections.

The Highway Capacity Manual defines a "high-type" intersection as having the following characteristics:

1. High-type geometric design.
2. Separate lanes for conflicting movements.
3. All conflicting movements separated by signal phasing.
4. Parking eliminated.
5. Minimum pedestrian-vehicle conflicts.

In general, where major arterials intersect major arterials or urban highways, a high-type intersection is necessary. Because these facilities are such a vital part of an urban transportation system, a great challenge lies in their design and operation. In designing future intersections or in reconstructing existing ones, the proper selection of the number of approach lanes and the timing of the signal system in accordance with traffic demand hold the key to providing safe, efficient operation.

The purpose of this report is to discuss factors affecting the design and signalization of the high-type intersection and to develop procedures for designing and signalizing such facilities.

Designing for Peak Flows

Efforts to increase operational efficiency at signalized intersections have gained impetus in recent years, and the divergency of treatment are testimonials to the many manifestations of the problem. A report (1) prepared for the Bureau of Public Roads presents an analysis of the impact of some 48 variables on traffic flow through signal-controlled intersections using a complex multiple regression procedure. Bellis (2) describes an empirical relationship for the Nth vehicle in the queue to attain the 85 percentile speed and clear the intersection. He also suggests that the signal timing should be such that the maximum number of vehicles per cycle occurs only once during the design hour.

Capelle and Pinnell (3), in developing a workable formula for determining the capacity of diamond interchanges, reported the presence of a peak period within the peak hour which complicated the signalization analysis and indicated the need for specific study of this factor. The Design Manual (4) used by the Texas Highway Department cautions designers of urban radial freeways concerning peak rates of flow within the peak hour which greatly exceed the average rate of flow for that hour. This manual further suggested that the peak characteristics are related to the population of the city (Fig. 1).

Because the presence of a peak or plateau within the design hour seriously damages the case for random arrivals throughout the peak hour, some factor to be applied to the average rate of flow is needed to provide for this peak. Without identifying any such peak, Sagi (5) concludes that, because the cycle length is not known, the highest 1-min volume should be multiplied by 60 to obtain a design figure. This, of course, raises a

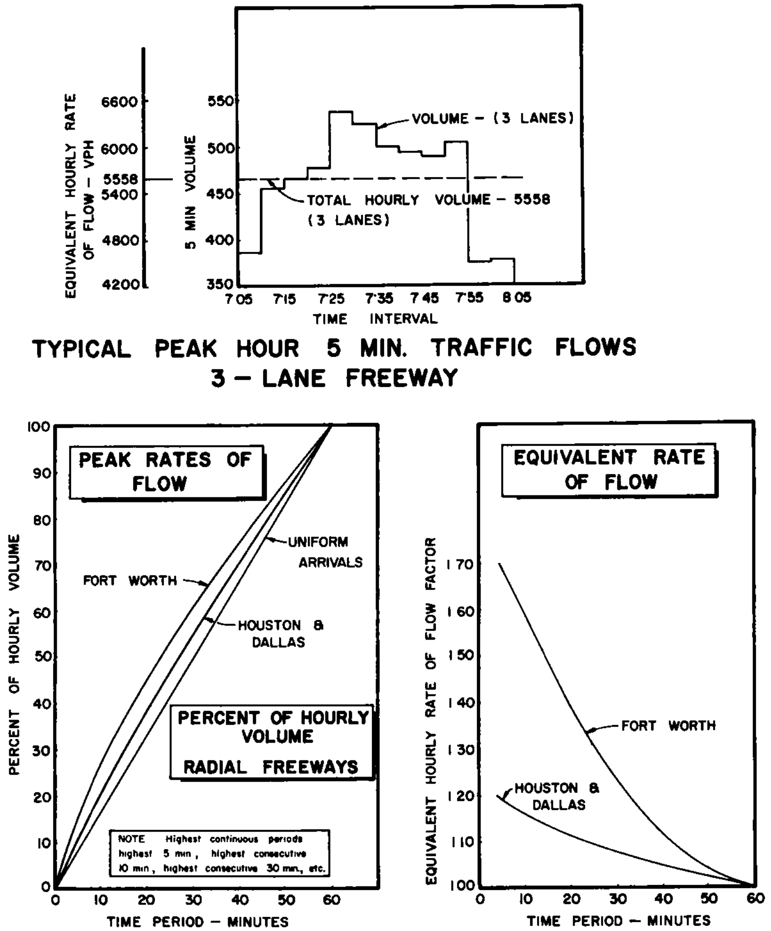


Figure 1. Peaking characteristics on urban radial freeways (from 13, Figs. 1-201.4 A, B, and C).

serious question in the case of new facilities where no 1-min volumes are available for expansion.

Time Apportionment

There are several methods available for apportioning green time to the various phases of a signal cycle. The relative precision of these solutions is proportional to the degree of realism achieved in the hypotheses regarding the arrival and departure rates.

The simplest procedure is based on the assumptions that the arrival rate is constant from cycle to cycle throughout the design hour and that the departure rate and hence the departure headways are constant throughout the green interval. Thus, the ratio of the duration of a given phase to the total cycle length is equal to the demand on the given phase divided by the demand on all phases. This has been referred to as the (G/C) method, and examples of its applications are described in the Highway Capacity Manual (6).

Greenshields (7) showed that the minimum average departure headways which result when a queue of vehicles is released by a light are gradually reduced until about the fifth or sixth vehicle in line, when a constant headway is developed. If arrivals are still assumed to be uniform, the computation of cycle length and apportioning of phases is still rational. The maximum capacity for a given phase may be obtained by a direct analysis of headways with allowances for time lost starting and stopping the queue, as suggested later in this paper.

Most existing procedures have been based on the assumption of a constant or average demand. However, traffic tends toward a random arrangement; the number of vehicles arriving at a given point in any interval of time can vary appreciably from the mean. The Poisson distribution is well established in predicting vehicle arrivals at intersections (8, 9, 10). The Poisson equation expresses the probability of a given number of vehicle arrivals per cycle based on the average number of arrivals per cycle. Inasmuch as it is apparent that for any reasonable cycle length some cycle failures must be expected, the number of tolerable failures may be used as a criterion for the cycle length determinations.

Objectives

After considering available research data which indicated the existence of a peak period within the peak hour, it was believed that a thorough analysis of peak traffic demand at signalized urban intersections was necessary to build more realism into a procedure for capacity design and time apportionment. Thus a research project to study peak traffic demand was planned with the following specific objectives:

1. To determine a practical means of defining the duration and magnitude of the peak period that exists within the peak hour and to find if there existed a means of predicting these two factors from known parameters.
2. To study the distribution of arrivals during the peak hour and to test the following two hypotheses concerning the application of a Poisson distribution:
   (a) Vehicle arrivals conform to a Poisson distribution throughout the peak hour.
   (b) Vehicle arrivals conform to a Poisson distribution during the peak period.
3. To illustrate the significance of any findings in relation to present theoretical concepts and to the solution of practical capacity-design and time apportionment problems at signalized intersections.

STUDY PROCEDURE

Identification of Peak Period

To define the peak period within the peak hour, it was necessary to utilize short time intervals for counting traffic demand. Intersection approach volumes recorded in 1-min intervals showed a marked fluctuation and little promise as a practical method of identifying the peak period. Therefore, a 5-min interval was arbitrarily chosen as a basis for grouping (Fig. 2). Still a distinct peak period was not apparent. However by superimposing the average peak hourly volume on the graph of 5-min volumes (Fig. 3), it was apparent that from 7:10 AM to 7:45 PM the average hourly rate of flow was exceeded. If the midpoints of the 5-min ordinates are connected, a polygon is formed which intersects the line of the average hourly volume at the extremities of what was designated as the peak period.

Thus, the duration of the peak period was defined as the continuous period of time within the peak hour in which the rate of arrivals, measured by 5-min intervals, exceeded the average hourly rate. The duration of the peak period was approximated either graphically (Fig. 3) or algebraically to the nearest minute. The peak hour was simply taken as that 60-min interval composed of the 12 highest consecutive 5-min volumes.

As suggested earlier, still another dimension was utilized in the identification of the peak period. This dimension, termed the magnitude of the peak period, was defined as the ratio of the average rate of arrivals during the peak period to the average rate of arrivals during the peak hour and may be represented by

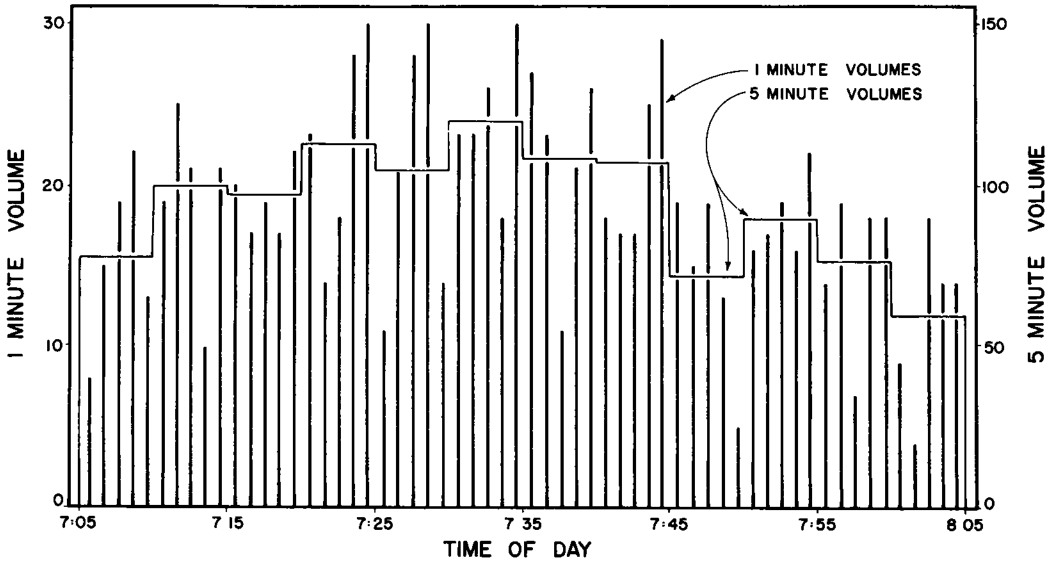


Figure 2. Peak-hour 1-min traffic demand, Heights Street approach at Sixth Street, Houston, Texas, Dec. 20, 1960.

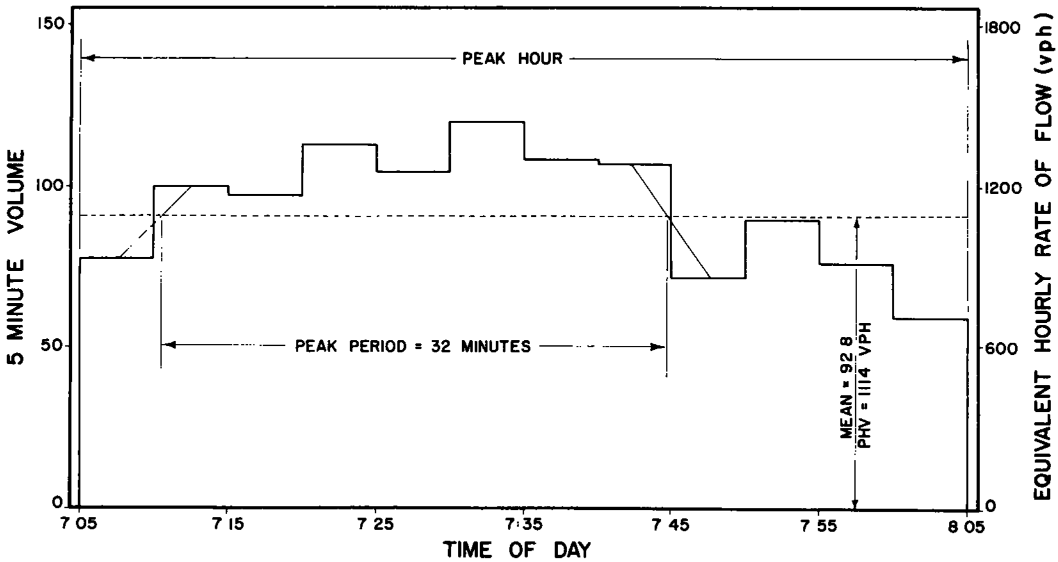


Figure 3. Peak-hour 5-min traffic demand, Heights Street approach at Sixth Street, Houston, Texas, Dec. 20, 1960.

$$\text{Magnitude} = \frac{\text{Average rate during peak period}}{\text{Average rate during peak hour}} \qquad (1)$$

This magnitude factor will be greater than 1.000; and if Eq. 1 is solved for the numerator, it becomes apparent that the magnitude factor represents the amount that the peak hourly volume must be increased to adjust for the higher rate of flow during the peak period.

Choice of Variables

In planning future facilities, the values of the duration and magnitude would necessarily have to be determined from some means other than 5-min traffic counts. One of the objectives of this report was to ascertain if these factors could be estimated in terms of known parameters.

The selection of these independent variables presented somewhat of a problem. A few of the possibilities included the location of the intersection, the demand on the approaches, the proximity of generators, the presence of traffic congestion on the streets composing the intersections, as well as capacity mitigating factors such as parking, busses, pedestrians, width of lanes, weather, speed limits, traffic composition, and turning movements. As was suggested, many of these are "capacity" factors and influence demand only indirectly; others are qualitative and therefore arbitrary; finally, many are impertinent in that they would be unknown in the case of the design of a new facility. The final choice of variables was arrived at through both a process of elimination and practical considerations in controlling the scope of this investigation.

In summary, the experimental design for this aspect of the study was based on relating the duration and magnitude of the peak period to the following three independent variables:

1. The population of the city in which the intersection is located.
2. The location of the intersection with respect to the central business district (CBD).
3. The peak hourly volume of the intersection approaches.

Selection of Intersections and Data Obtained

Pilot studies were conducted at Waco and Houston, Texas, to determine the final study procedure that would be necessary and to define the specific data that would be required. It was recognized that actual traffice demand must be measured (arrivals, not departures), and a counting technique was developed for this purpose. Vehicles on the approaches were counted before they were stopped by either the traffic signal or traffic queues at an intersection (Fig. 4). Because the queue length on the approaches increased greatly during the peak period, counting devices that depended on road tubes or other stationary sensing devices were too inflexible. It was found, however, that one man equipped with a manual counter, a stop-watch, and an ordinary watch was able to record efficiently the required data for one intersection approach. Lane distribution, traffic composition, and turning movements were not considered, as these were capacity factors and had no effect on vehicle arrivals.

With population designated as one of the independent variables in the investigation, it was necessary that the studies reflect a desirable range of population. The following eight cities (population shown in parentheses) were selected as locations for conducting the required traffic demand studies:

1. Houston	(941,000)
2. Dallas	(680,000)
3. San Antonio	(585,000)
4. Fort Worth	(356,000)
5. Austin	(170,000)
6. Corpus Christi	(168,000)
7. Amarillo	(137,000)
8. Waco	(101,000)

Because time and financial limitations precluded personal execution of the field work, letters were sent to the traffic engineers of these cities explaining the proposed project and soliciting their aid in obtaining field data. All responded by expressing a willingness to conduct the necessary studies with their personnel.

Eight studies requested from each city were to give equal representation to the morning and afternoon peaks (four morning and four afternoon studies). Mimeographed sheets explaining the method of study and limitations in the choice of intersection along with data sheets for recording information were provided to each of the cities. Copies of the instructions and a typical data form are shown in Appendix A.

Analysis of Arrivals

Inasmuch as volume counts recorded at 5-min intervals afforded no basis for analyzing the distribution of arrivals, it was necessary to select a shorter counting interval for this phase of the investigation. Theoretically, a counting interval that approximated the average cycle length at a signalized urban intersection was needed to give an indication of the distribution of arrivals per cycle. A 1-min counting interval corresponding to a minimum 60-sec cycle during the morning and afternoon peaks seemed reasonable.

A statistical test of significance was still to be considered. The χ^2 test seemed appropriate (8, 9, 10). Two restrictions imposed by the χ^2 analysis helped set a maximum limit in the determination of a volume-counting interval. Because the theoretical frequency must be at least five in any group and the degrees of freedom for the Poisson distribution are two less than the number of groups, there must be a minimum of three groups of five (or 15 intervals) in order to utilize the χ^2 test. Because peak periods of 15 min were conceivable, intervals greater than 1 min were prohibitive. Thus, in the selection of a counting interval the practical minimum established by the cycle length and the practical maximum established by the test of significance coincided. One-minute counts of vehicle volume were conducted for a sufficient duration to bracket the peak hour.

For this analysis of the distribution of arrivals, eight intersection approaches were chosen. The locations selected were in College Station, Bryan, Waco, and Houston, Texas. The study procedure was the same as previously discussed except that demand volumes were recorded by 1-min intervals.

ANALYSIS OF DATA

Multiple Regression Analyses of Peak Factors

The data obtained from 5-min volume studies conducted during peak hours are summarized in Appendix B under the cities in which the studies were conducted. The peak hour and peak periods are identified for each approach. The peak hour is represented by the period composed of the 12 highest consecutive 5-min volumes. The measurements pertinent to the regression analysis are summarized in Table 1. The dependent variables or peak factors are

Y = the duration of the peak period to the nearest one-tenth of a minute as determined by the method shown in Figure 3;

Y'= the magnitude of the peak period or the ratio of the average arrivals during the peak period to the average arrivals during the peak hour.

The dependent variables shown are

X_1 = the population of the cities where the intersections are located expressed in thousands;

X_2 = the distance of the intersection from the CBD in miles; or

X_2' = the ratio of the distance between the intersection and the CBD to the total distance between the CBD and the city limits;

X_3 = the peak hourly volume for the approach.

The organization of Table 1 suggested four separate analyses for each of the peaks—AM and PM. These combinations are $X_1X_2X_3Y$ (PM); $X_1X_2X_3Y'$ (PM); $X_1X_2X_3Y$ (AM); $X_1X_2X_3Y'$ (AM); $X_1X_2X_3Y$ (PM); $X_1X_2'X_3Y'$ (PM); $X_1X_2'X_3Y$ (AM); and $X_1X_2'X_3Y'$ (AM). Each analysis required the calculation of three regression coefficients "b" (one for each of the independent variables). This is accomplished through the solution of three simultaneous equations for each case. Because interval estimates and tests of significance for these coefficients require the calculation of the elements of an inverse matrix (c), a procedure utilizing (c) was selected. Snedecor (12) suggests tabular forms which, modified slightly, have been employed in Appendix C and D.

Two things remain to be evaluated. First, there must be an over-all test of significance of the regression. This evaluation of the over-all regression was accomplished by an "analysis of variance" procedure and an F-test (Appendix E). Second, the tests of significance for the regression coefficients indicated (for the population sampled) which of the independent variables is the best predictor of the dependent variable. This was determined by the t-test (Appendix D).

χ^2 Tests of Arrivals

Eight volume counts are summarized in Appendix F. They were conducted so as to bracket the duration of the peak hour, and the vehicle arrivals were recorded by 1-min intervals. The first step in the fitting of the Poisson distribution to the experimental data was the classification of the arrivals by frequency. Thus, for each peak period and each peak hour, the number of 1-min intervals in which 0, 1, 2, 3, etc., vehicles arrived was tabulated. These constituted the "observed" distributions, and the inference was then made that the postulated theoretical (Poisson) distribution is in fact the true population. The use of the χ^2 test as an index of the correlation of observed and expected frequencies of occurrence is well established in testing such an hypothesis. The calculations are shown in Appendix G.

So far in the analysis of arrivals, only the frequency has been considered. In the event that the χ^2 tests verified a Poisson distribution for the peak period, it would be well to check the independence of arrivals for successive intervals. Thus, if the average number of arrivals during the peak period is 10 vehicles per minute, the probability of 10 or more arrivals during a minute is 0.54 (from tables of the Poisson distribution). Similarly, during any consecutive pair of 1-min intervals, the probability of 10 or more arrivals for both intervals is 0.54×0.54. The probability of less than 10 arrivals for both intervals is 0.46×0.46. Two additional possibilities remain:

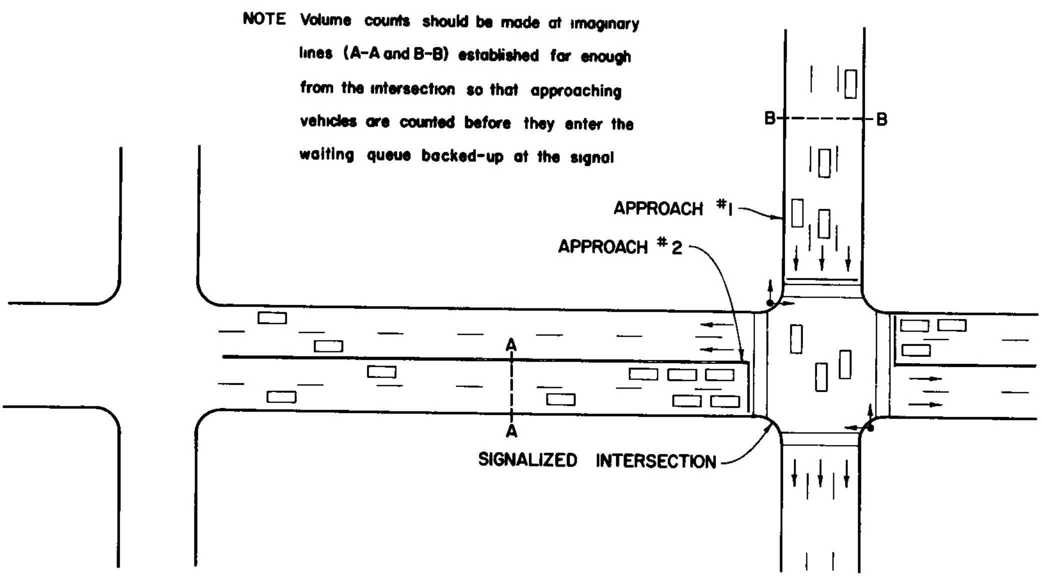


Figure 4. Field Measurement of demand.

TABLE I

SUMMARY OF PEAK VARIABLES AND PEAK FACTORS

PM PEAK						PM	CITY	A.M.	AM PEAK						
POPULATION	LOCATION		PEAK	PEAK FACTORS					PEAK FACTORS		PEAK	LOCATION		POPULATION
IN THOUSANDS X_1	CBD (MILES) X_2	RATIO X_2'	HOURLY VOL. X_3	DURATION Y	MAGNITUDE Y'	APPROACH NUMBER		APPROACH NUMBER	MAGNITUDE Y'	DURATION Y	HOURLY VOL. X_3	RATIO X_2'	CBD (MILES) X_2	IN THOUSANDS X_1
137	2 1	043	650	163	1 263	1	AMARILLO	1	1 311	2 1 1	850	043	2 1	137
137	2 1	0 34	1101	25.2	1 202	2		2	1 197	264	632	058	2.1	137
137	3 3	062	996	25.0	1 241	3		3	1 380	232	551	0.50	2 7	137
137	3 3	0 62	489	174	1 192	4		4	1 422	24 5	565	0.59	2.7	137
585	3 7	0 63	1778	35 1	1 151	5	SAN ANTONIO	5	1 158	30.2	2810	063	3 7	585
585	5 2	0 58	2047	38 7	1 164	6		6	1 117	24 1	1779	0 58	5 2	585
585	1 9	067	1595	22 3	1 086	7		7	1 146	24 5	1150	067	1 9	585
585	1 1	0 11	1292	209	1 082	8		8	1 131	25 5	1175	0 10	1 1	585
170	1 9	0 56	1155	169	1 196	9	AUSTIN	9	1 120	25 3	779	047	3 1	170
170	0 5	0 26	799	29.0	1 157	10		10	1.217	28 3	2171	048	3 1	170
170	3 1	0 48	2363	21 5	1 278	11		11	1084	36 0	1267	0 15	1 0	170
170	3 1	047	742	28 1	1 088	12		12	1 168	23 5	717	031	1 0	170
941	4 2	0 49	2636	28 3	1 119	13	HOUSTON	13	1 123	32 2	1114	044	3 7	941
941	2 9	0 35	2403	39 1	1 156	14		14	1 142	30 2	1100	0.52	4 3	941
941	3 8	0 45	947	279	1 115	15		15	1 104	33 9	1581	045	3 8	941
941	5 0	0 62	1059	17.3	1 084	16		16	1 149	20 3	1665	0 20	1 5	941
101	1 7	0 20	629	21 8	1 261	17	WACO	17	1 146	28 9	1063	029	2 3	101
101	1 7	0.20	1120	336	1 127	18		18	1.289	23 9	1065	0 33	2.0	101
101	1 4	0 18	719	18 1	1 420	19		19	1 136	28 8	897	028	1 7	101
101	2 6	043	931	21 4	1 159	20		20	1.262	23 1	821	0 28	1 7	101
680	3 3	0.35	907	136	1 163	21	DALLAS	21	1 126	25 8	1390	042	4.0	680
680	3 7	0 38	2387	20 4	1 113	22		22	1 156	34 6	1260	0 38	4.0	680
680	7 0	0 69	2561	30 1	1 069	23		23	1 162	24 2	2377	0 69	7.0	680
680	7 0	0 76	1195	33.0	1 095	24		24	1 113	13 5	1172	0 76	7.0	680
356	3 9	0 72	1591	30.8	1 108	25	FORT WORTH	25	1 192	25 0	872	0 72	3.9	356
356	3 9	061	1264	27 9	1 104	26		26	1 129	31 1	1183	061	3.9	356
356	3 1	0 37	708	30 4	1 168	27		27	1.203	26 4	718	0.37	3 1	356
356	3 1	0.50	763	28 7	1 170	28		28	1 144	32 8	705	0 50	3 1	356
168	1 0	0 17	743	19 4	1 174	29	CORPUS CHRISTI	A M Studies were not available from Corpus Christi						
168	1 8	0 16	1075	23.0	1 206	30								
168	1 8	0 16	716	21 5	1 189	31								
168	3 6	0 50	893	18 7	1 161	32								

(a) 10 or more in the first interval and less than 10 arrivals in the second, and (b) the reverse, or less than 10 arrivals in the first 1-min interval and 10 or more in the second. The probabilities of either of these combinations is 0.54 × 0.46. The point is that a run of consecutive 1-min arrivals above the mean followed by a similar run of arrivals below the mean might not yield a significant χ^2 value in the analysis of frequency of arrivals, yet it could obviate true randomness.

A typical analysis of this aspect of independence of arrivals for successive intervals is shown in Appendix H. The data were taken from Appendix C (Heights and Sixth in Houston) and also appear in Figure 2. The analysis consisted of two parts: (a) determining the observed combinations of arrivals for consecutive intervals, and (b) comparing these with the expected combinations of arrivals for consecutive intervals assuming a Poisson distribution. This later comparison was also affected by the χ^2 test.

STUDY RESULTS

Magnitude of Peak Period

The results of the regression analyses for the magnitude of the peak period are summarized in Table 2. It was found that the magnitude (Y') of the peak period may be expressed in terms of the population of the city (X_1), the location of the intersection within the city (X_2) or (X_2'), and the hourly volume on the intersection approach (X_3). This is true for both the AM and PM peaks. Moreover, (X_2'), the ratio of the distance of the intersection from the CBD to the city limits, contributes more to the estimation of the magnitude that (X_2), the distance between the intersection and the CBD.

Substituting in the general regression equation, the magnitude of the AM peak period becomes

$$\hat{Y}' = \bar{y}' + b_{Y1.23}(X_1 - \bar{x}_1) + b_{Y2.13}(X_{2'} - \bar{x}_{2'}) + b_{Y3.12}(X_3 - \bar{x}_3) \qquad (2)$$

in which $\bar{y}' = 1.1795$; $\bar{x}_1 = 424.3$; $\bar{x}_2' = 0.4546$; $\bar{x}_3 = 1,193.9$; and for b see Appendix D. Therefore,

$$Y' \text{ (AM)} = 1.222 - 0.000125X_1 + 0.09X_2' - 0.000027X_3 \tag{3}$$

Similarly, the following values for the PM peak—$\bar{y}' = 1.1644$; $\bar{x}_1 = 392.3$; $\bar{x}_2' = 0.4394$; $X_3 = 1,257.9$; and for b, see Appendix D. Therefore

$$\hat{Y}' \text{ (PM)} = 1.228 - 0.000145X_1 - 0.11X_2' + 0.000033X_3 \tag{4}$$

If Eqs. 3 and 4 are averaged, term by term, the result is a common expression for both peaks, except for one sign (AM, positive; PM, negative):

$$\hat{Y}' = 1.225 - 0.000135X_1 \pm (0.1X_2' - 0.00003X_3) \tag{5}$$

in which

X_1 = population of city/1,000;

$X_2' = \dfrac{\text{distance between intersection and CBD}}{\text{distance from CBD to city limits}}$;

X_3 = peak hourly volume per approach (PHV); and

$\hat{Y}'$ = magnitude of the peak period.

Since by definition

$$\hat{Y}' = \frac{\text{m (average arrivals during peak period)}}{\text{m}' \text{ (average arrivals during peak hour)}} \tag{6}$$

in which

$$m' = X_3/C' \text{ (number of cycles per hour)},$$

it is apparent that average arrivals per cycle during the peak period for an approach may be predicted directly from the three independent variables.

$$m = \frac{X_3}{C'} [1.125 - 0.000135X_1 \pm (0.1X_2' - 0.00003X_3)] \tag{7}$$

Duration of Peak Period

The results of the regression analyses for the duration of the peak period are summarized in Table 3. The duration of the peak period proved to be statistically unrelated to the three independent variables chosen. Although this regression was not significant, 14.3 percent of the variance of the duration of the PM peaks could be attributed to these three factors—population, location, and hourly volume. Thus, the best estimates of the duration of either an AM or PM peak period were their respective means.

TABLE 2[a]

RESULTS[b] OF ANALYSES FOR MAGNITUDE OF PEAK PERIOD (Y')

Peak	F-Test	t-Test X_1	X_2 or	X_2'	X_3	R^2 (%)
PM (X_2')	**	**		N	N	36.8
AM (X_2')	*	*		N	N	31.0
PM (X_2)	*	**	N	N	N	29.9
AM (X_2)	*	*	N	N	N	28.0

[a]Results taken from calculations in Appendices D and E.
[b]N = non-significant relationship.
* = significant relationship.
** = highly significant relationship.
R^2 = percentage of variance explained by multiple regression analysis.

The estimates for the duration of the AM and PM peak periods are obtained by

$$\hat{Y} = y \pm (t_{0.05} S_{\bar{y}}) \tag{8}$$

in which

$\hat{Y}$ = estimate of duration of peak period;

$\bar{y}$ = mean of sample durations; and

$S_{\bar{y}}$ = standard error of mean.

Thus, $\hat{Y}(AM) = 26.69 \pm (2.052)(0.92) = 26.69 \pm 1.89$; and $\hat{Y}(PM) = 25.04 \pm (2.042)(1.18) = 25.04 \pm 2.41$.

Distribution of Arrivals

The χ^2 tests show that the assumption of a Poisson distribution for vehicle arrivals during the entire peak hour was not valid. Out of the eight tests applied to the peak hour data (summarized in Table 4), four were significant at the 0.01 confidence level, or highly significant. The interpretation is that unless there was a one-in-a-hundred mischance in sampling, the null hypothesis is incorrect. Two of the remaining four studies were significant at the 0.05 level and the other two studies were not significant.

The same hypothesis for the peak period, however, was rejected only once in eight studies, which tends to establish that arrivals during the peak period did conform to a Poisson distribution. Expressed in the terms of the statistician, the conclusion is that the true distribution of arrivals during the peak period (of which the observed data constitute a sample) could be identical with the postulated (Poisson) distribution.

The check for the independence of arrivals for successive 1-min intervals proved to be academic (Appendix H). The observed variations in arrivals in consecutive intervals agreed almost exactly to that expected for a Poisson distribution. This property of recovery—or the tendency of fluctuations in the number of arrivals from one interval to another--has practical as well as theoretical ramifications. Thus, queues lengthened during one cycle have an opportunity to clear out in successive cycles.

In the introduction to this paper, assumptions regarding arrivals or demand were discussed with regard to their effects on capacity-design procedures. Currently, one of two assumptions is employed: (a) Poisson arrivals throughout the peak hour, or (b) uniform arrivals throughout the peak hour. Figures 5 and 6 show the relationships between arrivals predicted by these respective assumptions and the observed arrivals for the approaches studied. Superimposed on the graphs are the Poisson arrival and uniform arrival curves for the peak period. The curves for uniform arrivals are straight lines representing the average arrivals for the peak hour and the peak period, respectively. It is apparent that the best estimator of the observed demand is the assumption of a Poisson distibution during the peak period, whereas the least reliable is the assumption of uniform arrivals throughout the peak hour. Of singular importance is the fact that an assumption of uniform arrivals for the peak period presents a design tool as good as or better than the assumption of a Poisson distribution for the peak hour.

TABLE 3[a]

RESULTS[b] OF ANALYSES FOR DURATION OF PEAK PERIOD (Y)

Peak	F-test	t-Test X_1	t-Test X_2 or X_2'		t-Test X_3	R^2 (%)
PM (X_2')	N	N		N	N	14.3
AM (X_2')	N	N		N	N	8.7
PM (X_2)	N	N	N		N	14.3
AM (X_2)	N	N	N		N	9.2

[a]Results taken from calculations Appendices D and E.
[b]N = non-significant relationship.
* = significant relationship.
** = highly significant relationship.
R^2 = percentage of variance explained by multiple regression analysis.

Importance of Findings

Referring again to Figures 5 and 6, by definition, the average of arrivals during

TABLE 4

SUMMARY OF χ^2 TESTS FOR THE DISTRIBUTION OF ARRIVALS[a, b]

Study	Peak hour			Peak period		
	χ^2	d.f.	Probability	χ^2	d.f.	Probability
1	13.1	7	0.10>P>0.05	0.9	3	P>0.07
2	25.2**	7	P<0.001	10.6*	3	0.02>P>0.01
3	64.3**	6	P<0.001	2 1	2	0.50>P>0.30
4	17.0*	7	0.02>P>0.01	2.2	1	0.20>P>0.10
5	14.2*	6	0.05>P>0.02	4.2	4	0.50>P>0.20
6	4.0	7	P>0.70	2.7	2	0.30>P>0.20
7	22.7**	6	P<0.001	3.2	2	P = 0.20
8	24.3**	7	P = 0.001	0.4	1	P = 0.50

[a]See Appendix F.
[b] * = significant and the hypothesis of a Poisson distribution is rejected at the 5% confidence level.
** = highly significant and the hypothesis of a Poisson distribution is rejected at the 1% confidence level.

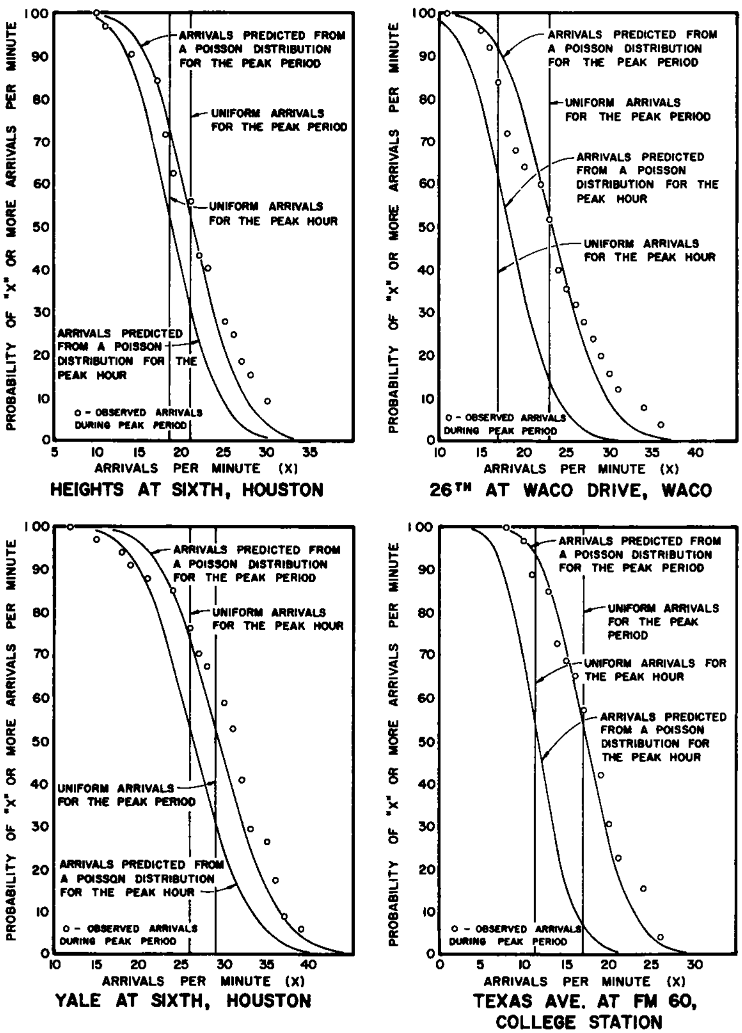


Figure 5. Relationship between observed and predicted arrivals during AM peak periods.

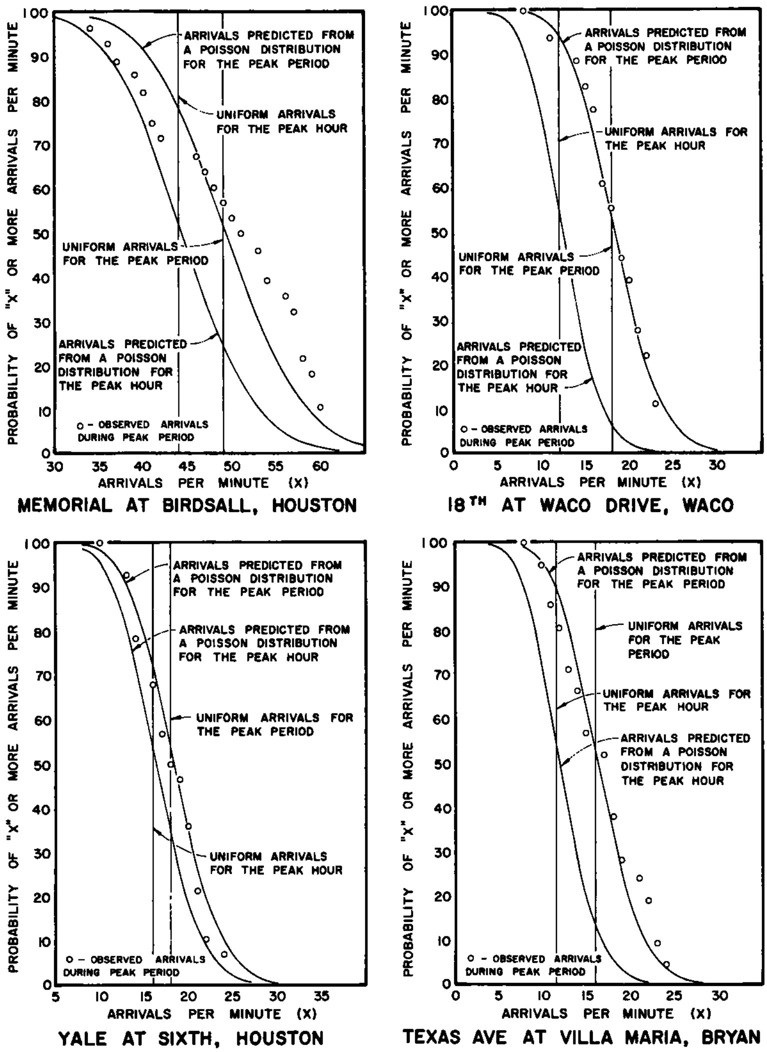


Figure 6. Relationship between observed and predicted arrivals during PM peak periods.

the peak period divided by the average of arrivals per peak hour is actually the magnitude factor (Y') evaluated through the multiple regression analysis. This magnitude factor serves a function analogous to the conversion between possible and practical capacity defined in the Highway Capacity Manual. Thus, it is apparent that these findings not only provide a basis for new design procedures, but present a very real means for perfecting existing procedures.

It would seem then that given an intersection capacity-design problem with peak-hour volumes obtained from a prediction of the 30th highest hour or design hourly volume, the first step would be to apply the magnitude factor to convert the hourly demand to the peak period demand. For a new facility this factor would be estimated by Eq. 5. The determination of the number of lanes and signal time apportionment could be based on this new increased volume under the assumption of uniform arrivals. If further sophistication is desired, it has been shown that demand for the duration of the peak period conforms to a Poisson distribution.

Thus, design can be placed in perspective, namely, a probability of demand exceeding capacity. To be completely meaningful, the duration of the peak period must be established because the peak period would have replaced the peak hour as the interval of design. The net effect of these findings is to replace the design hour with a shorter design interval called the peak period, just as the day was replaced by the hour as the interval for traffic design not too long ago. With this analogy in mind, the results of these findings will be applied to some of the aspects of capacity-design problems in the next section.

APPLICATION TO DESIGN AND TIME APPORTIONMENT PROCEDURE

Development of Capacity Equations

Until now the capacity of a high-type signalized intersection has been discussed in general terms. Because any design procedure is actually a systematic attempt to resolve the capacity-demand relationship, it is important that the development and limitations for capacity expressions be understood. Historically, the capacity of an intersection approach was derived through an analysis of vehicle headways (7). Many equations presently in use have preserved this relationship.

To visualize intersection performance, it is convenient to plot the conditions on a time-space diagram as shown in Figure 7. Although a simple two-phase system is shown, the theory can be extended to any multiphase combination. If the ordinate represents distance and the abscissa, time, the lines proceeding from bottom to top show the progress of vehicles approaching and leaving an intersection. If (x-1) vehicles cross the stop line during a time equal to $G - (K_1 + K_2)$, then the average minimum headway (D) is given by

$$\text{Average minimum headway} = \frac{\text{Time}}{\text{Volume}}$$

or

$$D = \frac{G - K}{x - 1} \tag{9}$$

in which

$K = K_1 + K_2$;
K_1 = starting delay for getting the entire queue into motion;
K_2 = time necessary for last vehicle to cross the intersection.

Because the last vehicle is legally allowed to cross on amber, the G-value equals green plus amber time. Rewritten, the expression becomes

$$G = (x-1)\ D + K$$
$$G = xD + (K - D) \tag{10}$$

For a given approach, K_1, D, and K_2 must be determined. K_2 is computed by dividing the width of an intersection plus the length of a vehicle by the speed of the last vehicle. K_1 and D are found from the measurement of the time intervals between vehicles as they cross the stop line. From both the time-space diagram of Figure 7 and the operational data of Table 5, it can be seen that these time intervals decrease until the average minimum headway (D) is reached. Thus, in reducing the data in Table 5, each interval is made up of two components, the deparature headway and a starting delay.

In this treatment of capacity, amber time does not directly affect the phase lengths or cycle lengths, and hence capacity. Because K_2, the time lost bringing the queue to a stop or the time lost in crossing an intersection, is a function of the intersection

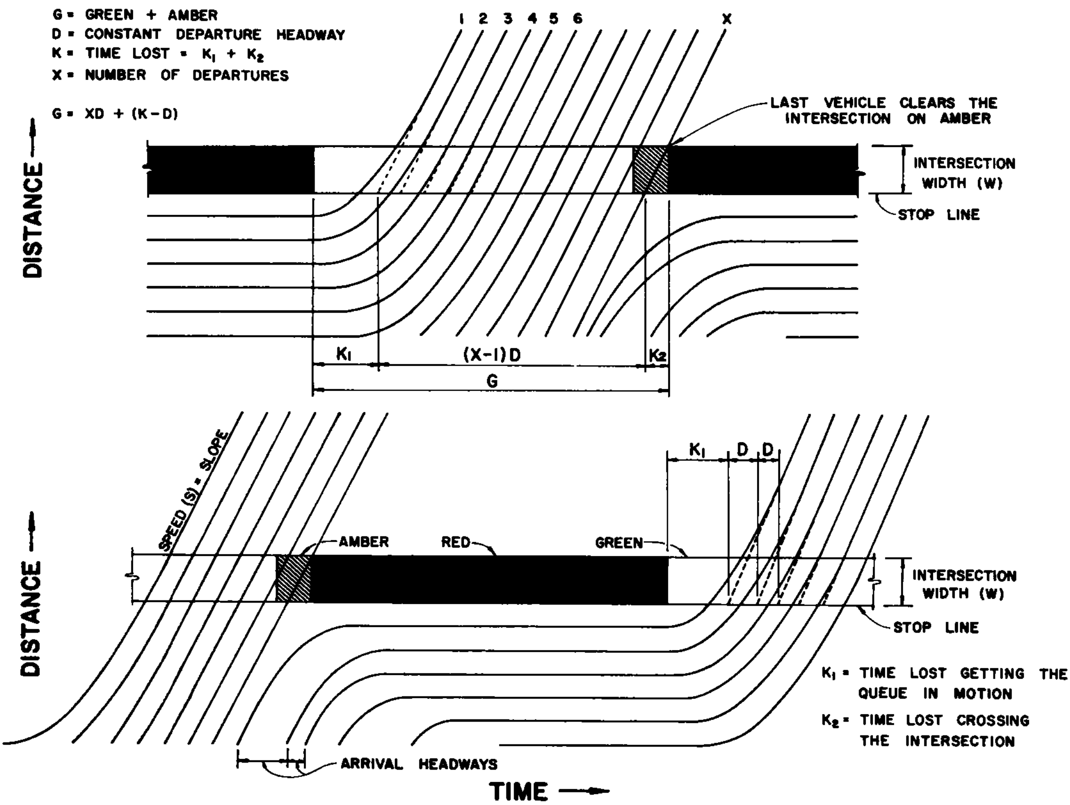


Figure 7. Time-space relationships for two-phase system.

TABLE 5

TYPICAL REDUCTION OF TIME INTERVALS BETWEEN SUCCESSIVE VEHICLES INTO DEPARTURE HEADWAYS AND STARTING DELAYS[a, b]

Vehicles	Interval, I	Headways, D	Delays, K_1
0-1	2.8	0.	2.8
1-2	2.6	2.0	0.6
2-3	2.1	2.0	0.1
3-4	2.1	2.0	0.1
4-5	2.0	2.0	0.0
5-6	2.0	2.0	0.0
Total	13.6	10.0	3.6

[a]See Appendix D for data.

[b] D (departure headway) = 2.0 sec;
K_1 (starting delay = Σ(I-D) = 3.6 sec.

width and approach speeds, there is some correlation between it and the amber time. However, one is not determined from the other. Increasing the amber time from 2 sec on an intersection approach theoretically should not reduce the capacity, because the legal stipulation regarding the position of the last vehicle crossing is the same; namely, that the vehicle be across the area of conflict when the signal turns red. The point is that in the capacity analysis, amber time should be disregarded. When the capacity analysis is complete, amber time may be considered based on such additional factors as geometrics, sight distance, and stopping distance.

In determining the parameters, D and K_1, recent headway studies reported by Capelle and Pinnell (3) were used. The measurements which are summarized in Appendix D were reduced by methods given in Table 5 and then classified as to movements (Table 6). Thus, values of

$D = 2.0$ sec and $K_1 = 4.0$ sec seem representative. K_2 equals approximately 2.0 sec (based on an intersection width of 50 ft, a speed of 30 mph, and an allowance of 30 ft for the length of vehicles).

In a capacity analysis, it is convenient to choose a critical lane volume per phase (V). This critical lane volume represents the maximum hourly volume per lane that can move through the intersection on a given phase.

$$V = \left(\frac{3,600}{C}\right) x \tag{11}$$

in which $x = \frac{G - (K-D)}{D}$ from Eq. 10.

The sum of hourly critical lane volumes (ΣV) for all phases gives the total critical lane volume that can negotiate the intersection per hour.

$$\Sigma V = \left(\frac{3,600}{C}\right) \Sigma x \tag{12}$$

$$\Sigma V = \left(\frac{3,600}{C}\right) \frac{\Sigma G - \varphi (K-D)}{D} \tag{13}$$

in which $C = \Sigma G$; φ = number of phases.

$$\Sigma V = \left(\frac{3,600}{C}\right) \frac{C - \varphi (K-D)}{D} \tag{14}$$

$$\Sigma V = \left(\frac{3,600}{D}\right) - \frac{3,600 \varphi (K-D)}{CD} \tag{15}$$

Substituting in $K = 6.0$ sec and $D = 2.0$ sec, an expression for (ΣV) can be obtained in terms of cycle length (C) for both three- and four-phase intersections ($\varphi = 3$, $\varphi = 4$).

$$\Sigma V (\varphi = 3) = 1,800 - \frac{21,600}{C} \tag{16}$$

$$\Sigma V (\varphi = 4) = 1,800 - \frac{28,800}{C} \tag{17}$$

Tabulations of (ΣV) for various cycle lengths (C) are given in Table 7. As C approaches infinity, ΣV approaches 1,8000 vehicles per hour per lane.

Last, Eq. 14 may be solved for cycle length (C):

$$C = \frac{3,600 \varphi (K-D)}{3,600 - D \Sigma V} \tag{18}$$

It should be remembered that Eqs. 10 through 16 are based on the assumption of uniform arrivals for every cycle over a 60-min period. This, of course, does not occur. However, if the hourly approach volumes are increased by the peak magnitude Factor (Y'), the capacity equations become applicable for uniform arrivals during the peak period.

Because it has been established (Figs. 5 and 6) that during the peak period the assumption of Poisson arrivals provides the best estimate of actual demand, it is well to consider its application in capacity determinations. Eq. 19 is the cumulative

TABLE 6

SUMMARY OF DEPARTURE HEADWAYS AND STARTING DELAYS FOR THROUGH AND TURNING MOVEMENTS

Type Movement	D	K_1
Through	2.0	4.0
Left turn	2.0	3.9
Right turn	2.0	4.1
Side-by-side turn:		
Inside lane	2.2	4.7
Outside lane	2.4	5.3

Poisson expression for determining the probability of (X+ 1) arrivals or more per cycle during the peak period based on an average of m arrivals per cycle.

$$P_{(x+1)(m)} = \sum_{x+1}^{\infty} \frac{m^{x+1} e^{-m}}{(x+1)!} \quad (19)$$

in which

$m = V \div (3,600/C)$;

$x = \frac{G - (K-D)}{D}$ from Eq. 10; and

$C = \Sigma G$.

These four equations can be reduced by successive approximations. This reduction is greatly facilitated through the use of a graph of cumulative Poisson curves (Fig. 8). The philosophy is to provide the designer with a figure of merit in the form of P, the percentage of cycle failures with a cycle failure defined as any cycle during which approach arrivals exceed the capacity for departures.

Illustration of Capacity-Design Procedure

Figures 9, 10, and 11 show seven steps to be followed in the design and signalization of a future high-type intersection:

1. The three conditions (population, location, and volumes) that affect the magnitude of peak period are determined.
2. Because the volumes are given in terms of ADT, they must be converted to peak hourly volumes. In an actual problem, the AM peak would also be checked.
3. The peak magnitude factor ($\hat{Y}'$) for each approach is calculated using the regression equation.
4. The peak magnitude factors ($\hat{Y}'$) are applied to the peak hourly rate of flow equivalent to the arrivals during the peak period.
5. Consistent with the assumption of a high-type facility, all conflicting movements must be separated by the signal phasing.
6. Design combinations are assumed by varying the number of approach lanes on the two streets. Volumes are assigned to each lane assuming equal lane volume required to move on a given phase is called the critical lane volume. The sum of these critical lane volumes for all phases provides the basis for calculating the minimum cycle length by use of either Eq. 18 or Table 7. In the example chosen, design alternative A yielded an unreasonable cycle length, and therefore only alternatives B and C merited further consideration.
7. The average arrivals per cycle (m) are calculated from the critical lane volumes. These values are used to enter the graph of Poisson curves (Fig. 11), and the phase lengths (G) are tabulated for various probabilities of failure (P). Any combination of $G_A + G_B + G_C + G_D$ that equals the assumed cycle length (C) is acceptable.

TABLE 7

CRITICAL LANE VOLUMES VS CYCLE LENGTHS FOR THREE- AND FOUR-PHASE HIGH-TYPE INTERSECTIONS

Cycle Length, C (Sec)	Summation of Hourly Critical Lane Volumes $\left(1,800 - \frac{7,200\varphi}{C}\right)$	
	$\varphi = 3$	$\varphi = 4$
40	1,260	1,180
50	1,368	1,224
60	1,440	1,320
70	1,491	1,389
80	1,530	1,440
90	1,560	1,480
100	1,584	1,512
110	1,604	1,538
120	1,620	1,560
∞	1,800	1,800

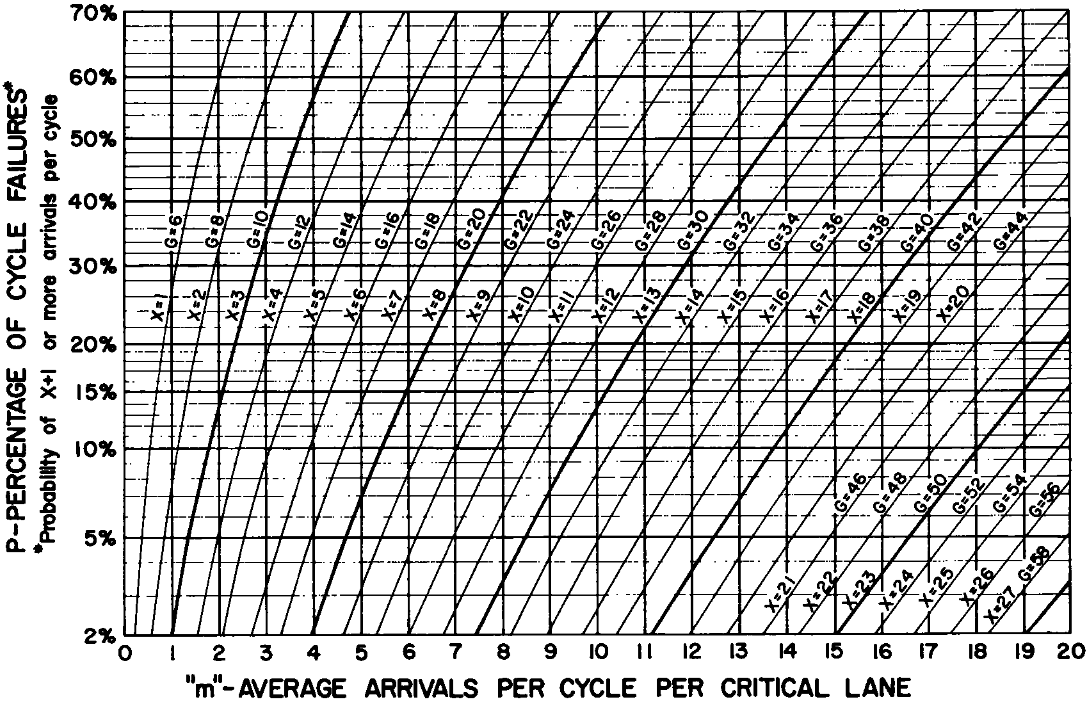


Figure 8. Green requirements for non-conflicting movements.

The versatility of the procedure is emphasized in the many phasing combinations available. The proximity of another intersection or a ramp might dictate favoring one phase at the expense of the others. Therefore, it would be possible to prevent excessive queues leading to interference on adjacent facilities and perhaps progressive failures. It is at this point in the procedure that the engineer's judgment must be used.

It should be remembered that the percentage of failures (P) is based on the number of cycles during the peak period, not the number during the peak hour. Thus, assuming a duration of 25 min for the PM peak period (consistent with the sample analyzed earlier in this report), the number of failures and total failure time may be calculated for designs B and C (Table 8).

TABLE 8
PROPERTIES OF DESIGNS B AND C

Property	Design B	Design C
Length of cycle	100	60
No. of cycles in peak period	15	25
Percentage of failures (%)	43	40
No. of cycle failures	6.5	10
Total "failure time" (min)	10.7	10.0

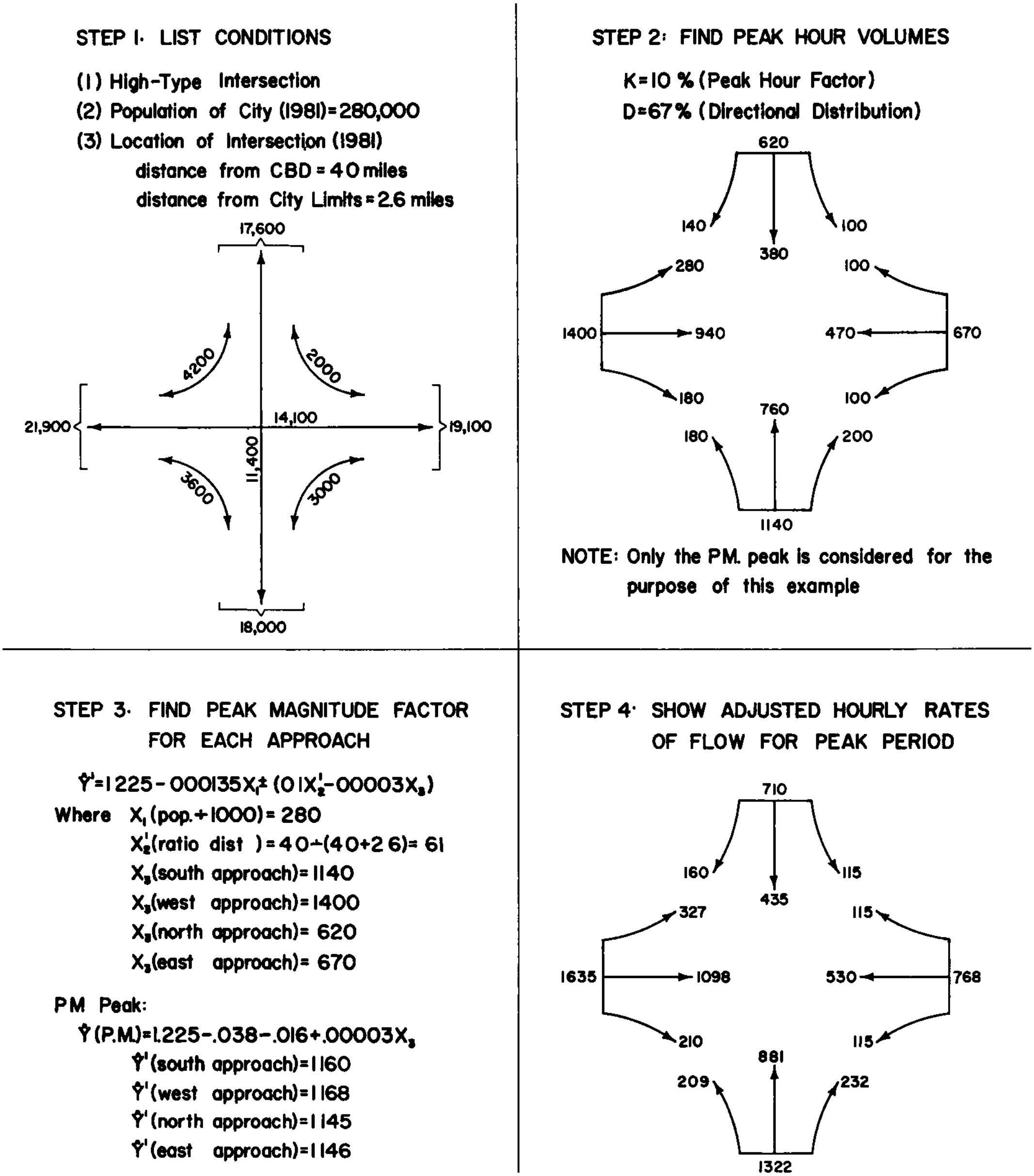
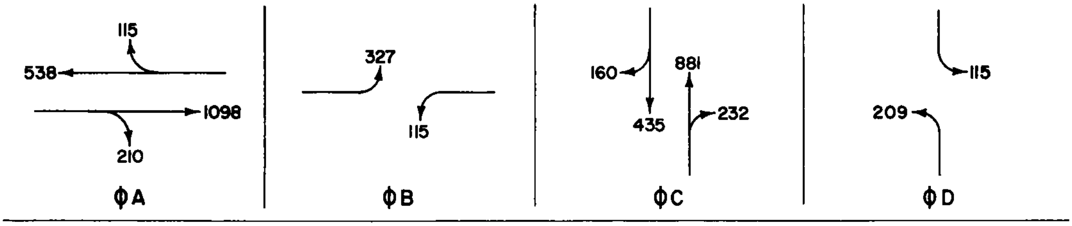


Figure 9. Capacity-design procedure (steps 1 to 4).

Although additional research is needed in deciding just what percentage of failures may reasonably be allowed, it seems that a level of 30 to 35 percent during the peak period represents a practical design level (remembering that this would be only about 10 to 15 percent of the peak hour). Step 7 for designs B and C could be repeated assuming longer cycle lengths (say 120 and 80 sec, respectively) to obtain a lower level of failure. The excessive cycle length of 120 sec required for design B might preclude its use. However, because the conditions used in the calculations are for projected volume data, design alternative B might possibly offer 15 years of desirable operation and thus still merit consideration.

STEP 5 ASSUME PHASING



STEP 6: ASSUME VARIOUS LANE COMBINATIONS. DETERMINE CRITICAL LANE VOLUMES (V) AND MINIMUM CYCLE LENGTHS (C).

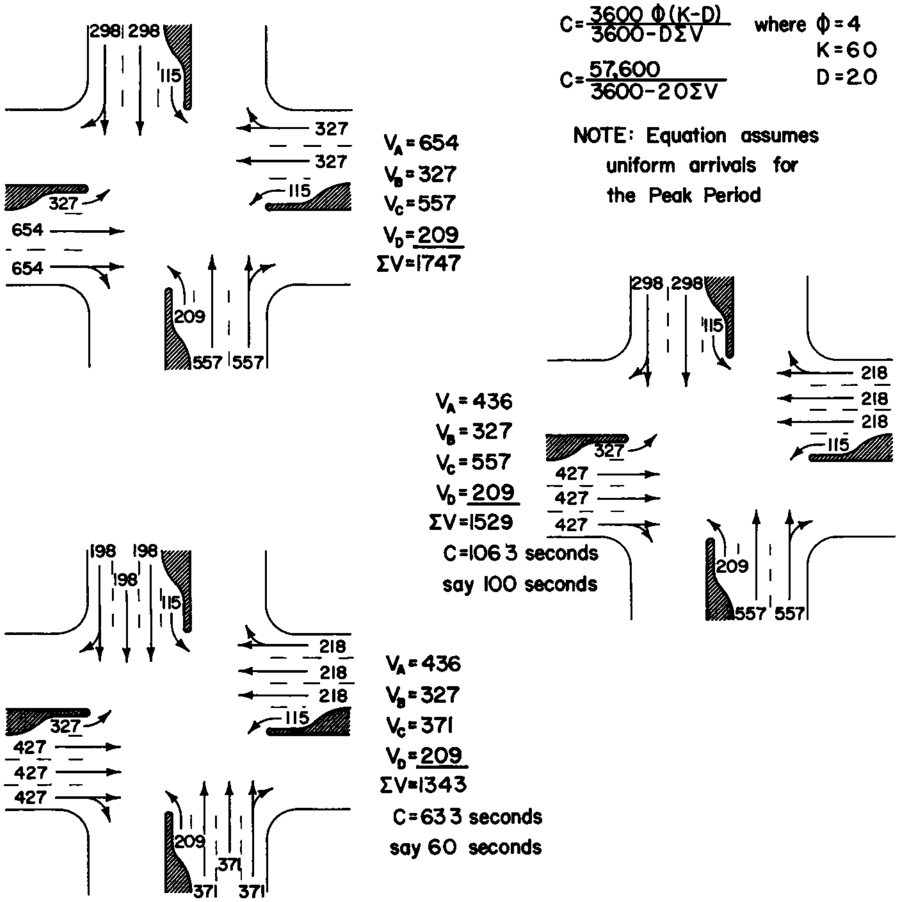


Figure 10. Capacity-design procedure (steps 5 and 6).

DESIGN ALTERNATIVE "B"
(Assuming Cycle Length Equals 100 Seconds)

Phase ϕ	Avg. Arrivals per Cycle* m	Phase Lengths (G) for Various Percentages of Failure (P) (Using the Design Chart for Poisson Arrivals during the Peak Period - Figure 8). 2%	5%	10%	20%	30%	40%	50%	60%	70%
A	11.9	42	39	36	33	30	28	26	25	23
B	9.1	34	32	29	26	24	22	21	19	18
C	15.5	52	48	44	40	38	36	34	32	30
D	5.8	25	23	21	19	17	16	14	13	12

$C = \Sigma G =$ Between 102 & 95

Note: Interpolating between 102 and 95 for C = 100, it is seen that P = 43%. However, since P need not be the same for all phases, there can be an infinite combination of phase lengths as long as their summation equals the assumed cycle length.

DESIGN ALTERNATIVE "C"
(Assuming Cycle Length Equals 60 Seconds)

Phase ϕ	Avg. Arrivals per Cycle* m	Phase Lengths (G) for Various Percentages of Failure (P) (Using the Design Chart for Poisson Arrivals during the Peak Period-Figure 8). 2%	5%	10%	20%	30%	40%	50%	60%	70%
A	7.2	29	26	24	21	20	18	17	15	14
B	5.5	25	22	20	18	16	15	14	13	11
C	6.2	26	24	22	19	18	16	15	14	13
D	3.5	19	17	15	13	12	11	10	9	8

$C = \Sigma G = 60$

*m = V ÷ (3600/C)

where m = avg. arrivals per cycle per critical lane.
V = hourly rate of arrivals per peak period.
(3600/C) = number of cycles per hour.

Figure 11. Capacity-design procedure (step 7).

SUMMARY

Conclusions

It is apparent that a problem exists in deciding what volumes should be used to apportion phases at an existing installation or to determine the number of lanes at a proposed intersection facility. The design hourly volumes obtained from origin-destination survey assignments are average hourly volumes. Use of the average hourly volume as a design basis may render the facility underdesigned for the entire peak period (25 to 30 min) within the design hour. In many locations this represents an intolerable situation. Furthermore, because there are 2 peaks each day (morning and evening), 10 each week, and 520 a year, an intersection could very conceivably be underdesigned well over 200 hours a year. Thus, the 30th highest hour, which is based on average hourly rates, definitely does not mean that the facility is underdesigned only 29 hours a year and, such being the case, is not only impertinent but misleading.

Specifically, the following may be concluded from this research with regard to traffic demand on high-type urban signalized intersections:

1. Peak periods were found to exist within the peak hours. These peak periods may be characterized by two quantitative properties—their duration in minutes and their magnitude expressed as the ratio of average peak-period arrivals to average peak-hour arrivals.
2. The magnitude of the peak period may be approximated by the following expression:

$$Y' = 1.225 - 0.000135X_1 \pm (0.1X_2' - 0.00003X_3)$$

in which the factors within the parentheses are to be added for the AM peak and subtracted for the PM peak; X_1 = population of city ÷ 1,000; X'_2 = location of the intersection as a fraction of distance from CBD to city limits; and X_3 = peak hourly volume (PHV) for the approach.

3. The duration of the peak period was not significantly related to the variables of population, location, and volume. Therefore, the mean for each period AM and PM represented the best available estimate. These values were 26.69 min for the AM peak based on 28 approaches and 25.04 min for the PM peak based on 32 approaches.
4. χ^2 tests made on arrivals during the AM and PM peak hours showed that the assumption of a Poisson distribution of arrivals for the peak hour was not valid. However, Poisson arrivals were verified for the duration of the peak period.

With respect to the application of these findings, additional conclusions are evident:

5. The peak hour is not the logical interval of time for a capacity analysis and design procedure because it contains two distinct populations, one of which is the peak period.
6. If hourly volumes are to be used, they must be expanded to accommodate the peak period. The peak magnitude factor should be used for this conversion.
7. There are four possible assumptions regarding demand which may be utilized in the determination of cycle length and phase lengths (Figs. 5 and 6). The most realistic assumption is that of Poisson arrivals during the peak period; the least realistic is uniform arrivals for the duration of the peak hour. The remaining two (Poisson arrivals for the peak hour and uniform arrivals for the peak period) are comparable, although the latter is much simpler to use.
8. A design procedure is suggested where the determination of the number of approach lanes is based on the expansion of hourly volumes by the peak magnitude factor to accommodate the average peak rate. Timing of the signal system is accomplished by a method of successive approximations in the application of the Poisson distribution to the peak period, facilitated either by graphic techniques (Fig. 8) or through programming the solution on a high-speed digital computer.
9. The peak magnitude factor is more critical than the peak duration factor. The former provides the conversion from the peak-hour average arrivals to the peak-period average arrivals. The latter merely fixes the number of failing cycles after the per-

centage of failures is computed. For an intersection with actuated equipment, the overlapping of peaks from the various phases becomes the basis for calculating the number of failing cycles.

Recommendations

It has been shown that design procedures based on assumptions regarding uniform peak hourly demand are inefficient. In short, the hour as a basis for traffic capacity design has outlived its usefulness. The advantages in relating design criteria to statistical distributions are well established in traffic engineering. Speeds are fitted to the normal distribution, and gaps to the exponential. Thus, events may be predicted within specified confidence limits. Because of the equivalency of its parameters (mean and variance) the Poisson distribution is especially powerful. However, the recommendation is that it be used with the appropriate duration of time; namely, the peak period.

The design procedures explained in the previous chapter represent attempts to place a capacity analysis on a rational basis through an appreciation of the underlying assumptions and limitations regarding arrivals. The percentage of cycle failures and "failure time" concepts suggests a quantitative basis of comparison, or figures of merit. However, it is conceded that stronger bases are needed. It is urged that future studies be devoted to the development of a relationship between percentage of failure and both queue lengths and delays. Delay is important because a motorist is not likely to be impressed by the reduction in cycle failures accomplished at the expense of very long cycles. Thus, the remaining time in the peak hour, after the peak period has been evaluated, could be subjected to an analysis of delay as a basis for modifying the cycle length. This is especially true for fixed-time equipment. On the other hand, it can be argued that future studies along these lines be slanted toward the idea of "space" rather than "time" as the governing factor. The length of queue may provide a better indication of failure than delay. A maximum delay of 120 sec on an approach may have less significance than a maximum queue length of 15 vehicles, especially if the geometrics show that this will obstruct operation at an adjacent site causing progressive breakdown.

A cycle failure as defined in this report does not take into account the effect of a failure on the next cycle. Greenshields (13) suggests a method for checking this which is perhaps too tedious to be applied generally. The effect of this secondary aspect of failure on delay and queue length should still be considered.

Eventually, as design procedures are improved, origin and destination methods of assignment must be reappraised. It is evident that much is to be gained by assigning hourly volumes directly instead of applying peak-hour and directional distribution factors to ADT assignments. Logically, assignments should be made on the basis of the peak period.

Finally, although the discussion has been devoted to urban signalized intersections, the relevance to other type installations is apparent. Many of the concepts developed would have direct application to freeway and ramp capacity analyses. Similar investigations in these areas should be undertaken.

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Appendix A

DESCRIPTION OF STUDY PROCEDURE

Any signalized intersection can be included in the sample within the following limitations:

1. There must be a definite peak period within the hour. May low-volume intersections exhibit only random fluctuations throughout the peak hour. An intersection leg with an ADT of 9,000 seems to represent the minimum. Thus, allowing 67 percent for directional distribution and, say, 11 percent for the K-ratio of DHV to ADT, this gives an hourly volume of $9,000 \times 0.67 \times 0.11 = 660$ vehicles per peak hour on the intersection approach. Thus, approaches with less than 660 vehicles per peak hour should be excluded.
2. The intersection approach measured should be far enough "downstream" from another signal so that "demand" is measured, not just the volume that passes through the previous signal. This will usually preclude the use of intersections in the CBD and other locations where demand so exceeds capacity that traffic in the peak period is backed up to a previous signal.
3. The same intersection need not be used for AM and PM studies; thus, one-way streets may be included as an approach.

Five-minute manual volume counts are needed:

1. The count should be conducted on the two approaches to an intersection with the most pronounced peaks, usually toward the CBD in the AM peak and away from the CBD in PM peak.
2. Counts should be made for at least 16 consecutive 5-min periods, so as to bracket the peak hour.
3. The 5-min time intervals should be controlled as accurately as possible.
4. Lane distributions, traffic composition, and turning movements need not be considered.
5. The location of the intersection may be described to the nearest one-tenth of a mile, either by scaling from a map or by driving the route.
6. It is important that "demand" on the intersection be measured, not "capacity." Therefore, vehicles should be counted before their speed is greatly reduced by either the signal or vehicles waiting at the signal. Thus, as traffic increases during the counting period it may be necessary to move farther from the intersection.

In addition to this information, data forms (Fig. 12) were sent to the city traffic department where the desired information pertinent to the study was to be recorded.

CITY: Ft Worth, Texas

POPULATION (CITY): 356,268

POPULATION (AREA): 573,215

DATE OF STUDY: 4-3-61

A.M. OR P.M. STUDY: A M

APPROACH	STREET NAME	ARTERIAL OR FEEDER	ADT (16 hour count)	1-WAY OR 2-WAY	SPEED LIMIT	NO. OF LANES AT MID-BLOCK	DIST. FROM CITY-LIMITS TO INTERS'CT'N	DIST. FROM CBD TO INTERSECTION
#1 (EAST)	Jacksboro Hwy (S H 199)	Arterial	18,374	2-way	40 MPH	2 lanes in each direction	5 5 mi	3 4 mi
#2 (NORTH)	Ephriam Ave (S H 183)	Arterial	19,189	2-way	45 MPH	2 lanes in each direction	1 1 mi	1 1 mi

5-MINUTE ARRIVALS			
TIME BEGIN. AT		APPROACH #1	APPROACH #2
6:45 A M or	4:30 P.M.	51	62
6:50	4:35	64	72
6:55	4:40	64	67
7:00	4:45	66	84
7:05	4:50	60	103
7:10	4:55	80	90
7:15	5:00	84	104
7:20	5:05	86	100
7:25	5:10	88	115
7:30	5:15	98	122
7:35	5:20	59	117
7:40	5:25	58	113
7:45	5:30	65	96
7:50	5:35	48	71
7:55	5:40	50	58
8:00	5:45	39	67
8:05	5:50	49	62
8:10	5:55	45	46
8:15	6:00	40	53

REMARKS:

Figure 12. Data form for intersection study.

Appendix B

SUMMARY OF FIVE-MINUTE VOLUME COUNT STUDIES
Amarillo

A.M. APPROACH NUMBER 1	2	3	4		P.M. APPROACH NUMBER 1	2	3	4
West 10th at Georgia	Georgia at West 10th	Frontal Road at Washington	Washington at Frontal Road	Name of Street and Location	West 10th at Georgia	Georgia at West 10th	N.E. 8th at Grand	Grand at N.E. 8th
7,300	7,200	3,800	3,100	ADT (one way)	6,900	9,000	22,000 (2 way)	11,000 (2 way)
35	35	35	30	Speed Limit	35	35	35	40
2	2	2	2	No. Lanes at Mid-Block	2	2	3	2
5 Minute Arrivals				Time A.M. / P.M.	5 Minute Arrivals			
11	18	18	14	6:45 / 4:30	24	77	100**	29
13	9	22	16	6:50 / 4:35	27	76*	118	34
16	14	26	18	6:55 / 4:40	42	67	105	32
23	15	28	12	7:00 / 4:45	29	76	90	43*
31	18	20	31	7:05 / 4:50	47*	91**	102**	31
34	21	11	27*	7:10 / 4:55	42	97	63**	33
37*	30*	36*	29	7:15 / 5:00	59	121	67	41**
54	36	28	27	7:20 / 5:05	42**	121	92	48
59	40	25	40	7:25 / 5:10	73	119	81	54
63**	51**	39**	45**	7:30 / 5:15	76	95**	59	48**
78**	55	64	58	7:35 / 5:20	62	77	66	32
88	65	70	75	7:40 / 5:25	50**	80	53*	33
105	82	71	78	7:45 / 5:30	45	81*	60	45
104	63	62	75	7:50 / 5:35	48	64	47	44
63	59	44**	48**	7:55 / 5:40	50	76	49	37*
50	50**	33	30	8:00 / 5:45	56*	81	62	31
71	53	37	33*	8:05 / 5:50	44	76	50	34
78*	48*	42*	22	8:10 / 5:55	25	67	52	33

* Identifies the beginning and ending of the peak hour.
** Identifies the beginning and ending of the peak period.

SUMMARY OF FIVE-MINUTE VOLUME COUNT STUDIES

San Antonio

A.M. APPROACH NUMBER						P.M. APPROACH NUMBER			
5	6	7	8			5	6	7	18
McMullen at Commerce	Broadway at Hildebrand	Buena Vista at Zarzamora	San Pedro at Cypress	Name of Street and Location		McMullen at Commerce	Broadway at Hildebrand	Commerce at Zarzamora	Soledad at Martin
9,500	14,300	11,300	10,000	ADT (one way)		8,400	19,300	15,100	10,700
-	-	-	-	Speed Limit MPH		-	-	-	-
3	3	3	2	No. Lanes at Mid-Block		3	3	3	3
5 Minute Arrivals				Time		5 Minute Arrivals			
				A.M.	P.M.				
127	48	63	50	6:45	4:10	106*	115	107	93
170	68	58	43	6:50	4:15	70	132	108	70
157*	63	75	35	6:55	4:20	115	141	90	90
192	64	69	50	7:00	4:25	158**	125	110	92*
197	112	86*	52	7:05	4:30	175	159	113	100
191	125	82	66	7:10	4:35	173**	114	75	120
220	106	91	81*	7:15	4:40	137	133	123	105
218	142*	78	83	7:20	4:45	143**	142*	109	100
258**	121	98	102**	7:25	4:50	191	139	127	109
233	160**	82	109	7:30	4:55	178	142	132*	90
306	174	105**	132	7:35	5:00	162	147	121	113**
221	160	133	109	7:40	5:05	170**	130	135**	112
307	171	97	105	7:45	5:10	67	150**	138	124
310**	162	106	97**	7:50	5:15	88	206	167	120
142	136**	107**	93	7:55	5:20	66	192	140	107**
97	124	85*	83	8:00	5:25	75	175	138**	85
27	135	78	95	8:05	5:30	55	183	122	92
42	162	75	86*	8:10	5:35	63	181	121	89
30	132*	82	74	8:15	5:40	52	190	120	73
28	118	88	77	8:20	5:45	30	176	127	65
27	102	60	70	8:25	5:50	37	175**	134*	72

* Identifies the beginning and ending of the peak hour

** Identifies the beginning and ending of the peak period.

SUMMARY OF FIVE-MINUTE VOLUME COUNT STUDIES
Austin

P. M. APPROACH NUMBER 9	10	11	12		A. M. APPROACH NUMBER 9	10	11	12
South Lamar at Barton Springs	Barton Springs at South Lamar	Expressway at Airport Blvd.	Airport Blvd. at Expressway	Name of Street and Location	Airport Blvd. at Expressway	Expressway at Airport Blvd.	North Lamar at Enfield Road	Enfield Road at North Lamar
15,894	12,057	32,097	12,182	ADT (two way)	13,010	29,171	15,998	9,062
30	30	45	35	Speed Limit MPH	35	45	30	30
2	2	4	2	No. Lanes at Mid-Block	2	4	2	1
5 Minute Arrivals				Time (A.M. / P.M.)	5 Minute Arrivals			
62	38	106	63*	4:30 / 6:45	37	65	29	8
73	46	116	61**	4:35 / 6:50	46	60	34	15
77	46	128	71	4:40 / 6:55	41	78	37	15
71	57	146	64**	4:45 / 7:00	59*	94	35	16
51	56*	161*	56	4:50 / 7:05	61	100	55	32
77*	46	177	60	4:55 / 7:10	73	140*	66	21
85	58**	162**	62**	5:00 / 7:15	41	140	91*	37
98**	80	250	67	5:05 / 7:20	60	150	88	40*
129	79	292	71	5:10 / 7:25	59	186**	97**	58**
120	84	262	68**	5:15 / 7:30	70**	206	127	68
104**	73	225	50	5:20 / 7:35	76	217	118	65
72	77	185**	49*	5:25 / 7:40	79	270	112	77
115	69**	180	45	5:30 / 7:45	69	221	111	77
78	57	158	46	5:35 / 7:50	71	210	126	57**
105	60	148	43	5:40 / 7:55	61**	154**	108	63
79	60*	163*	46	5:45 / 8:00	43	160	123**	46
93*	45	145	45	5:50 / 8:05	49	117*	88	56
69	51	117	42	5:55 / 8:10	40	122	98*	60
65	38	102	26	6:00 / 8:15	44	110	74	50*

* Identifies the beginning and ending of the peak hour.
** Identifies the beginning and ending of the peak period.

SUMMARY OF FIVE-MINUTE VOLUME COUNT STUDIES

Houston

A.M. APPROACH NUMBER 13	14	15	16	Name of Street and Location		P.M. APPROACH NUMBER 13	14	15	16
Heights at Sixth	Heights at Eleventh	Yale at Sixth	Houston at Washington			Memorial at Birdsall	Heights at Washington	Yale at Sixth	Heights at Twentieth
-	-	-	-	ADT (one way)		-	-	-	-
35	35	35	35	Speed Limit MPH		45	35	35	35
2	2	2	2	No. Lanes at Mid-Block		3	3	2	2
5 Minute Arrivals				Time		5 Minute Arrivals			
				A.M.	P.M.				
	31			6:45	4:30	166	176*		91
	54		92	6:50	4:35	174	177	60	71
	47		101	6:55	4:40	222	209**	77*	87
	77*		126*	7:00	4:45	217**	210	65	90*
77*	81	126**	117	7:05	4:50	246	205	82**	72
96**	89	152	117	7:10	4:55	248	202	100	93
94	93**	163	136	7:15	5:00	241	205	87	84
113	92	156	134**	7:20	5:05	249	200	79	89
104	116	128	168	7:25	5:10	258	206	81	84
120	101	143	167	7:30	5:15	208**	220	98**	96
108	115	148	165	7:35	5:20	218	183**	49	71
106**	115	132**	142**	7:40	5:25	189	190*	82	91
71	82**	118	126	7:45	5:30	255	175	77	97
90	76	99	136	7:50	5:35	218*	158	70*	102
76	63*	110	131*	7:55	5:40	192	167		90*
59*	65	106*	111	8:00	5:45		176		83
68	59		104	8:05	5:50		143		
			98	8:10	5:55		139		
			101	8:15	6:00				

* Identifies the beginning and ending of the peak hour.

** Identifies the beginning and ending of the peak period.

SUMMARY OF FIVE-MINUTE VOLUME COUNT STUDIES

Waco

A.M. APPROACH NUMBERS						P.M. APPROACH NUMBERS			
17	18	19	20			17	18	19	20
Waco Drive at Twenty-Sixth	Twenty-Sixth at Waco Drive	Franklin at Twenty-Sixth	Twenty-Sixth at Franklin	Name of Street and Location		Waco Dr. (Eastbd) at Eighteenth	Waco Dr. (Westbd) at Eighteenth	Eighteenth at Waco Drive	Twenty-fifth at Bosque
16,000	9,000 (1 way)	10,000	5,000 (1 way)	ADT (two way)		16,000	16,000	13,500	7,000 (1 way)
-	-	-	-	Speed Limit MPH		-	-	-	-
2	3	2	3	No. Lanes at Mid-Block		3	3	2	3
5 Minute Arrivals				Time		5 Minute Arrivals			
				A.M.	P.M.				
46	39			7:00	4:30	42*	73*	54*	66
45	28			7:05	4:35	68**	77	48	57
43	45			7:10	4:40	59	80	42	53
51	49			7:15	4:45	75	95**	40	67*
58	50			7:20	4:50	60**	110	54	69
74	71*	43	44	7:25	4:55	50	100	51	75
87**	78	60*	58*	7:30	5:00	53	109	41**	52
89**	91**	60**	69**	7:35	5:05	31	96	105	81**
113	133	97	76	7:40	5:10	51	110	79	81
100	97	79	96	7:45	5:15	42	115	92	115
98	135	92	98	7:50	5:20	56	78**	57**	87
102	113	83	78	7:55	5:25	42*	77*	56*	79**
102	73**	81	64**	8:00	5:30	34	70	41	70
80**	69	78**	66	8:05	5:35	36	78	56	75
66	61	64	46	8:10	5:40	32	78	40	80*
66	67	61	53	8:15	5:45	37	64	46	62
78	77*	72	65	8:20	5:50	38	56	43	
82*	53	70*	52*	8:25	5:55	31	49	24	
53				8:30	6:00				

* Identifies the beginning and ending of the peak hour.

** Identifies the beginning and ending of the peak period.

SUMMARY OF FIVE-MINUTE VOLUME COUNT STUDIES

Dallas

A.M. APPROACH NUMBER 21	22	23	24			P.M. APPROACH NUMBER 21	22	23	24
Inwood at Lemmon	Lemmon at Inwood	Garland at Buckner	Buckner at Garland	Name of Street and Location		Second at Hatcher	Hatcher at Second	Garland at Buckner	Buckner at Garland
12,570	11,815	14,900	10,935	ADT (one way)		7,981	12,466	16,430	10,785
35	35	35	35	Speed Limit MPH		30	35	35	35
3	3	3	3	No. Lanes at Mid-Block		2	3	3	3
5 Minute Volumes				Time		5 Minute Volumes			
				A.M.	P.M.				
50	45	141	104*	6:45	4:30	42	147	100	68
61	43	145	90	6:50	4:35	54	216*	127	81*
54	41	147	111	6:55	4:40	91*	176	170	125**
62	39	148	88**	7:00	4:45	70	205	160	100
86	49	171*	108	7:05	4:50	74	174	167	102
64	56	206**	116	7:10	4:55	71	178	167	99**
107*	59	269	100**	7:15	5:00	80	209**	188**	98**
111	68*	232**	87	7:20	5:05	71	200	203**	103
114**	87	155	85	7:25	5:10	68**	224	247	112
121	70	187**	105	7:30	5:15	87	267	243	118
141	116**	242	93	7:35	5:20	97	157**	216	84**
146	101	207**	85*	7:40	5:25	78**	210	267	82
118	139	176	94	7:45	5:30	59	171*	179	91*
127**	103	178	72	7:50	5:35	61*	201	221**	54
101	131	202	86	7:55	5:40	59	138	191	132
128	116	152*	84	8:00	5:45	53	140	224	71
92	149	138	86	8:05	5:50	54	152	197	104
84*	84**	162	71	8:10	5:55	51	96	185*	91
102	96*	160	84	8:15	6:00	48	137	157	101

* Identifies the beginning and ending of the peak hour.
** Identifies the beginning and ending of the peak period.

SUMMARY OF FIVE-MINUTE VOLUME COUNT STUDIES

Fort Worth

A. M. Approach Number 25	26	27	28		P. M. Approach Number 25	26	27	28
Jacksboro Hwy at S. H. 199	Ephriham at Hwy. 183	Berry at Hemphill	Hemphill at Berry	Name of Street and Location	Jacksboro Hwy. at S.H. 199	Ephriham at S.H. 183	Berry at Hemphill	Hemphill at Berry
19,189	18,374	15,768	13,507	ADT (two way)	19,189	18,374	15,768	13,507
16 Hour-Counts					16 Hour-Counts			
40	45	30	30	Speed Limits MPH	40	45	30	30
2	2	2	2	No. Lanes at Mid-Block	2	2	2	2
5 Minute Arrivals				Time A.M. P.M.	5 Minute Arrivals			
51	62	42	26	6:45 4:30	88	98*	56*	35
64*	72*	30	24	6:50 4:35	65	113**	66	50**
64	67	32	30	6:55 4:40	153**	119	83	78
66	84	21	34	7:00 4:45	143	118	57**	68**
60	103	50	56*	7:05 4:50	139*	103**	42	43
80**	90	40	55**	7:10 4:55	108	95**	41	48
84	104**	49*	73	7:15 5:00	120	121	64**	59**
86	100	46	61	7:20 5:05	110	121	76	70
88	115	55	64	7:25 5:10	120	97**	67	75
98	122	59**	70	7:30 5:15	140**	122	62**	81
59**	117	76	61	7:35 5:20	159	79	49	74**
58	113	59	82	7:40 5:25	147	80*	45*	51
65*	96**	79	51**	7:45 5:30	146**	73	46	66
48	71	87	46	7:50 5:35	106*	82	58	48
50	58	63**	41	7:55 5:40	109	61	51	49
39	67	53	45*	8:00 5:45	107	72	48	43
49	62	47	46	8:05 5:50	105	70	43	50
45	46	45*	40	8:10 5:55	87	69	50	46
40	53	42	43	8:15 6:00	92	73	40	50

* Identifies the beginning and ending of the peak hour.
**Identifies the beginning and ending of the peak period.

SUMMARY OF FIVE-MINUTE VOLUME COUNT STUDIES

Corpus Christi

A. M. Studies were not available from Corpus Christi

	P.M. APPROACH NUMBER			
	29	30	31	32
Name of Street and Location	Sante Fe at Booty-Ayres	Staples at Baldwin	Baldwin at Staples	Alameda at Doddridge
ADT (two way)	8,332	14,902	11,370	9,134
Speed Limit MPH	30	30	30	30
No. Lanes at Mid-Block	2	2	2	2
Time P.M.	5 Minute Arrivals			
4:30		61	29	41
4:35	35	54	40	52
4:40	59**	58	51	55
4:45	66	59	39	57
4:50	48	79	67*	67
4:55	47	72	57	56
5:00	57	73*	32	76*
5:05	63**	66	60**	60
5:10	77	102**	62	78**
5:15	86	117	79	94
5:20	64	122	85	88
5:25	60**	102	62**	83
5:30	61	92**	54	67**
5:35	55*	83	39	79
5:40	49	95	55	63
5:45	44	72	64*	66
5:50	36	76	26	61
5:55	44	75*	41	78*
6:00	26	60	53	55

* Identifies the beginning and ending of the peak hour.

** Identifies the beginning and ending of the peak period.

CALCULATION OF ELEMENTS OF INVERSE MATRICES - P.M. PEAK ($X_1X_2X_3$)

	Sums of Squares and Products			Solution		
	X_1	X_2	X_3	1	2	3
X_1	2,688,222.	7,891.95	3,322,686			
X_2	7,891.95	72.9788	13,772.71			
X_3	3,322,686.	13,772.71	12,412,673.			
(1) (1/2,688,222.)	1	.00293575	1.2360164	1	0	0
(2) (1/72.9788)	108.140309	1	188.722067	0	1	0
(3) (1/12,412,673.)	.2676850	.00110957	1	0	0	1
(4) (1/1.2360164)	.8090508	.00237517	1	.8090508	0	0
(5) (1/188.722067)	.5730136	.00529880	1	0	.00529880	0
(6)	.2676850	.00110957	1	0	0	1
(7) - (9)	.5413658	.00126560		.8090508	0	-1
(8) - (9)	.3053286	.00418923		0	.00529800	-1
(10) (1/.00126560)	427.754235	1		639.262599	0	-790.139060
(11) (1/.00418923)	72.884181	1		0	1.2648625	-238.707351
(12) - (13)	354.870054			639.262599	-1.2648625	-551.431711
				1.801399	-.0035644	- 1.553897
(14) (1/354.870054)				-131.2938	1.524687	-125.4530
			Inverse Matrix:	- .336528	- .0007376	1.555155
$(C_{11})^{1/2}$	.0008186		1	.000000670	.00004884	-.000000125
$(C_{22})^{1/2}$	.14454	Decoded:	2	-.000048840	.02089219	-.00001011
$(C_{33})^{1/2}$	.0003540		3	-.000000125	.00001011	.000000125
			Cij	1	2	3

CALCULATION OF ELEMENTS OF INVERSE MATRICES - A.M. PEAK ($X_1X_2X_3$)

	Sums of Squares and Products			Solution		
	X_1	X_2	X_3	1	2	3
X_1	2,458,344.	6,079.70	2,035,655.			
X_2	6079.70	65.2900	9,471.20			
X_3	2,035,655.	9,471.20	8,219,093.			
(1) (1/2,458,334)	1	.00247310	.8280628	1	0	0
(2) (1/65.2900)	93.118395	1	145.063563	0	1	1
(3) (1/8,219,093)	.2476739	.00115234	1	0	0	1
(4) (1/.8280628)	1.2076379	.00298661	1	1.2076379	0	0
(5) (1/145.063563)	.6419144	.00689353	1	0	.00689353	0
(6)	.2476739	.00115234	1	0	0	1
(7) - (9)	.9599640	.00183427		1.2076379	0	-1
(8) - (9)	.3942405	.00574119		0	.00689353	-1
(10)(1/.00183427)	523.24934	1		658.37521	0	-545.17601
(11)(1/.00574119)	68.66259	1		0	1.2008145	-174.17992
(12) - (13)	454.68675			658.37521	-1.2007145	-370.99609
(14)(1/454.68675)				1.44975	- .0026408	- .0815938
			Inverse Matrix:	-99.42155	1.3820609	-118.15540
				- .244058	-.0009385	1.338242
$(C_{11})^{1/2} = .000767$			1	.000000589	-.00004044	-.000000099
$(C_{22})^{1/2} = .1455$		Decoded:	2	.000040440	.02116800	-.000014370
$(C_{33})^{1/2} = .000404$			3	.000000099	-.00001437	.000000163
			Cij	1	2	3

CALCULATION OF ELEMENTS OF INVERSE MATRICES - P.M. PEAK ($X_1X_2'X_3$)

	Sums of Squares and Products			Solution		
	X_1	X_2'	X_3	1	2	3
X_1	2,688,222.	556.625	3,322,686.5			
X_2'	556.625	1.093	1,157.790			
X_3	3,322,686.5	1,157.790	12,412,673.			
(1) (1/2, 688,222.)	1	0.00020706	1.2360164	1	0	0
(2) (1/1.093)	509.26344	1	1,059.27722	0	1	0
(3) (1/12,412,673.)	.1676850	0.00009367	1	0	0	1
(4) (1/1.2360164)	0.8090508	0.00016752	1	0.8090508	0	0
(5) (1/1,059.27722)	0.480765	0.0009440	1	0	0.0009440	0
(6) (1/1)	0.2676850	0.00009367	1	0	0	1
(7) - (9)	0.5413658	0.00007385		0.8090508	0	-1
(8) - (9)	0.2130800	0.00085033		0	0.0009440	-1
(10) (1/.00007385)	7,330.6134	1		10,955.3257	0	-13,540.9614
(11) (1/.00085033)	250.5851	1		0	1.11016	- 1,176.0140
(12) - (13)	7,080.0283			10,955.3257	-1.11016	-12,364.9474
(14) (1/7,080.0283)				1.547356	-.00015680	-.174645
		Inverse Matrix:		-387.7429	1.14844	-738.4116
				- .37788	-.0000657	1.5367
$(C_{11})^{1/2}$ = .000759			1	.000000576	-.000143458	-.00000141
$(C_{22})^{1/2}$ = 1.026		Decoded:	2	.000144238	1.05163769	-.000059489
$(C_{33})^{1/2}$ = .000352			3	.000000141	-.00006011	.000000124
			Cij	1	2	3

CALCULATION OF ELEMENTS OF INVERSE MATRICES - A.M. PEAK ($X_1X_2{}'X_3$)

	Sums of Squares and Products			Solution		
	X_1	$X_2{}'$	X_3	1	2	3
X_1	2,458,334.	231.931	2,035,655.			
$X_2{}'$	231.931	0.8021	461.750			
X_3	2,035,655.	461.750	8,219,093.			
(1) (1/2,458,334)	1	0.00009434	.8280628	1	0	0
(2) (1/0.8021)	289.1547	1	575.6763	0	1	0
(3) (1/8,219,093)	.2476739	0.00005618	1	0	0	1
(4) (1/.8280628)	1.20764	0.0001139	1	1.20764	0	0
(5)	0.50229	0.0017371	1	0	0.0017371	0
(6)	0.24767	0.0000562	1	0	0	1
(7) - (9)	0.95997	0.0000577	0	1.20764	0	-1
(8) - (9)	0.35462	0.0016809	0	0	0.0017371	-1
(10)	16,637.2717	1		20,929.6460	0	-17,331.0225
(11)	151.4784	1		0	1.0334	594.9194
(12) - (13)	16,485.2617	0		20,929.6360	-1.0334	-16,736.1031
(14) (1/16,485.7833 = 0.000060)						
			Inverse Matrix:	1.26956	-0.0000627	-1.015840
				-192.366	1.043156	-442.80434
				-.30362	-.00004310	1.276317
$(C_{11})^{1/2}$ = .000718		Decoded:	1	.0000005164	-.00007817	-.00000012352
$(C_{22})^{1/2}$ = 1.140			2	.00007825	1.30053	-.0000001553
$(C_{33})^{1/2}$ = .000394			3			
			Cij	1	2	3

Appendix D

REGRESSION COEFFICIENTS AND TESTS OF SIGNIFICANCE

Magnitude of P.M. Peak ($X_1X_2X_3Y'$)

Independent Variable	X_1	X_2	X_3
$\Sigma xy'$	-338.835	-.7961	-193.636
$b = \Sigma c \Sigma xy'$	-.00016393	.0018735	.00002621
$S_b = S_{Y'1.23}\ C^{1/2}$	.00005239	.0092506	.00002266
$t = b/S_b$	3.129**	0.203	1.157
$t_{.05} = 2.048;\ t_{.01} = 2.763$			

Magnitude of A.M. Peak ($X_1X_2X_3Y'$)

Independent Variable	X_1	X_2	X_3
$\Sigma xy'$	-342.00	-.7639	-236.100
$b = \Sigma c \Sigma xy$	-.000127	.0039	-.000026
$S_b = S_{Y'.123}\ C^{1/2}$	.0000572	.0109	.000030
$t = b/S_b$	2.220*	.358	.867
$t_{.05} = 2.064$			

REGRESSION COEFFICIENTS AND TESTS OF SIGNIFICANCE

Magnitude of P.M. Peak ($X_1X_2'X_3Y$)

Independent Variable	X_1	X_2'	X_3
Σ xy'	-338.835	-.1593	-193.636
$b = \Sigma c \Sigma xy'$	-.000145	-.110	.0000333
$S_b = S_{Y'.123} C^{1/2}$	.00004615	.0624	.0000214
$t = b/S_b$	3.142**	1.763	1.556
$t_{.05} = 2.048$; $t_{.01} = 2.763$			

Magnitude of A.M. Peak ($X_1X_2'X_3Y'$)

Independent Variable	X_1	X_2'	X_3
Σ xy'	-342.00	.10326	-436.10
$b = \Sigma c \Sigma xy'$	-.000125	.093	-.000027
$S_b = S_{Y'.123} C^{1/2}$	.0000524	.0832	.000029
$t = b/S_b$	2.385*	1.118	.931
$t_{.05} = 2.064$			

REGRESSION COEFFICIENTS AND TESTS OF SIGNIFICANCE

Duration of P.M. Peak ($X_1X_2X_3Y$)

Independent Variable	X_1	X_2	X_3
Σxy	19,208.25	92.4813	43,233.59
$b = \Sigma c \, \Sigma xy$	.002942	.5570	.002078
$S_b = S_{Y.123} \; C^{1/2}$	.00534	.942	.00231
$t = b/S_b$	.551	.591	.900
$t_{.05} = 2.048$ (no significant coefficients)			

Duration of A. M. Peak ($X_1X_2X_3Y$)

Independent Variable	X_1	X_2	X_3
Σxy	4,257.80	-32.190	9,441.39
$b = \Sigma c \, \Sigma xy$	.00288	-.990	.00158
$S = S_{Y.123} \; C^{1/2}$	.00379	.719	.00200
$t = b/S_b$	.760	1.377	.790
$t_{.05} = 2.064$ (no significant coefficients)			

REGRESSION COEFFICIENTS AND TESTS OF SIGNIFICANCE

Duration of P.M. Peak ($X_1X_2X_3Y$)

Independent Variable	X_1	X_2	X_3
Σxy	19,208.25	9.3749	43,233.59
$b = \Sigma c \Sigma xy$	.0036	4.516	.0020
$S_b = S_{Y.123} C^{1/2}$	.00494	6.679	.00229
$t = b/S_b$	.729	.676	.873
$t_{.05} = 2.048$ (no significant coefficients)			

Duration of A.M. Peak ($X_1X_2'X_3Y$)

Independent Variable	X_1	X_2	X_3
Σxy	4,257.80	-5.1176	9,441.39
$b = \Sigma c \Sigma xy$	.00143	-7.50	.00122
$S_b = S_{Y.123} C^{1/2}$	.00355	5.632	.00195
$t = b/S_b$	.403	1.333	.626
$t_{.05} = 2.064$ (no significant coefficients)			

Appendix E

ANALYSES OF VARIANCE

Magnitude of P. M. Peak ($X_1X_2X_3Y'$)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Total	31	$\Sigma y'^2 = 0.163678$	
Regression	3	$\Sigma \hat{y}'^2_{123} = 0.048978$	.016326*
Deviations	28	$\Sigma d^2_{Y'.123} = 0.114700$	$S^2_{Y'.123} = .004096$

$F = .016326/.004096 = 3.986 > F_{.05} = 2.95$ (significant); $R^2 = .048978/.163678 = 29.9\%$

Magnitude of A. M. Peak($X_1X_2X_3Y'$)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Total	27	$\Sigma y'^2 = 0.185528$	
Regression	3	$\Sigma \hat{y}'^2_{123} = 0.051960$	.017320*
Deviations	24	$\Sigma d^2_{Y.123} = 0.133568$	$S^2_{Y'.123} = .005565$

$F = .017320/.005565 = 3.11 > F_{.05} = 3.01$ (significant); $R^2 = .051960/.185528 = 28.0\%$

ANALYSES OF VARIANCE

Magnitude of P.M. Peak ($X_1X_2'X_3Y'$)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Total	31	$\Sigma y'^2 = .163678$	
Regression	3	$\Sigma \hat{y}'^2_{123} = .060206$	.020067**
Deviations	28	$\Sigma d^2_{Y.123} = .103472$	$S^2_{Y.123} = .003695$

$F = .02067/.003695 = 5.431 > F_{.01} = 4.57$ (highly significant); $R^2 = .060206/.163678 = 36.8\%$

Magnitude of A. M. Peak ($X_1X_2'X_3Y'$)

Source of Variation	Degrees of Freedom	Sum Squares	Mean Square
Total	27	$\Sigma y'^2 = .185528$	
Regression	3	$\Sigma \hat{y}'^2_{123} = .057555$	.019185*
Deviations	24	$\Sigma d^2_{Y'.123} = .127973$	$S^2_{Y.123} = .005332$

$F = .019185/.005332 = 3.598 > F_{.05} = 3.01$ (significant); $R^2 \doteq .057555/.185538 = 31.0\%$

ANALYSES OF VARIANCE

Duration of P.M. Peak ($X_1X_2X_3Y$)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Total	31	$\Sigma y^2 = 1385.65$	
Regression	3	$\Sigma \hat{y}_{123}^2 = 197.85$	65.95
Deviations	28	$\Sigma d_{Y.123}^2 = 1187.80$	$S_{Y.123}^2 = 42.42$

$F = 65.95/42.42 = 1.56 < F_{.05} = 2.95$ (not significant); $R^2 = 197.85/1385.65 = 14.3\%$

Duration of A.M. Peak ($X_1X_2X_3Y$)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Total	27	$\Sigma y^2 = 643.15$	
Regression	3	$\Sigma \hat{y}_{123}^2 = 59.00$	19.67
Deviations	24	$\Sigma d_{Y.123}^2 = 584.15$	$S_{Y.123}^2 = 24.34$

$F = 19.67/24.34 = .808 < F_{.05} = 3.01$ (not significant); $R^2 = 59.00/643.15 = 9.2\%$

ANALYSES OF VARIANCE

Duration of P.M. Peak ($X_1X_2'X_3Y$)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Total	31	$\Sigma y^2 = 1385.65$	
Regression	3	$\Sigma \hat{y}_{123}^2 = 197.96$	65.99
Deviations	28	$\Sigma d_{Y.123}^2 = 1187.69$	$S_{Y.123}^2 = 42.42$

$F = 65199/.4242 = 1.56 < F_{.05} = 2.95$ (not significant); $R^2 = 197.96/1385.65 = 14.3\%$

Duration of A. M. Peak ($X_1X_2'X_3Y$)

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square
Total	27	$\Sigma y^2 = 643.15$	
Regression	3	$\Sigma Y_{123}^2 = 55.99$	18.67
Deviations	24	$\Sigma d_{Y.123}^2 = 587.16$	$S_{Y.123}^2 = 24.47$

$F = 18.67/24.47 = .763 < F_{.05} = 3.01$ (not significant); $R^2 = 55.99/643.15 = 8.7\%$

Appendix F

SUMMARY OF ONE-MINUTE VOLUME COUNT STUDIES (AM PEAK)

Location	Time	7:00	7:10	7:20	7:30	7:40	7:50	8:00	8:10	8:20
26th St. at Waco Drive, Waco	:00	11	11	9	20	26	31	16[a]	10	10
	:01	8	9	4	17	30	29	16	17	15
	:02	4	9	17	11	27	23	17	9	20
	:03	6	7	10	15	16	24	12	11	20
	:04	10	9	10	15	34	28	12	14	12[b]
	:05	3	9	14[b]	19	15	36	15	12	9
	:06	7	11	14	11	23	16	15	14	15
	:07	8	8	15	20[a]	17	22	17	19	11
	:08	6	10	15	23	25	22	14	11	11
	:09	4	11	13	18	17	17	8	11	7
Yale St. at 6th St., Houston	:00		30	32	35	18	16	20	21	
	:01		31	32	36	24	28	14	16	
	:02		31	28	19	35[a]	24	29	21	
	:03		36	32	12	30	14	23	11	
	:04		24	32	41	25	17	20[b]	23	
	:05	20[b]	36	24	39	23	23	12		
	:06	27	37	26	21	34	22	21		
	:07	30	28	33	26	24	23	22		
	:08	21	31	15	27	16	28	16		
	:09	28[a]	31	30	35	21	14	16		
Texas Ave. at F.M. 60, College Station	:00				6[b]	13	24	15	6	14
	:01				7	17	24	10	9	4
	:02				4	21	17	19	5	7
	:03				3	20	13	10[a]	9	7
	:04				8	19	24	10	12	6
	:05				8	17	14	8	6	5
	:06				12	26	16	11	7	5
	:07				7	20	11	7	5	4
	:08				17[a]	21	13	5	4	4
	:09				8	19	16	4	10	6[b]
Heights St. at 6th St., Houston	:00		19	23	23	18	16	9		
	:01		25	14	23	17	17	4		
	:02		21[a]	18	26	17	19	18		
	:03		10	28	18	25[a]	16	14		
	:04		21	30	30	29	22	14[b]		
	:05	8[b]	19	11	27	19	14	13		
	:06	15	17	21	23	15	19	10		
	:07	19	19	28	11	19	7	13		
	:08	22	17	30	21	13	18	16		
	:09	13	22	14	26	5	18	16		

[a] Beginning and ending of peak period.
[b] Beginning and ending of peak hour.

SUMMARY OF ONE-MINUTE VOLUME COUNT STUDIES (PM PEAK)

Location	Time	4:30	4:40	4:50	5:00	5:10	5:20	5:30	5:40	5:50
18th St. at Waco Dr., Waco	:00	17[a]	5	11	5	14	11	6	15	7
	:01	10	8	14	5	18	16[b]	9	6	7
	:02	11	8	13	7	23	10	8	4	16
	:03	8	8	9	8	16	14	4	8	5
	:04	8	13	7	16[b]	8	6	14	7	8
	:05	5	9	13	22	22	15	13	6	4
	:06	9	6	9	21	18	10	5	10	10
	:07	9	6	8	20	15	8	11	13	0
	:08	12	10	6	23	20	8	13	11	6
	:09	13	9	15	19	17	15[a]	14	6	4
Yale St. at 6th St., Houston	:00		26[a]	12	17	21	10	22		
	:01		8	19	16	13	9	16		
	:02		13	18[b]	14	13	11	15		
	:03		12	20	19	20	5	8		
	:04		18	13	21	14	14	16		
	:05	20	15	24	21	21	19	15		
	:06	10	9	19	14	20	20	12		
	:07	15	17	20	10	17	15	16		
	:08	10	16	24	18	18	20	13		
	:09	5	8	13	16	22[b]	13	4[a]		
Texas Ave. at Villa Maria, Bryan	:00	4[a]	10	7	8	15	10			
	:01	12	14	6	10	18	8			
	:02	8	8	13	17	22	9			
	:03	8	10	9	17	23	7			
	:04	8	7	5	10	22	8			
	:05	11	12	11	11	14	9			
	:06	11	12	9	18	12	10			
	:07	12	8	10	21	17	7			
	:08	10	8	14	24	13	8			
	:09	13	11	12[b]	19	14[b]	9[a]			
Memorial at Birdsell, Houston	:00	33	33[a]	48	53	57	38	55	52	
	:01	37	48	49	56	57	47	58	31	
	:02	21	37	42	39	47	40	43	26	
	:03	35	54	53	57	51	57	48	35	
	:04	40	50	54	36	46	36	51	48	
	:05	38	32	29	60	41[b]	35	42		
	:06	41	42	64	50	52	41	54		
	:07	31	50	37	40	31	37	43		
	:08	31	59[b]	58	59	50	46	38		
	:09	33	34	60	40	34	30	41[a]		

[a]Beginning and ending of peak hour.
[b]Beginning and ending of peak period.

CHI-SQUARE TEST FOR THE DISTRIBUTION OF ARRIVALS

Heights at Sixth, Houston, Texas

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Hour 7:05 A.M. to 8:05 A.M. m=18.6; n=60)			
- 13	9	5.9	13.7
14-15	8	7.0	9.1
16-17	7	9.8	5.0
18	6	5.5	6.5
19	8	5.5	11.6
20	0	5.2	0
21-22	7	8.8	5.6
23-24	4	6.0	2.7
$\geq$ 25	11	6.4	18.9
	60	60.1	73.1
$\Sigma (f_x^2/F_x)-n=13.1$			
d.f.=9-2=7; .10>P>.05			
(Peak Period 7:12 A.M. to 7:44 A.M. m=21; n=32)			
$\leq$ 16	5	5.2	4.8
17-19	9	7.1	11.4
20-21	4	5.5	2.9
22-23	5	5.1	4.9
$\geq$ 24	9	9.1	8.9
	32	32.0	32.9
$\Sigma (f_x^2/F_x) -n=0.9$			
d.f. = 5-2=3; P > .70			

Memorial at Birdsall, Houston, Texas

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Hour 4:40 P.M. to 5:40 P.M. m=44; n=60)			
$\leq$ 35	8	5.8	11.0
36-38	7	6.5	7.5
39-40	4	5.9	2.7
41-42	6	6.9	5.2
43-44	2	7.2	.6
45-46	2	6.9	.6
47-48	5	6.1	4.1
49-51	7	6.8	7.2
$\geq$ 52	19	7.8	46.3
	60	59.9	85.2
$\Sigma (f_x^2/F_x) -n=25.2$**			
d.f.=9-2=7; P < .001			
(Peak Period 4:48 P.M. to 5:16 P.M. m=49; n=28)			
$\leq$ 42	9	5.0	16.2
43-46	1	5.3	2
47-50	4	6.3	2.5
51-54	4	5.4	3.0
$\geq$ 55	10	6.0	16.7
	28	28.0	38.6
$\Sigma (f_x^2/F_x)-n=10.6$*			
d.f.=5-2=3; .02>P>.01			

CHI-SQUARE TEST FOR THE DISTRIBUTION OF ARRIVALS

Yale at Sixth, Houston, Texas

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Hour 7:05 A.M. to 8:05 A.M. m=26; n=60			
≤19	10	5.8	17.2
20-21	6	5.6	6.4
22-23	5	7.8	3.2
24-25	6	9.2	3.9
26-27	4	9.2	1.7
28-29	6	7.9	4.6
30-31	8	6.0	10.7
≥32	15	8.5	26.5
	60	60.0	74.2

$\Sigma(f_x^2/F_x)-n = 14.2$*

d.f. =8-2=6; .05 > P > .02

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Period 7:09 A.M. to 7.43 A.M. m=29;n=34)			
≤23	5	5.2	4.8
24-26	5	6.0	4.2
27-28	4	5.0	3.2
29-30	2	5.0	.8
31-33	9	6.1	13.3
≥34	9	6 7	11.9
	34	34.0	38.2

$\Sigma(f_x^2/F_x)-n=4.2$

d.f.=6-2=4; .50>P>.30

Yale at Sixth, Houston, Texas

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Hour 4:40 P.M. to 5:40 P.M. m=16;n=60)			
≤11	9	7.6	10.7
12-13	10	8.9	11.2
14	6	5.6	6.4
15	4	5.9	2.7
16	7	5.9	8.3
17	3	5.6	1.6
18	3	5.0	1.8
19-20	10	7.6	13.2
≥21	8	7.9	8.1
	60	60.0	64.0

$\Sigma(f_x^2/F_x)-n = 4.0$

d.f.= 9-2=7; P>.70

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Period 4:52 P.M. to 5:20 P.M. m=18;n=28)			
≤14	9	5.3	14.0
15-17	5	7.3	3.4
18-19	4	5.1	3.1
≥20	10	9.8	10.2
	28	28.0	30.7

$\Sigma(f_x^2/F_x)n=2.7$

d.f.=4-2=2; .30>P>.20

CHI-SQUARE TEST FOR THE DISTRIBUTION OF ARRIVALS

Texas Ave. at F.M. 60, College Station, Texas

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Hour 7:30 A.M. to 8:30 A.M. m=11.3; n=60)			
≤ 7	23	7.5	70.5
8-9	6	11.0	3.3
10	4	7.0	2.3
11	2	7.1	0.6
12	2	6.7	0.6
13	3	5.9	1.5
14-15	3	8.3	1.1
≥ 16	17	6.5	44.4
	60	60.0	124.3

$\Sigma(f_x^2/F_x) - n = 64.3^{**}$

d.f.=8-2=6; $P < .001$

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Period 7:38 A.M. to 8:04 A.M. m=17; n=26)			
≤ 13	7	5.2	9.4
14-16	4	7.0	2.3
17-19	7	7.0	7.0
≥ 20	8	6.8	9.4
	26	26.0	28.1

$\Sigma(f_x^2/F_x) - n = 2.1$

d.f.=4-2=2; $.50 > P > .30$

Texas Ave. at Villa Maria, Bryan, Texas

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Hour 4:30 P.M. to 5:30 P.M. m=11.7; n=60)			
≤ 7	7	6.2	7.9
8-9	15	10.0	22.5
10	8	6.6	9.7
11	5	7.0	3.6
12	6	6.8	5.3
13	3	6.2	1.5
14	4	5.1	3.1
15-16	1	7.0	0.1
≥ 17	11	5.2	23.3
	60	60.1	77.0

$\Sigma(f_x^2/F_x) - n = 17.0^{*}$

d.f.=9-2=7; $.02 > P > .01$

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Period 4:59 P.M. to 5:20 P.M. m=16; n=21)			
≤ 13	7	5.8	8.4
14-16	3	6.1	1.5
≥ 17	11	9.1	13.3
	21	21.0	23.0

$\Sigma(f_x^2/F_x) - n = 2.2$

d.f. = 3-2=1; $.20 > P > .10$

CHI-SQUARE TEST FOR THE DISTRIBUTION OF ARRIVALS

26th Street at Waco Drive, Waco, Texas

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Hour 7:25 A.M. to 8:25 A.M. m=18; n=60)			
≤ 12	13	5.5	30.7
13-14	6	7.0	5.1
15-16	12	10.0	14.4
17	7	5.6	8.8
18	1	5.6	0.2
19-20	6	10.1	3.6
21-22	2	7.5	0.5
≥ 23	13	8.7	19.4
	60	60.0	82.7
$\Sigma(f_x^2/F_x)-n=22.7$**			
d.f.=8-2=6; P < .001			
(Peak Period 7:35 A.M. to 8:00 A.M. m=23; n=25)			
≤ 19	9	5.9	13.7
20-22	3	5.9	1.5
23-25	5	5.9	4.2
≥ 26	8	7.3	8.8
	25	25.0	28.2
$\Sigma(f_x^2/F_x)-n=3.2$			
d.f.=4-2=2; P = .20			

18th Street at Waco Drive, Waco, Texas

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(Peak Hour 4:30 P.M. to 5:30 P.M. m=12; n=60)			
≤ 7			
8-9	10	5.4	18.5
10	16	9.2	27.8
11	4	6.3	2.5
12	3	6.9	1.3
13	2	6.9	0.6
14	4	6.3	2.5
15-16	2	5.4	0.7
≥ 17	7	7.6	6.4
	12	6.0	24.0
	60	60.0	84.3
$\Sigma(f_x^2/F_x)-n=24.3$**			
d.f.=9-2=7; P = .001			
(Peak Period 5:04 P.M. to 5:22 P.M. m=18;n=18)			
≤ 15	4	5.1	3.1
16-18	6	5.0	7.2
≥ 19	8	7.9	8.1
	18	18.0	18.4
$\Sigma(f_x^2/F_x-n=0.4$			
d.f.=3-2=1; P = .50			

Appendix H

TYPICAL ANALYSIS OF THE INDEPENDENCE OF ARRIVALS FOR SUCCESSIVE INTERVALS

Once it had been established that the distribution of arrivals during the peak period could be a Poisson distribution based on the χ^2 test (Appendix G), an investigation of the independence of arrivals for successive intervals was made. The data for the following example were taken from the 1-min volume count study for Heights Street at Sixth Street in Houston (Appendix F). A graph of these arrivals is shown in Figure 2.

Time (AM)	Arrivals per Minute[a]		Combinations[a]			
	x_1	x_2	$x_1 \geq m$, $x_2 \leq m$	$x_1 < m$ $x_2 \geq m$	$x_1 > m$ $x_2 < m$	$x_1 < m$ $x_2 < m$
7:12	21	10			1	
7:13	10	21		1		
7:14	21	19			1	
7:15	19	17				1
7:16	17	19				1
7:17	19	17				1
7:18	17	22		1		
7:19	22	23	1			
7:20	23	14			1	
7:21	14	18				1
7:22	18	28		1		
7:23	28	30	1			
7:24	30	11			1	
7:25	11	21		1		
7:26	21	28	1			
7:27	28	30	1			
7:28	30	14			1	
7:29	14	23		1		
7:30	23	23	1			
7:31	23	26	1			
7:32	26	18			1	
7:33	18	30		1		
7:34	30	27	1			
7:35	27	23	1			
7:36	23	11			1	
7:37	11	21		1		
7:38	21	26	1			
7:39	26	18			1	
7:40	18	17				1
7:41	17	17				1
7:42	17	25		1		
7:43	25	End of Peak	-	-	-	-
Total observed freq. (f)			9	8	8	6
Expected freq. (F)[b]			8.7	7.75	7.75	6.0
	(f-F)		0.3	0.25	0.25	-0.8
	$(f-F)^2/F$		0.01	0.01	0.01	0.09

[a] m = mean number of vehicle arrivals per minute during the peak period (m = 20.9).
x_1 = observed arrivals per minute t.
x_2 = observed arrivals for the minute after t.

$\chi^2 = \Sigma\,(f-F)^2/F = 0.12$ on 3 degrees of freedom; P = 0.99.

[b] Expected frequencies (F) were determined by multiplying total number of pairs of intervals (31) by the probability of each of the four combinations. Thus, assuming a Poisson distribution, 53 percent of intervals would have equal to, or more than the mean number of arrivals ($P_{x \geq m}$ = 0.53). Probability of both x_1 and x_2 exceeding m would be 0.53 x 0.53 = 0.28. Lastly, 0.28 x 31 = expected frequency (F=8.7).

The preceding analysis consists of two parts: (a) the determination of the observed combinations of arrivals with respect to the mean for successive 1-min intervals, and (b) the comparison of the observed with the expected combinations by the χ^2 test assuming a Poisson distribution.

Because a probability equal to or less than 0.05 is needed to reject the hypothesis that arrivals during the peak period are of a Poisson distribution, the hypothesis is definitely accepted.

Appendix J

TIME INTERVALS BETWEEN SUCCESSIVE PASSENGER VEHICLES*

Vehicles	LEFT TURN No. Observ.	LEFT TURN Interval (seconds)	THROUGH No. Observ.	THROUGH Interval (seconds)	RIGHT TURN No. Observ.	RIGHT TURN Interval (seconds)
	(So. Frontage Road at Gulf Freeway and Cullen Blvd., Houston					
0-1	13	3.2	14	2.8	14	3.0
1-2	20	2.4	22	2.6	22	2.6
2-3	19	2.2	19	2.1	20	2.1
3-4	21	2.0	18	2.1	16	2.2
4-5	17	2.0	18	2.0	15	1.8
5-6	12		14		9	1.8
	(No. Frontage Road at Gulf Freeway and Cullen Blvd., Houston					
0-1	12	2.8	12	3.1	14	3.2
1-2	32	2.5	35	2.3	37	2.6
2-3	38	2.2	33	2.0	34	2.0
3-4	33	2.1	33	2.1	28	2.1
4-5	27	1.8	32	2.0	23	2.1
5-6	27	2.0				
	(No. Frontage Road at Gulf Freeway and Wayside Dr., Houston					
0-1	31	3.3	33	3.4	31	3.2
1-2	32	2.5	28	2.3	25	2.6
2-3	32	2.3	25	2.0	26	2.3
3-4	33	2.1	23		28	2.1
4-5	32	2.2	22	1.8	23	1.8
5-6	27	2.0	22		22	2.0
	(So. Frontage Road at Gulf Freeway and Wayside Dr., Houston†					
0-1	43	4.0	46	4.1		
1-2	38	3.2	45	2.7		
2-3	43	2.5	42	2.2		
3-4	40	2.4	34	2.3		
4-5	37	2.4	45	2.2		
5-6	26	2.4	26	2.2		

* Data taken from studies made by Capelle and Pinnell[12]
† Side-by-side left turns.

Variations in Flow at Intersections as Related to Size of City, Type of Facility and Capacity Utilization

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•BEFORE publication of the Highway Capacity Manual by the Department of Commerce in 1950, the Highway Capacity Committee of the Highway Research Board in cooperation with the State, county, and city highway departments, and the Bureau of Public Roads made the first comprehensive analysis of intersection capacities based on data gathered at about 250 locations scattered throughout the United States. The results of this study as published in the Manual have been the principal basis of capacity for intersection design. Certain factors developed through specific studies in various metropolitan areas have been made from time to time to modify the capacities as determined from the Manual. These so-called "city factors" generally increased the values for intersection capacities above those shown by the Manual, usually about 10 percent but sometimes as much as 20 percent. Design capacities as shown by the AASHO policies were being modified as much as 40 percent.

In 1956 the Bureau of Public Roads began gathering new information on traffic volumes through intersections in order to update the Manual. Far more comprehensive information was obtained during these studies than was obtained for the original intersection capacity analysis. Data for a total of 1,600 intersection approaches were recorded during periods of peak traffic flow. The data included a total of 43 variables at each location and permitted a far more comprehensive analysis than had previously been possible. A few of the more important variables not previously obtained included the degree to which the intersection was loaded, pedestrian volumes, and coordination with adjacent intersections. Also, studies at each location were continued for a period of at least $1\frac{1}{2}$ hr and usually 2 hr so that the conditions immediately preceding the succeeding the peak-hour period would be known.

TERMINOLOGY

The following are the definitions of a few of the more important variables that were obtained and analyzed in this study, some for the first time:

Load Factor

The ratio between the total number of green phases that were fully utilized by traffic during the peak hour divided by the total number of green phases for that approach during the same period. This factor is a relative measure of the degree to which the capacity of the intersection approach under the prevailing conditions was utilized during the peak hour. A green phase was considered loaded if there were vehicles entering the intersection during the entire phase with no unused time or exceedingly long spacings between vehicles at the end of the phase due to a lack of traffic.

Peak-Hour Factor

The ratio between the number of vehicles entering the intersection during the peak hour divided by four times the number of vehicles entering the intersection during the peak 15-min period. The peak 15-min period was used because it is considered the shortest time interval on which an index of the variation in traffic flow during the peak hour may be based. Traffic flow through an intersection, from a capacity viewpoint, during a 15-min period is influenced to some extent by the flow during a preceding period but not to the extent that the flow during any shorter period is influenced by the flow during a preceding period. For example, traffic entering an intersection during a specific green phase can be influenced to a large degree by the traffic conditions during the preceding green phase. This is especially true if the preceding green phase included one or more right- or left-turning movements. Likewise, the flow during a 5-min period will be influenced to a large extent by traffic conditions during the preceding 5-min period. Over a 15-min period, however, these effects are diluted sufficiently so that it is possible for an intersection to accommodate four times as much traffic during an entire 1-hr period as it accommodates during the peak 15-min period, provided of course that there is sufficient traffic approaching the intersection to utilize fully the green phases during the entire hour. It can be shown that this is not true for individual cycles or for peak periods shorter than 15 min because many of the significant variables that periodically influence the flow of traffic are not likely to occur during the shorter periods.

Vehicles per Hour of Green

The rate of traffic flow during green signal phases for a specific intersection approach. The rate includes vehicles entering the intersection or completing their movements through the intersection during the amber periods. No study made to date indicates that a more accurate capacity analysis can be obtained by including all or a portion of the amber phase with the green phase. At locations where vehicles are permitted to enter during the entire amber period, an appropriate modification may be made in the application of the results of this analysis.

Figures 1 through 5 are examples of the type of information recorded at each of the 1,600 intersection approaches. They further clarify the definitions for the load factor, the peak-hour factor, and the vehicles per hour of green as a measure of traffic flow. In these figures, each cycle during the peak hour is represented by a vertical bar showing the number of vehicles that entered the intersection from that approach during the specific cycle. The vertical bars with the close crosshatching represent green phases of the traffic signal that were loaded or fully utilized by traffic entering the intersection. The white bars that do not have the close crosshatching represent green phases that were not fully utilized by traffic. For example, in Figure 1 there were 50 green phases that were fully utilized by traffic and 16 green phases that were not fully utilized. The load factor at this location during the peak hour was therefore 0.76 (50/66).

During the peak 15-min period (or in this case, during 14.4 min which represents 16 complete cycles of 54 sec each) a total of 367 vehicles entered the intersection from this one approach (Fig. 1). This is a rate of flow of 1,529 vehicles per hour or 2,759 vehicles per hour of green for the 30-sec green periods.

During the entire peak hour 1,427 vehicles entered the intersection from this approach. This is a rate of flow of 2,569 vehicles per hour of green. The peak-hour factor was therefore 0.93 (1,427 divided by 1,529 or 2,569 divided by 2,759). This approach handled 93 percent as much traffic during the peak hour as it would have handled if the flow for the entire hour was the same as it was during the peak 15-min period. During many of the green phases that were not fully loaded, more traffic entered the intersection than during some of the pahses that were fully loaded. Also, both the loaded and unloaded phases were well distributed throughout the hour. In fact, during the peak 15-min period (14.4 min) relatively fewer green phases were loaded than during the hour. It is believed, therefore, that the load factor and the peak-hour factor are two extremely important variables to consider in a study of intersection capacities as related to design. They provide a means for accounting for a large share of the difference in traffic flow at intersections with similar geometric design characteristics. The peak-hour factor is as important in studying short-time fluctuations in traffic flow as the 30th hour factor is in a study of seasonal and daily flucuations of traffic.

Figure 2 shows the same type of information as Figure 1, but for another approach. This approach also had two traffic lanes. In addition, Figure 2 shows whether both of the traffic lanes were loaded during each green phase. If both lanes were loaded during the green phase, the entire vertical bar is crosshatched. If only the right-hand lane

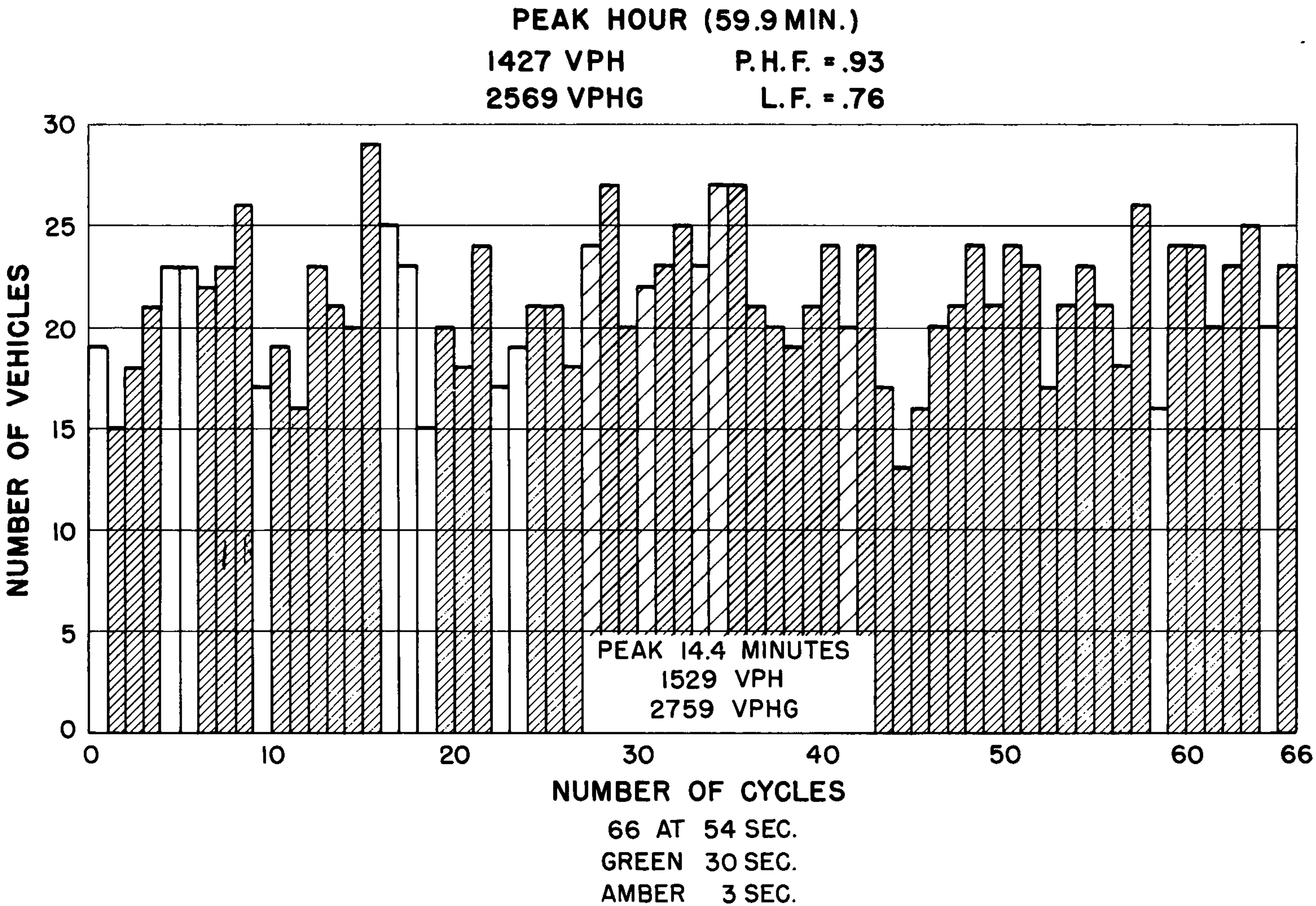

Figure 1. Two-way street, no parking, 20-ft approach.

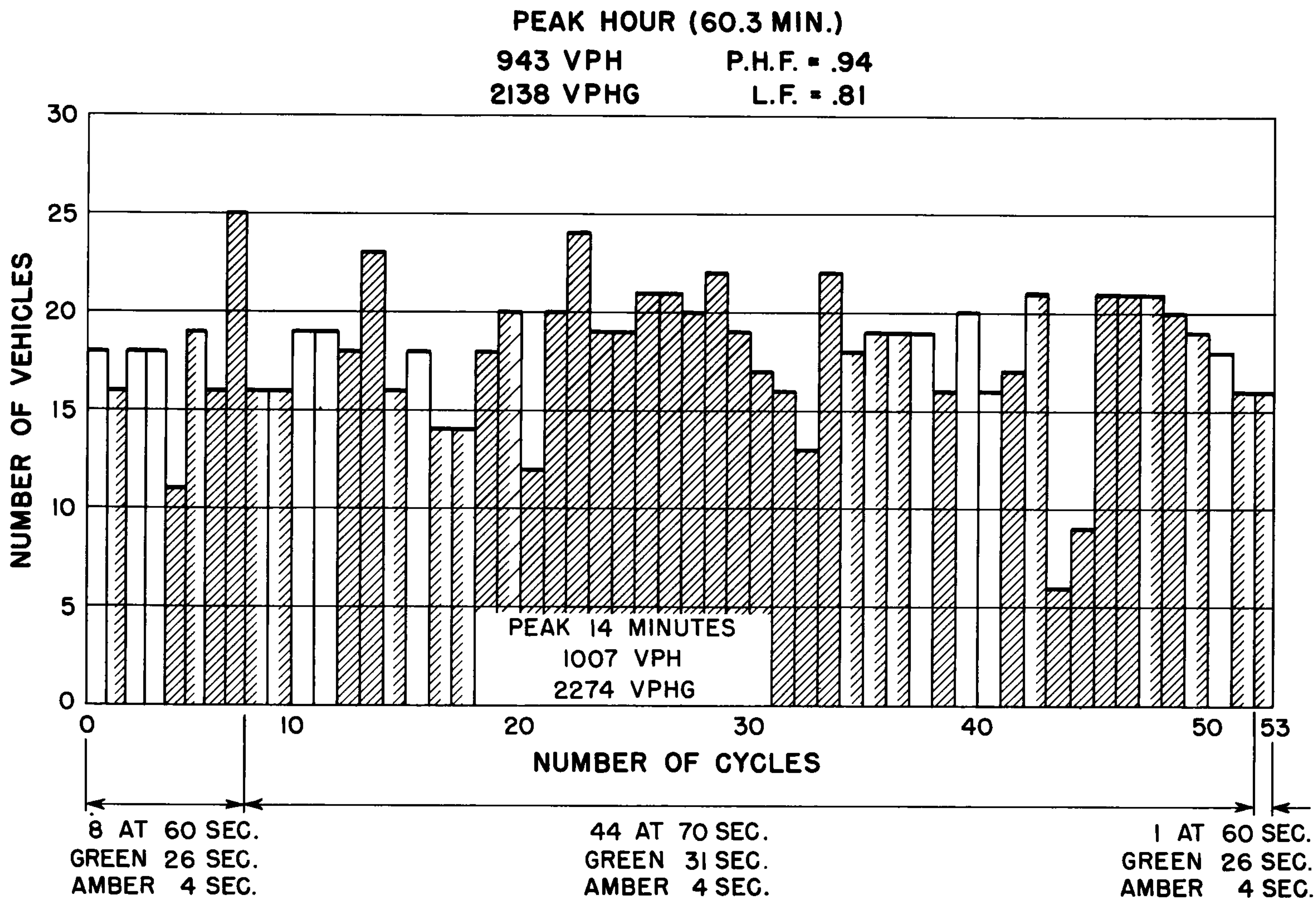

Figure 2. Two-way street, no parking, 22-ft approach.

was loaded during the green period, the right side of the bar is crosshatched, and if only the left-hand lane was loaded for the entire period the left side of the bar is crosshatched. In calculating the load factor, all the green phases with either one or both lanes loaded were divided by the total number of green phases during the peak hour. It might have been desirable to make some distinction between the green phases in which all lanes were loaded and those in which only a single lane was loaded, but inasmuch as this information was not available for all locations included in the study, such a refinement was not possible.

Figures 3, 4, and 5 show information similar to that in Figures 1 and 2 except the data are for other intersection approaches. In Figure 4, the 13th, 16th, and 17th cycles accommodated an exceedingly large number of vehicles during the green phase. Although this probably represents the most unusual condition that was recorded during any of the intersection studies, it was very commonly found that the first cycles during the initial period when an intersection was fully loaded accommodated a much larger number of vehicles than the succeeding cycles. Apparently as a loaded condition continues over a period of time it is less likely that the high initial volumes can be maintained. This is directly related to the finding discussed in a subsequent section of this report which indicates that an intersection will carry higher rates of flow for short periods if the flow on the facility builds up very suddenly than if the rate of flow increases gradually.

Figure 5 illustrates a condition that occurred on one intersection approach when the cycle length and the length of green phase were changed during the peak hour. In this case the cycle length was changed from 50 to 90 sec and the green phase from 22 to 55 sec. The large increase in the number of vehicles entering the approach after the cycle was changed from 50 to 90 sec was due to the length of the green phase rather than due to the fact that the cycle length was increased. Had this change taken place a little earlier during the peak hour, no congestion or backlog of vehicles would have occurred on this approach. Data similar to those shown in Figures 1 through 5 were available for most of the 1,600 locations at which data were obtained for this research project.

RELATION BETWEEN INTERSECTION CAPACITY, DELAY AND LOAD FACTORS

The principal objective of studying traffic flow at intersections is to improve the efficiency of traffic movement, particularly with respect to reducing delays and traffic accidents. Many studies made in the past have been concerned principally with a measurement of delay. In some specific instances this approach is justified but generally for a study of intersection capacities from data based on actual observations of existing facilities, a study of delays, no matter how accurately recorded, can result in some very erroneous conclusions. Figure 6 was prepared to illustrate this point and to define further the significance of the load factor.

Figure 6 shows the increase in the average delay per vehicle with an increase in traffic volume approaching an isolated signalized location. Referring to Curve A, when the traffic volume is very low, there is a certain amount of delay because some of the vehicles reach the signal while it is amber or red. As the traffic volume increases, there is at first a very slight but constant increase in the average delay due to some interference between vehicles. As the traffic volume continues to increase, a point will be reached where any further increase will result in the green time during one cycle in the peak hour being fully utilized. Up to this point, the load factor as defined for this study has been 0.00. As the traffic volume continues to increase more and more of the green phases will become loaded and the delay will increase at a more rapid rate with each increase in the number of loaded phases during the peak hour. As the approach volume continues to increase, all of the cycles during the peak hour will eventually become loaded. At this point the possible capacity of the intersection under the prevailing traffic conditions has been reached and the load factor is 1.00.

The delay will continue to increase even though there is no further increase in the approach volume but there can be no further increase in the capacity of the approach as indicated by the vertical portion of Curve A. There is, however, no limit to the amount of delay that might occur after the possible capacity has been reached. The

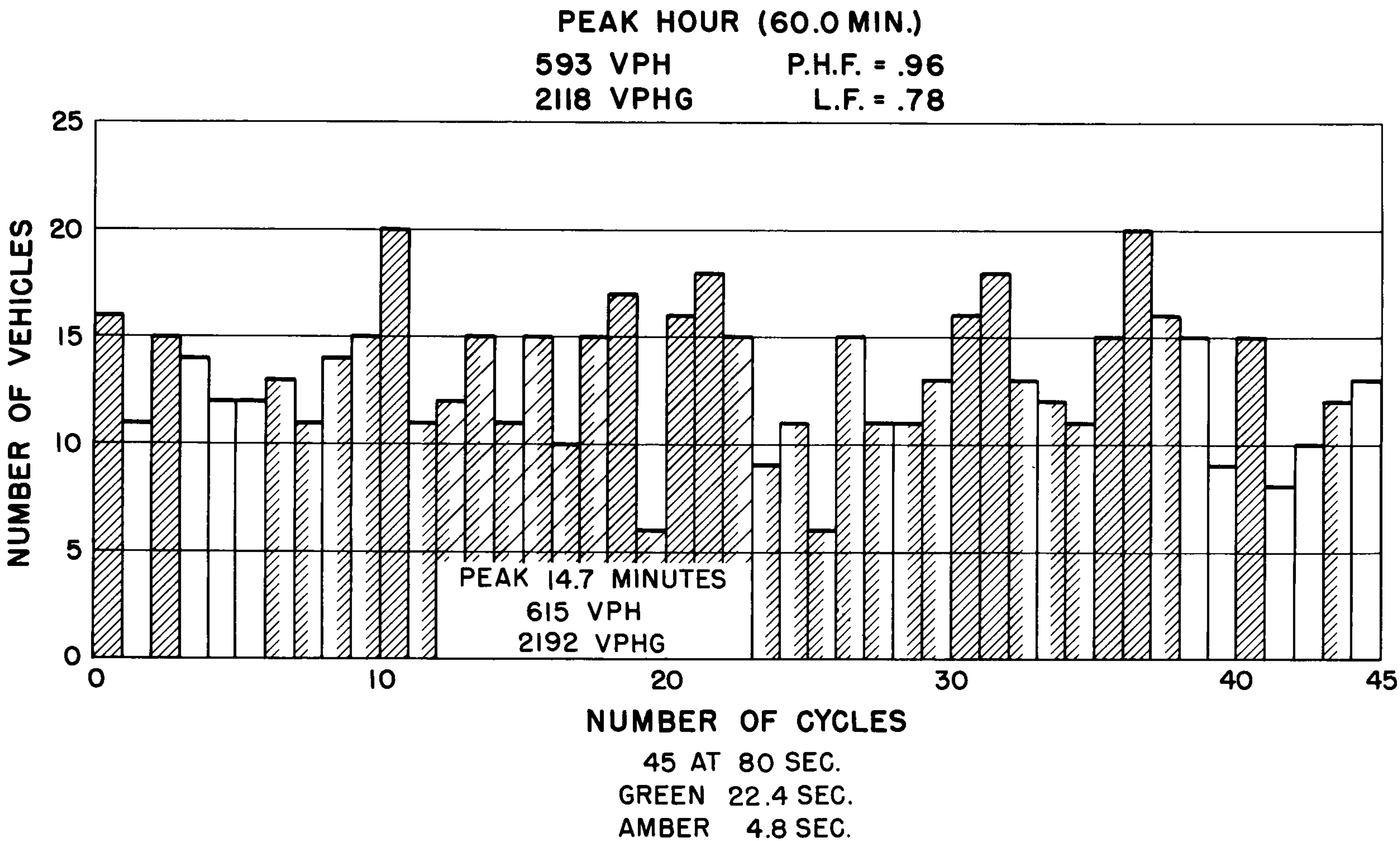

Figure 3. Two-way street, no parking, 22.5-ft approach.

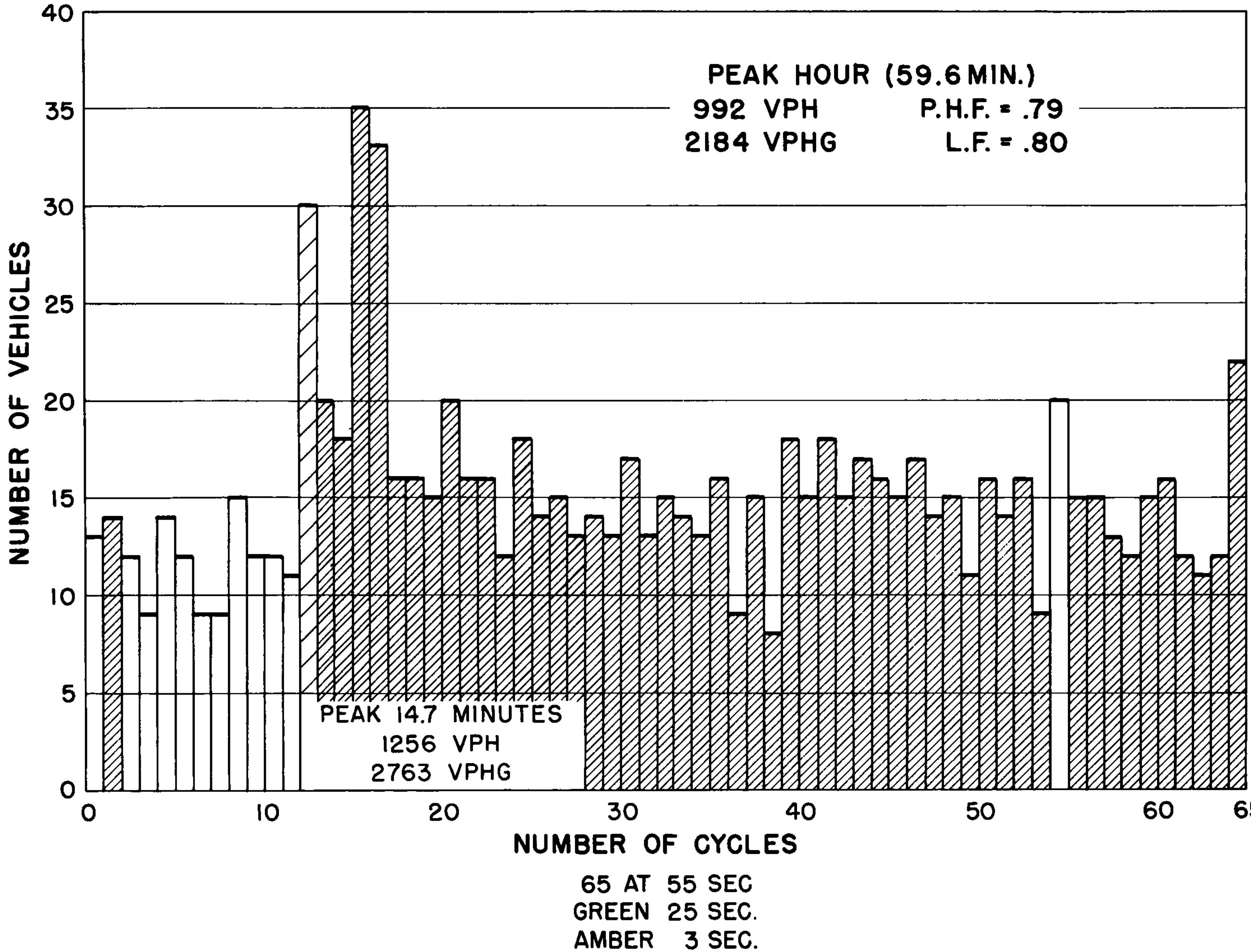

Figure 4. Two-way street, no parking, 21-ft approach.

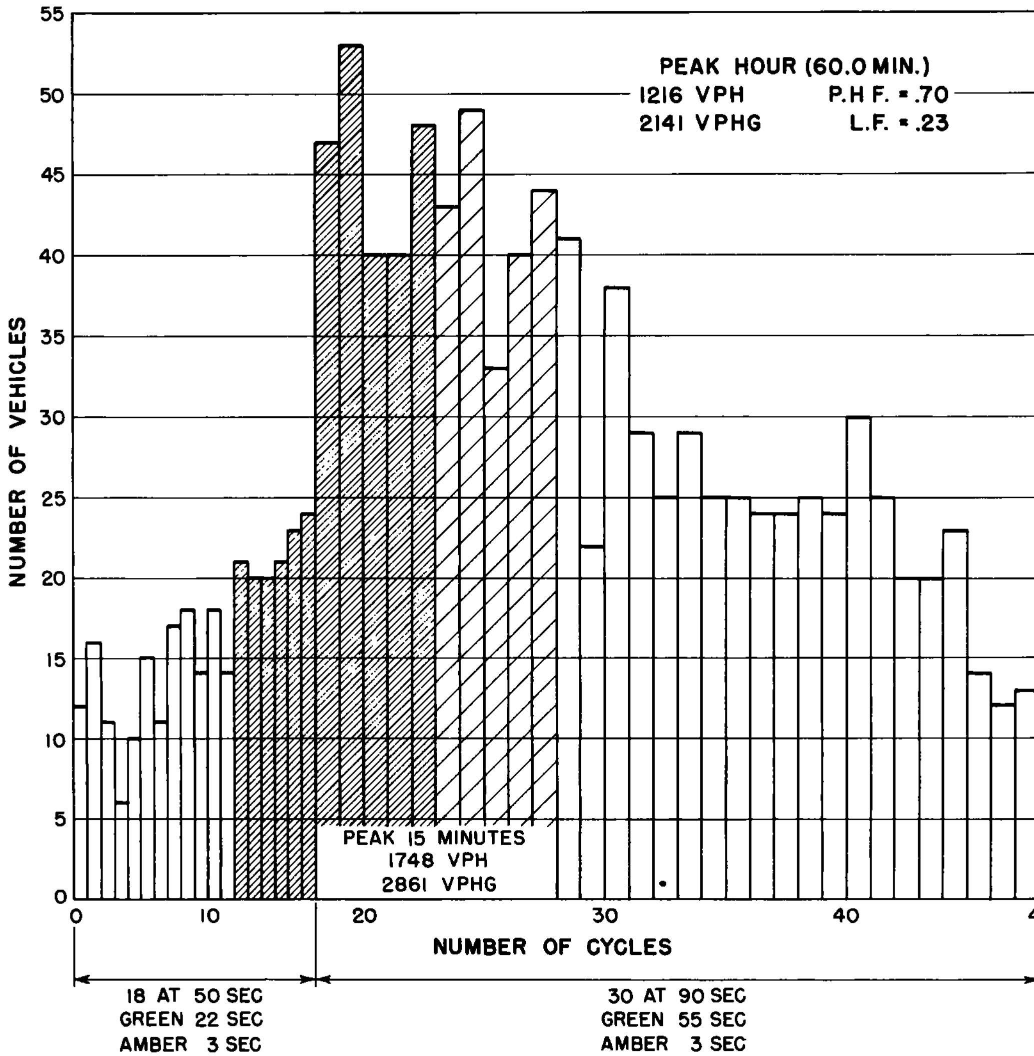

Figure 5. Two-way street, no parking, 22-ft approach.

average delay will depend entirely on the extent and length of time that the approach volume exceeds the intersection's capacity.

Curve B of Figure 6 represents the delay that might occur at the same intersection if it were located near another signalized intersection and the two signals were not properly coordinated. In this case the delay also increases very little with an increase in traffic volume until a point is reached where the delay starts to increase very rapidly. This delay curve would also become vertical at approximately the same number of vehicles per hour of green as for the condition represented by Curve A.

In the case of adjacent intersections, the magnitude of the delay at the second intersection is affected by the "offset" between the green phases at the two intersections. This is one reason that the load factor rather than vehicular delays is more appropriate for a study of intersection capacities. Admittedly, a further refinement in the load factor as recorded by the field parties would have been desirable, but delays, no matter how accurately recorded, would not have been as useful for a capacity deter-

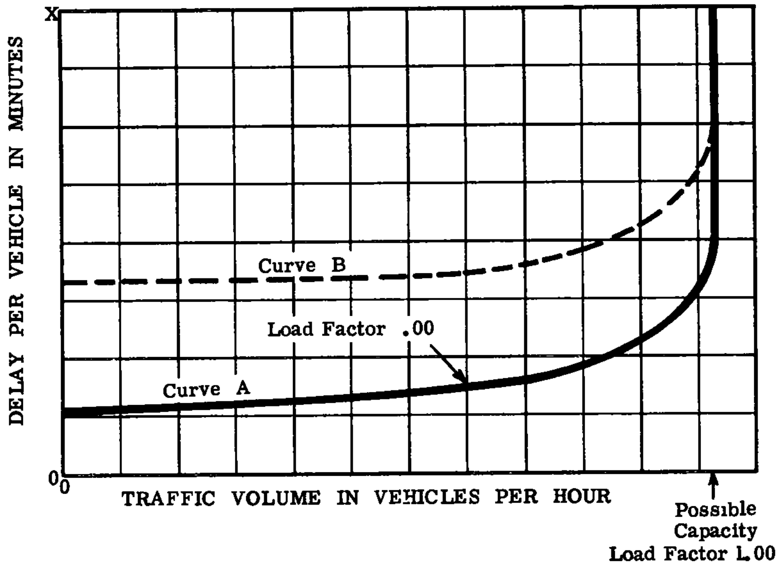

Figure 6. Relation between intersection capacity, delay, and load factors.

mination as the load factor unless for each of the intersections sufficient preliminary investigations had been made to be assured that the offsets to nearby intersections were properly adjusted so as to give a minimum over-all delay per vehicle.

For a study of intersection capacities, it is necessary to include only those approaches operating at or near capacity volumes. The load factor is a means whereby this determination can be made. An intersection approach where none of the green phases was loaded should obviously not be used. Furthermore, a number of conditions can occur at an intersection operating with relatively light traffic volumes that will cause an occasional green phase to become loaded. For this reason only those intersection approaches having about 10 percent or more of the green phases loaded were used for this analysis. This requirement, together with the requirement that the other data be complete and accurately recorded eliminated data for some locations from the analysis. Also, unusual layouts or intersections with more than four approaches were not included. This reduced the total number of approaches for analysis from 1,600 to 792 under fair-weather conditions. Approximately 200 additional studies made during inclement weather were suitable for analysis.

VARIATION IN PEAK-HOUR FACTOR

As defined, it is possible for the peak-hour factor to vary from 0.25 to 1.00. If the traffic flow is uniform during the entire peak hour so that each 15-min period carries the same amount of traffic as the other three 15-min periods during the hour, the peak-hour factor will be 1.00. At the other extreme, if all the traffic during the peak hour occurs during a single 15-min period with no traffic during the other three 15-min periods, the peak-hour factor will be 0.25. It is very unlikely, however, that such a condition will occur. In fact, the lowest peak-hour factor recorded during any of these studies was 0.47, with over one-half of the hourly flow during the peak 15-min period. At most locations, however, the peak-hour factor was in the neighborhood of 0.85 with 75 percent of the locations between 0.80 and 0.95.

Figures 7 through 11 show the distribution of the peak-hour factors for one-way and two-way streets with and without parking. There was some difference in this factor between the various types of streets that were included in this study, but apparently this was due to the method of sampling rather than due to any marked characteristic of the different types of streets as related to the peak-hour factor. The average peak-hour factor for all of the locations included in this analysis was 0.853.

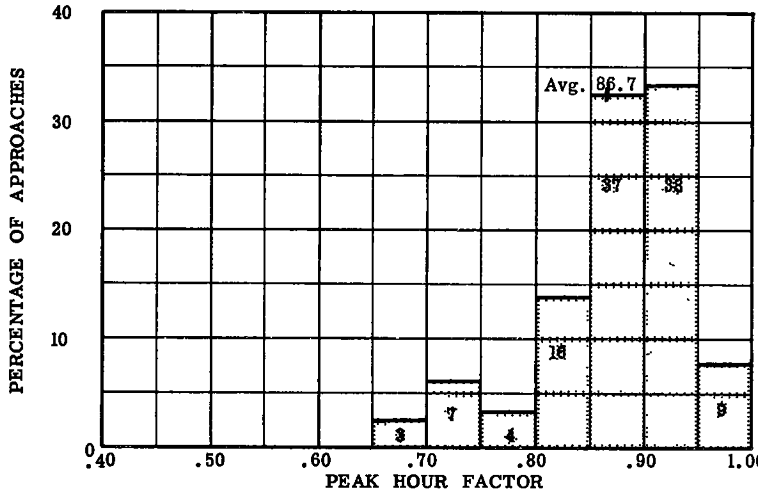

Figure 7. Distribution of peak-hour factors, one-way streets.

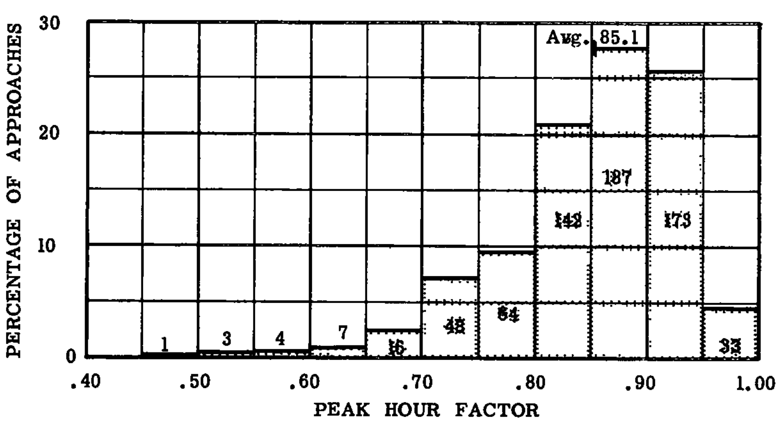

Figure 8. Distribution of peak-hour factors, two-way streets.

RELATION BETWEEN PEAK-HOUR FACTOR AND APPROACH CAPACITIES

Figure 12 shows, for the various peak-hour factors on two-way streets, the average traffic flow in terms of the vehicles per hour of green. As the peak-hour factor increases, the flow increases but at a very non-uniform rate.

Many other factors other than the peak-hour factor influenced the traffic flow. This is shown by Figures 13 through 17. For example, Figure 13 shows that the average street with a low peak-hour factor was narrower than the average street with a high peak-hour factor. Likewise, Figures 14 through 17 show that the average load factor, the average city size, the cycle length, and the length of the green phase all have a tendency to be higher at locations with high peak-hour factors than at locations with low peak-hour factors. Figure 12 does not therefore show the true relationship between the peak-hour factor and the traffic flow because all of these other variables also had a tendency to influence the relationship.

One would expect a direct relationship between the possible capacity in terms of the hourly flow and the peak-hour factor. For example, an intersection having a peak-hour factor of 1.00 should be expected to accommodate 25 percent more traffic during the peak hour than an identical intersection with a peak-hour factor of 0.80 without exper-

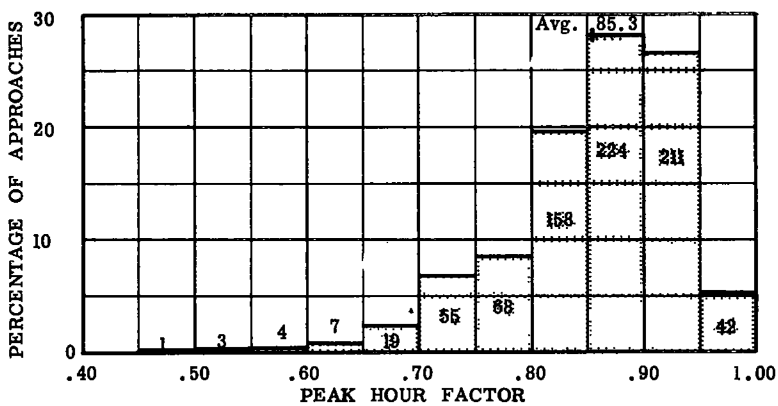

Figure 9. Distribution of peak-hour factors, all streets.

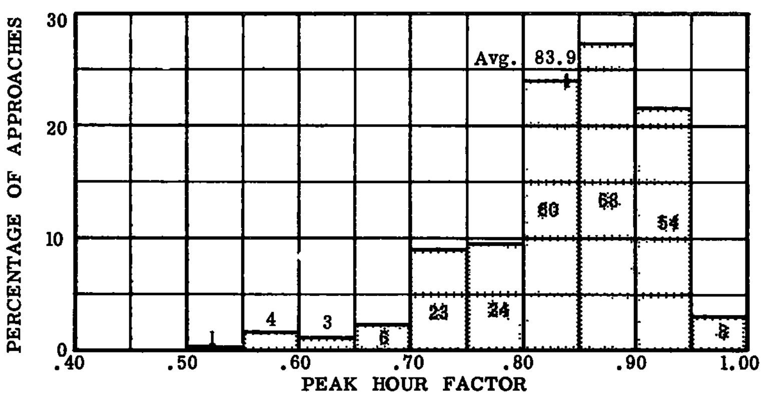

Figure 10. Distribution of peak-hour factors, two-way streets with parking.

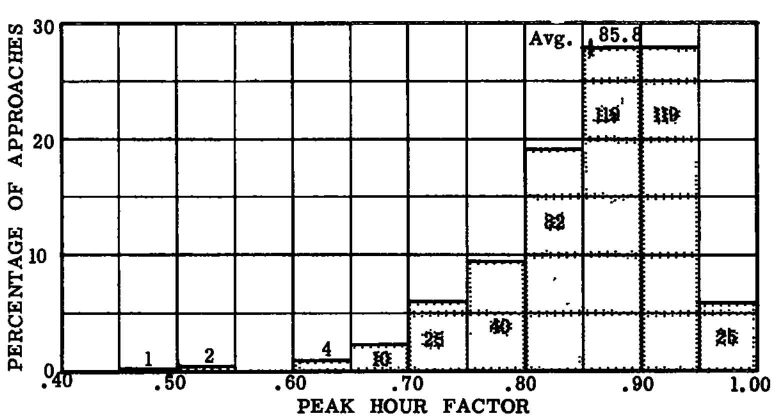

Figure 11. Distribution of peak-hour factors, two-way streets, no parking.

iencing any more congestion during the peak period. This would be true if both intersections accommodated the same number of vehicles during the peak 15-min period. A subsequent analysis shows, however, that the peak rate of flow for a 15-min period

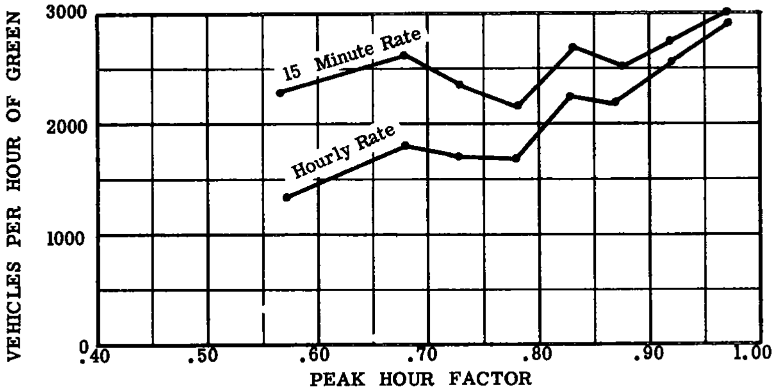

Figure 12. Traffic flow on two-way streets by peak-hour factors (no parking and uncorrected for street width and other factors).

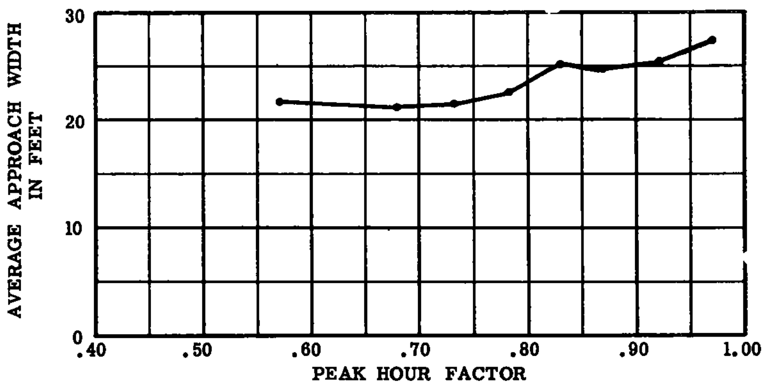

Figure 13. Average width of approaches on two-way streets by peak-hour factor.

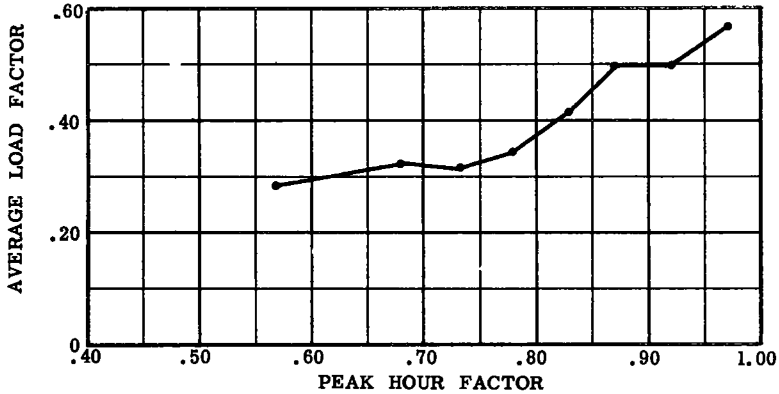

Figure 14. Average load factors for two-way streets by peak-hour factor.

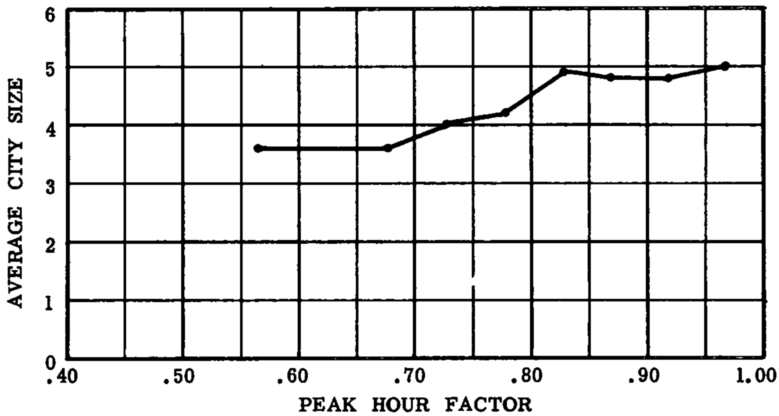

Figure 15. Average city size as related to approaches on two-way streets with no parking, by peak-hour factor.

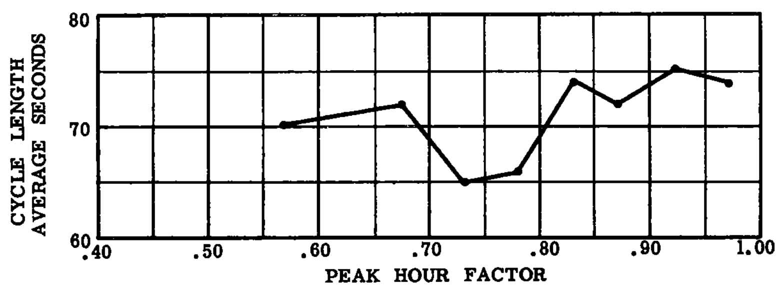

Figure 16. Average cycle length as related to approaches on two-way streets with no parking, by peak-hour factor.

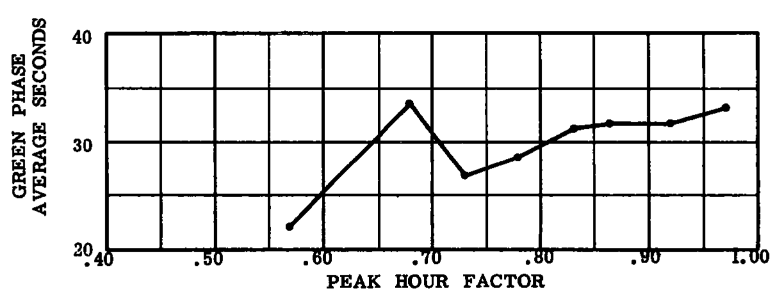

Figure 17. Average green phase as related to approaches on two-way streets with no parking, by peak-hour factor.

can be higher on a street with a low peak-hour factor than on an identical street with a high peak-hour factor. In other words, the same street will accommodate an extremely high rate of flow for a short period if the flow preceding the peak is low. Evidently the lower the flow preceding the peak, the less likelihood there is for the peak flow to be reduced as a result of the preceding flow.

METHOD OF ANALYSIS

Shortly after the field data were compiled and placed on punch cards, a contract was let to a contractor having the necessary equipment and personnel to perform a comprehensive analysis of the data. This firm worked on the analysis for a period of two years, using the most recent high-speed computer equipment available and employing the latest statistical methods and systems analysis procedures. The final result was a series of five equations, each with 14 different variables. Each equation represented intersection capacities for one of the following conditions:

1. Adverse weather conditions;
2. Locations in the central business districts;
3. Locations in the fringe business districts;
4. Noncentral locations with lane lines;
5. Noncentral locations with no lane lines.

The results were very disappointing, however, because the effect that the individual factors in the equation had on intersection capacities was not in line with the results as obtained from other research and from experience by professional engineers in traffic operations. For example, the effect of parking on intersection capacities as included in the equations was far from anything that could be considered reasonable. Furthermore, the application of the equations to the field data used for the analysis showed that the ability to predict the capacity of an intersection without considerable error was rather remote. For example, in 12 percent of the cases the predicted capacity was more than 50 percent higher or lower than the actual flow. Also in 48 percent of the cases the predicted capacity was more than 20 percent off and in 71 percent of the cases the predicted capacity was more than 10 percent off. To be useful for design purposes, it should be possible to obtain an accuracy of within 10 percent in most cases and within 20 percent except in rare cases involving unusual intersection layouts. It is believed that the following were the principal reasons that the results were so erratic:

1. Separate equations were not provided for one-way and two-way streets.
2. Separate equations were not provided for streets with and without parking.
3. Lane lines have an important effect on intersection capacities but there is a large variation in this effect depending on the specific width of the street.
4. The effect of certain variables, such as the length of the green phase, is not a straight-line relationship.

It is believed, however, that the principal reason for the erratic results and lack of correlation was the fact that each equation was derived from data including a mixture of one- and two-way streets some with and some without parking. The results of the current analysis definitely show that many of the variables have a different effect on one-way streets than on two-way streets and on streets with parking as compared with streets without parking. The analysis made by the contractor was, however, beneficial in that certain variables were found to affect intersection capacities to a much greater degree than other variables.

For the current study, the data were separated into five primary groups based on the type of street and parking conditions. A separate analysis was made for each. They include intersection approaches on the following:

1. One-way streets with no parking;
2. One-way streets with parking on one side;
3. One-way streets with parking on both sides;
4. Two-way streets with no parking;
5. Two-way streets with parking on both sides.

The effect of each of the more important variables was determined separately for intersection approaches involving these five types of streets. A separate analysis was also made for inclement weather conditions.

The following were four principal variables found to affect the hourly flow of traffic through intersections, other than one- and two-way operation and the presence of parked vehicles:

1. Peak-hour factor;
2. Load factor;
3. The approach width at the intersection;
4. Size of the city.

It was immediately apparent that the effect of these variables had to be accurately determined before the data could be used to obtain the effect of the many other variables such as right and left turning movements, commercial vehicles, cycle length, lane width, type of signal control, location of bus stops, and pavement markings. These variables for simplicity are referred to as secondary variables whereas the four previously listed are referred to as the primary variables for each of the five types of streets.

Many independent studies conducted over the past several years have produced information relating to the effects on capacity of several of the secondary variables. With the exception of the approach width, however, no comprehensive studies have been made to determine the effect of the primary variables on intersection capacities or hourly flows. It therefore seemed most appropriate first to analyze the primary factors as related to traffic flow at intersections because they are the principal measures or variables that indicate the patterns of mass traffic movement on the approaches whereas the secondary variables relate principally to traffic control measures, and the traffic movements within the intersection. Furthermore, when the intersections included in any one of the primary classifications (street type) are grouped according to the magnitude of any one of the variables, the average values for the secondary variables are generally about the same for all the groups when each contains in the neighborhood of 10 or more intersections. This is not true for the averages that involve the primary variables. Also, because an approximate value for the effect of each of the secondary variables on capacity is already known, reasonable adjustments can be made when plotting a curve whenever the average value of a secondary variable for one of the points is out of line. For example, the average value for each of the secondary variables was approximately the same for the points shown in Figure 18. The only significant exceptions were the following:

1. For the point representing 4 approaches, the average percentage of left-turning movements was double the percentage at the locations represented by the other points (14 percent against 7 percent).
2. For the point representing 3 locations, there were only one-half as many turning movements as at the other locations.
3. For the point representing 6 locations, there were only one-half as many local busses on a percentage basis as at the other locations.

Any reasonable adjustment made for these conditions would tend to make these three points fall closer to the average line, showing the effect of the peak-hour factor on the traffic flow, than they are now located.

One of the rather unusual features of this analysis was that no extensive statistical procedures were employed as the individual relation between one variable and the hourly capacity or traffic flow was developed. This would have been a waste of effort at this stage of the analysis and the statistical results would probably have been improperly interpreted to the same degree that they were in the initial analysis performed by contract. It is obvious that any one variable is not likely to have as great an effect on capacity as the combined effect of some 43 other variables. There is therefore bound to be a wide dispersion of the points when one of the independent variables is plotted against the traffic flow as the dependent variable. The resulting dispersion (or the coefficient of correlation) is only an indicator of how close the traffic flow can be predicted from that one variable. It is no measure whatever of the accuracy of the relation developed between the independent and dependent variables. This is the reason for leaving any statistical analysis until the combined effect of the accuracy of all variables in predicting the traffic flow can be determined. A comparison of the actual

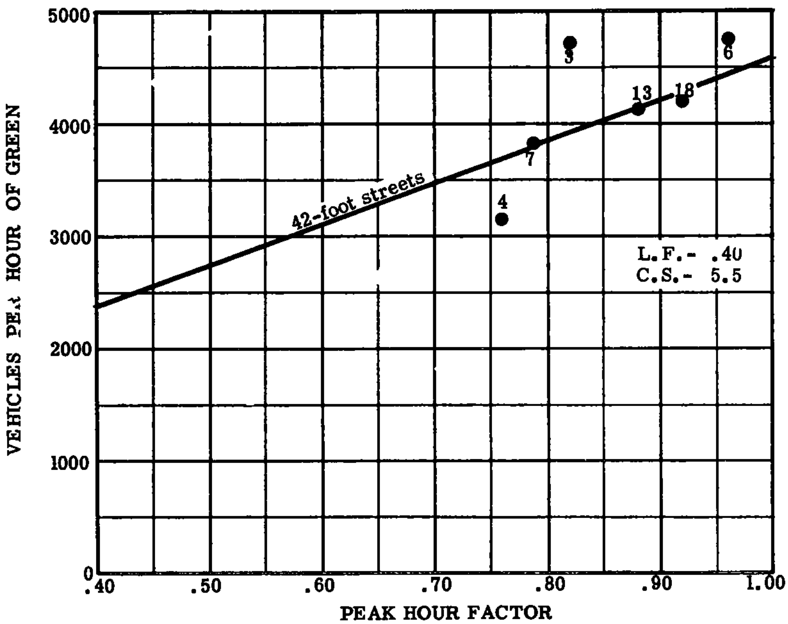

Figure 18. Effect of peak-hour factor on hourly intersection capacities, one-way streets with no parking.

traffic flow at individual intersections with the predicted flow calculated from the combined effect as determined for each independent variable will then be the "proof of the pudding."

Another consideration in plotting the curve to show the relation between an independent and a dependent variable from a series of points, other than the commonly used method of least squares, was the assumption that the curve for one set of points should have some relation to the curve for another set of points involving the same variables but of different magnitude. In other words, when more than two variables are involved, each curve for two of the variables should fit into a series forming a family of related curves in the same manner as would be the case by applying multiple correlation to three or more variables. This procedure was especially important in selecting the most appropriate curve when two or more curves would fit a series of plotted points equally well. Also, theoretical relationships and the results obtained by other studies in the same area influenced the selection in such instances.

Multiple correlation was not used for this analysis in view of the results previously obtained by the contract. Also, because new variables were being investigated, multiple correlation would not have disclosed whether the proper form of equation had been used or whether the data had been properly classified or segregated into appropriate groups.

The effect on capacity of the four primary variables was determined by a series of successive approximations because the method of multiple correlation had resulted in producing a relation that could not be considered reasonable. Each of three of the four variables was first assigned an assumed effect to determine a preliminary effect of the fourth variable. The results were then applied together with the previously assumed values for two variables to arrive at a more exact effect for the third variable which had previously been assumed. This procedure was then applied to determine more exact values for the other variables, then the entire series of calculations was repeated for all variables until there was no change in the resulting effect that any one of the variables had on the intersection capacities. This was a rather time-consuming process involving about 100 IBM tabulations and thousands of manual calculations. Before starting this procedure, however, special IBM tabulations were made listing all the variables for each intersection with two related variables shown in adjacent columns and in order by the magnitude of one of the variables. This was done to discover "odd balls" in the data and permit a thorough check in each such case with the original field sheets and in some cases with field conditions.

For example, when the intersection approach widths listed in order of magnitude were compared with the number of lanes as shown in the adjacent column, many cases such as a 24-ft approach with three lanes and parking, or a 12-ft approach with two lanes were discovered to be included in the data. In each such case, the data were corrected when the check produced reliable evidence that an error had been made while the field data were being recorded or in processing the data to IBM cards. In no case were data changed without complete information, and in no case were the data for any intersection discarded regardless of how unreasonable it appeared.

Some 300 substantial corrections were made in the data which involved over 40,000 items. There were undoubtedly many errors that were not detected, as evidenced by the "odd balls" that repeatedly appeared in successive tabulations. On the whole, however, the data were remarkably accurate and whatever errors remained could not have had a significant effect on the average values obtained by the analysis.

PEAK-HOUR FACTOR

Theoretically at least, the total flow that an intersection approach will accommodate during a peak hour should be directly related and proportional to the peak-hour factor. This can be true in practice, however, only if the approach can accommodate the same rate of flow for an hour as for a 15-min period. Furthermore, the peak 15 min for the hours with the higher peak-hour factors must be loaded to the same extent and carry the same rate of flow as the 15-min periods for the hours with the lower peak-hour factors. It was impossible to observe locations or analyze these data in such a manner as to control these two variables because, as shown later, the peak-hour factor does not change greatly from day to day at locations carrying capacity or near-capacity volumes for at least 15 min during the peak hours. (This statement may not and probably does not apply to changes in the peak-hour pattern that take place over a long period of time, such as those that occur with a large increase in the yearly flow.)

For this analysis of the effect of the peak-hour factor on intersection capacity, the other factors including the "load" factor during the peak hour were held constant. The conditions required for the theoretical relation between the peak-hour factor and the total traffic flow during the peak hour as set forth in the preceding paragraph could not be fulfilled. To illustrate this point, assume two intersection approaches of identical geometric design both having the same load factor of 0.40 and 60 traffic signal cycles per hour. The first has a peak-hour factor of 0.60 and the second a peak-hour factor of 0.90. In the first case, the 24 loaded cycles would necessarily be concentrated in and near the peak 15-min period, whereas in the second case, the 24 loaded cycles would most likely be reasonably well distributed throughout the hour. The flow during the peak 15-min period would therefore be somewhat greater in the first case than in the second, but the total hourly flow would be considerably lower and the total delays to traffic considerably greater in the first case. A series of successive green phases that are loaded indicates a backlog of vehicles on an approach, whereas a distribution of loaded phases throughout the hour separated by phases that are not fully loaded indicates a uniformly high flow during the hour with little or no backlog on the approach at any time.

Figures 18 through 22 show the effect of the peak-hour factor on hourly intersection capacities for the five types of streets. In each case, the load factor and city size are constant and both correspond with the average values represented by the data for the specific type of street. The load factor had approximately the same effect on the hourly

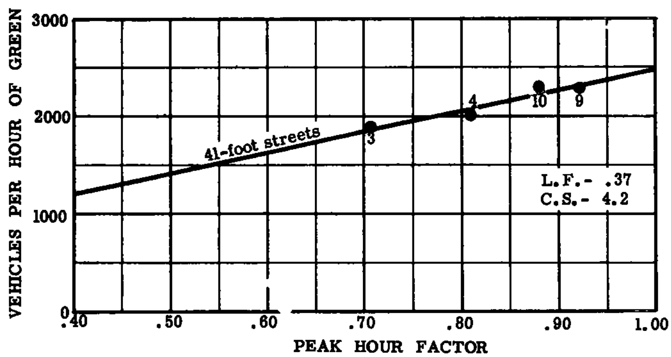

Figure 19. Effect of peak-hour factor on hourly intersection capacities, one-way streets with parking on one side.

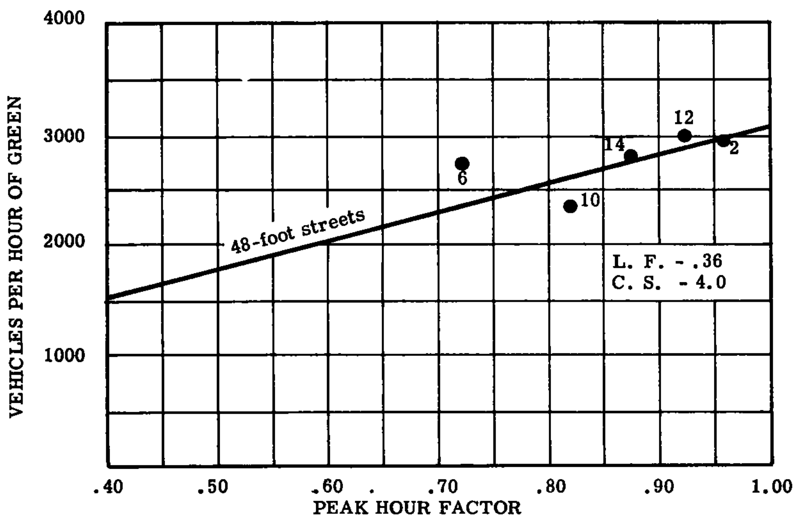

Figure 20. Effect of peak-hour factor on hourly intersection capacities, one-way streets with parking on both sides.

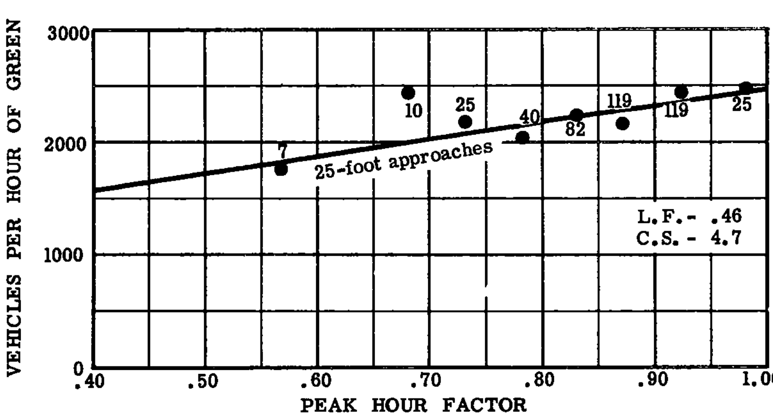

Figure 21. Effect of peak-hour factor on hourly intersection capacities, two-way streets, no parking.

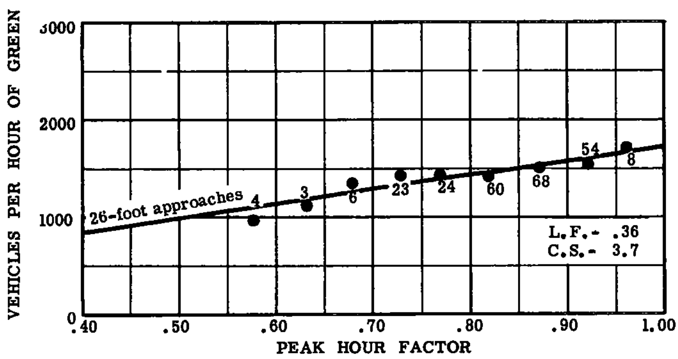

Figure 22. Effect of peak-hour factor on hourly intersection capacities, two-way streets with parking.

capacities for the three types of one-way streets as for the two-way streets with parking when the change in capacity is considered on a percentage basis. The effect for these four types of streets was, however, considerably different than for the two-way streets with no parking. This is illustrated by Table 1 which gives for each type of street the percentage increase in the peak-hour traffic with an increase in the peak-hour factor from 0.75 to 1.00.

On two-way streets with no parking, the effect of a change in the peak-hour factor on the peak-hour flow may be represented by the following equation:

$$\text{Change in peak-hour flow} = \left(\frac{\text{New PHF} + 0.653}{\text{Observed PHF} + 0.653} - 1\right) \text{observed flow}$$

The change and the observed flow may be either in terms of VPHG or VPH. For example, if 900 vehicles had been observed entering an intersection approach on a two-way street without parking during a peak hour while the peak-hour factor was 0.70 and 40 percent of the cycles were loaded, that same approach would accommodate 133 more vehicles or a total of 1,033 vehicles if the traffic pattern changed so that the peak-hour factor was 0.90, providing all other conditions including the number of loaded cycles during the hour remained the same. The effect of a change in the peak-hour factor is much greater for the other four types of streets, including all one-way streets and two-way streets with parking, than for two-way streets without parking. The change on these streets may be represented by the following equation:

$$\text{Change in peak-hour flow} = \left(\frac{\text{New PHF} + 0.20}{\text{Observed PHF} + 0.20} - 1\right) \text{observed flow}$$

If the effect of the peak-hour factor on traffic flow is to be of any value in intersection design, or to improve traffic conditions through better control methods, an understanding of the conditions that produce or cause changes to occur in this factor must be understood. This discussion is, however, deferred until after the analysis of the effect on capacity of other factors.

TABLE 1

INCREASE IN HOURLY FLOW

Type of Street	Fig. No.	Increase in Hourly Flow (%)
One-way:		
No parking	18	25
Parking one side	19	27
Parking both sides	20	27
Two-way:		
No parking	21	18
Parking both sides	22	26

EFFECT OF LOAD FACTOR

Figures 23 through 27 show the effect of the load factors on the traffic flows entering intersections from approaches on the five different types of streets when the

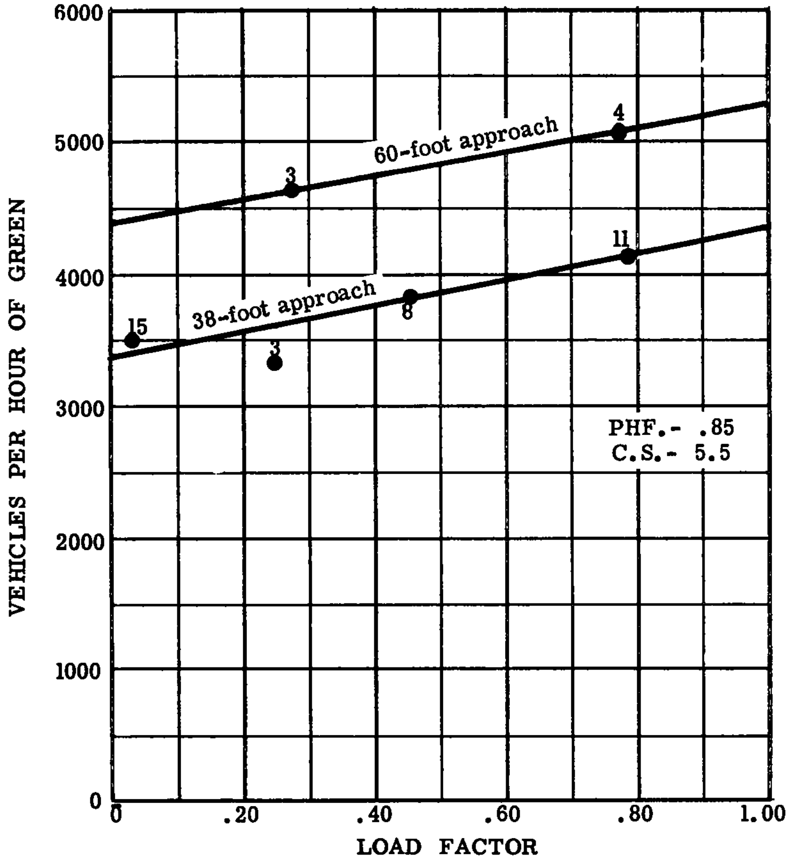

Figure 23. Effect of load factor on hourly intersection capacities, one-way streets with no parking.

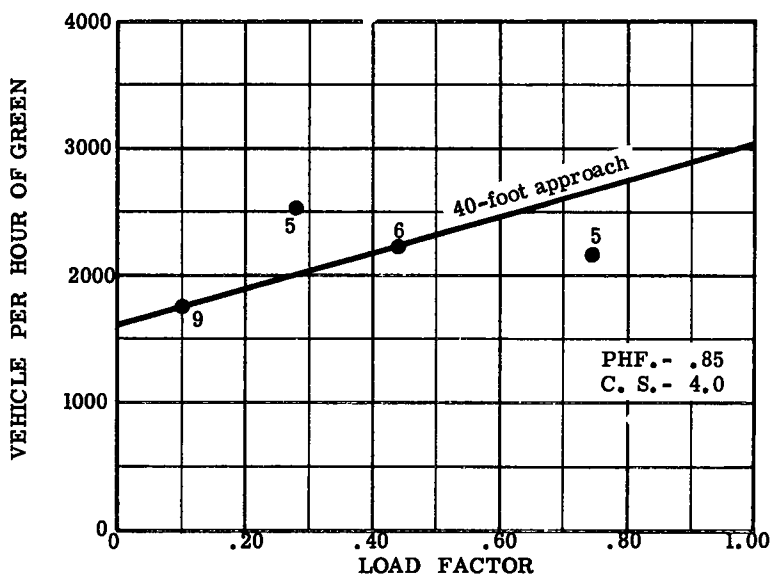

Figure 24. Effect of load factor on hourly intersection capacities, one-way streets with parking on one side.

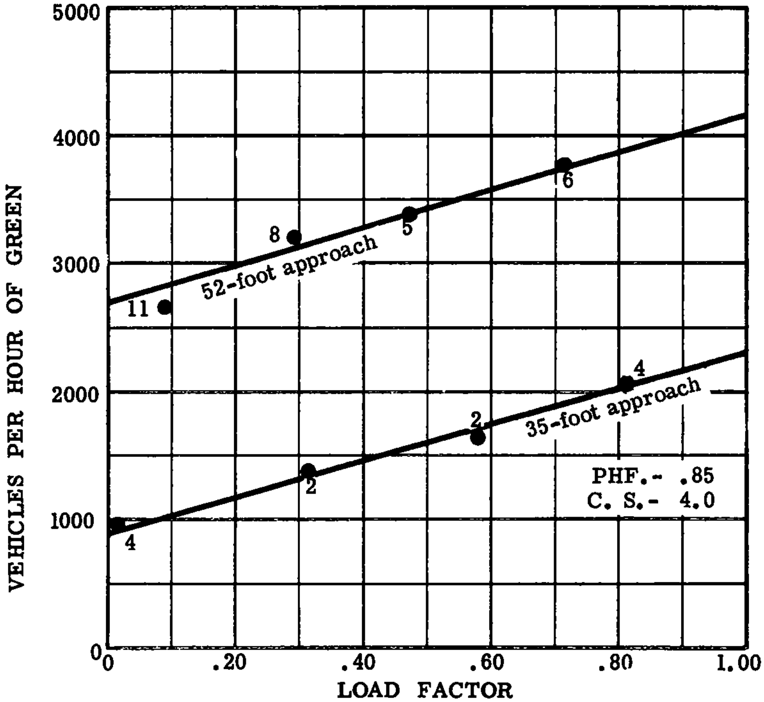

Figure 25. Effect of load factor on hourly intersection capacities, one-way streets with parking on both sides.

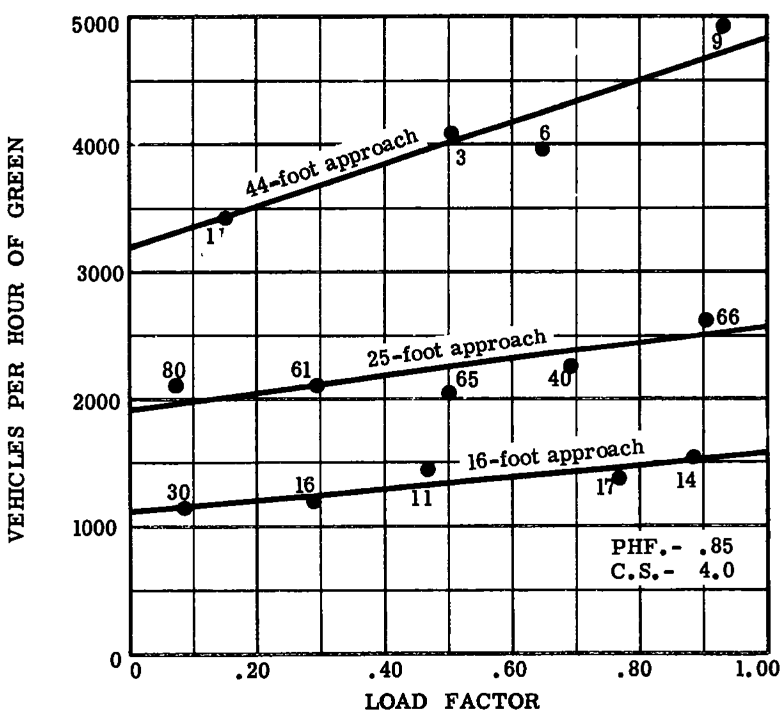

Figure 26. Effect of load factor on hourly intersection capacities, two-way streets, no parking.

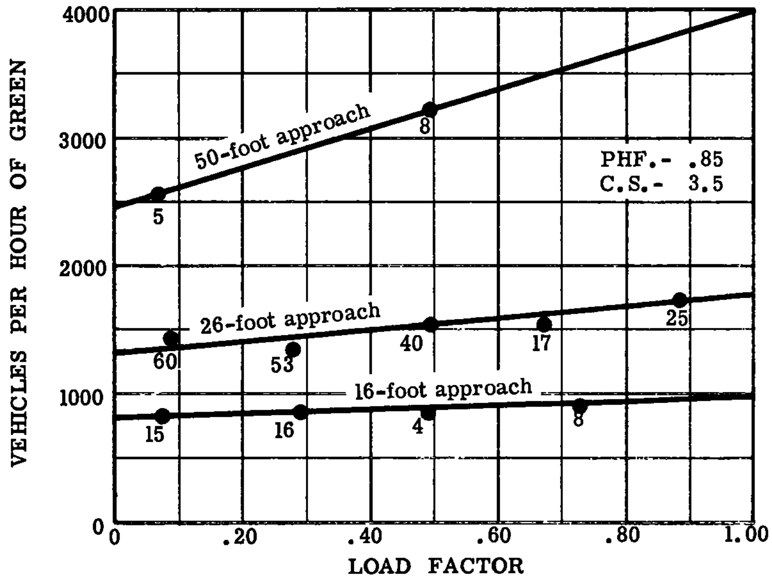

Figure 27. Effect of load factor on hourly intersection capacities, two-way streets with parking.

peak-hour factors and the city size are held constant. The outstanding characteristic of the results shown on these figures is that the lines representing the change in traffic flow with a change in load factor for the various widths of one-way streets are parallel (Figs. 23 and 25), whereas the lines tend to converge toward a common point for the two-way streets with the lines for the wider streets having a greater slope than the lines for the narrower streets (Figs. 26 and 27).

This means that for each type of one-way street, a specific change in the load factor will cause the same change in the traffic flow in terms of vehicles per hour regardless of the width of the street. For the two-way streets, a specific change in the load factor will cause a greater change in the flow on the wider streets than on the narrower ones.

In all cases, the load factor has a very marked effect on the traffic flow. The change in the volume of traffic on one-way streets, regardless of width, amounts to about 10 vehicles per hour of green period on the streets without parking for each change of 0.01 in the load factor. The corresponding figure for one-way streets with parking on one or both sides is 15 vehicles per hour of green. The effect of a change in the load factor on two-way streets where the change varies with the approach width is shown by Figures 28 and 29. The change is greater when there is no parking than with parking and much greater on the wider streets than on the narrower streets, neither of which was the case for one-way streets.

Figures 30 and 31 show the same information as Figures 26 and 27 plotted in a more usable form from which the effect of the load factor for any width of two-way street may be determined.

At this point it is well to refer to Figure 6 to obtain the complete significance of the curves shown in Figures 30 and 31. The curves of Figures 30 and 31 which represent load factors of zero show the highest hourly volumes than can be accommodated without traffic delays at signalized intersections being appreciably higher than at any lower volume. The volumes represented by the curves for a load factor of 0.00 are therefore certainly the minimum values that should be used for design or operation to obtain a very high level of traffic service. Any appreciable delays to traffic at these volumes must be charged to conditions other than the traffic load on the approaches to the intersections.

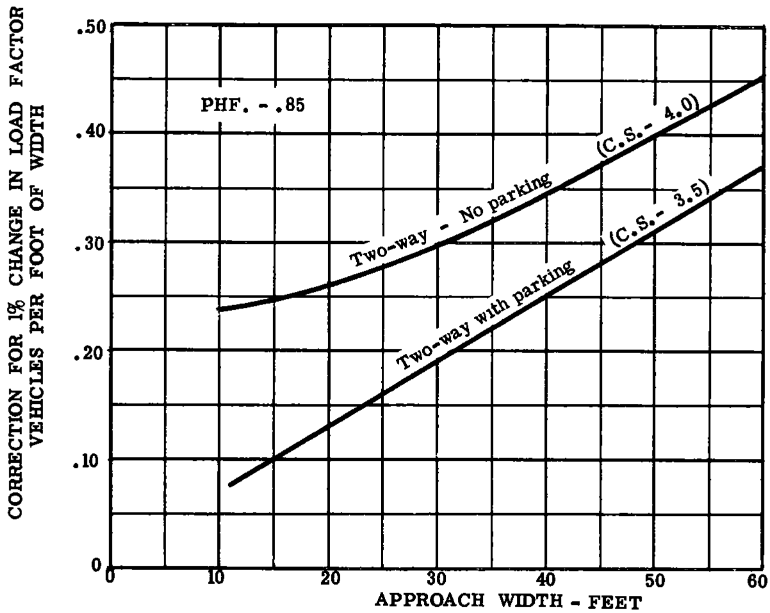

Figure 28. Chart for adjusting load factors to a common base.

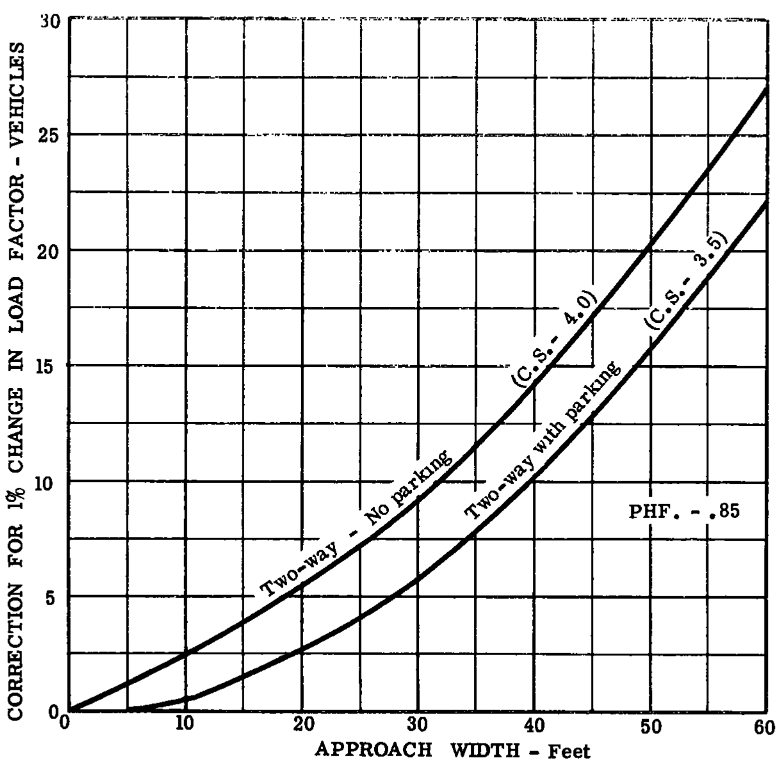

Figure 29. Total adjustment for a change in load factor.

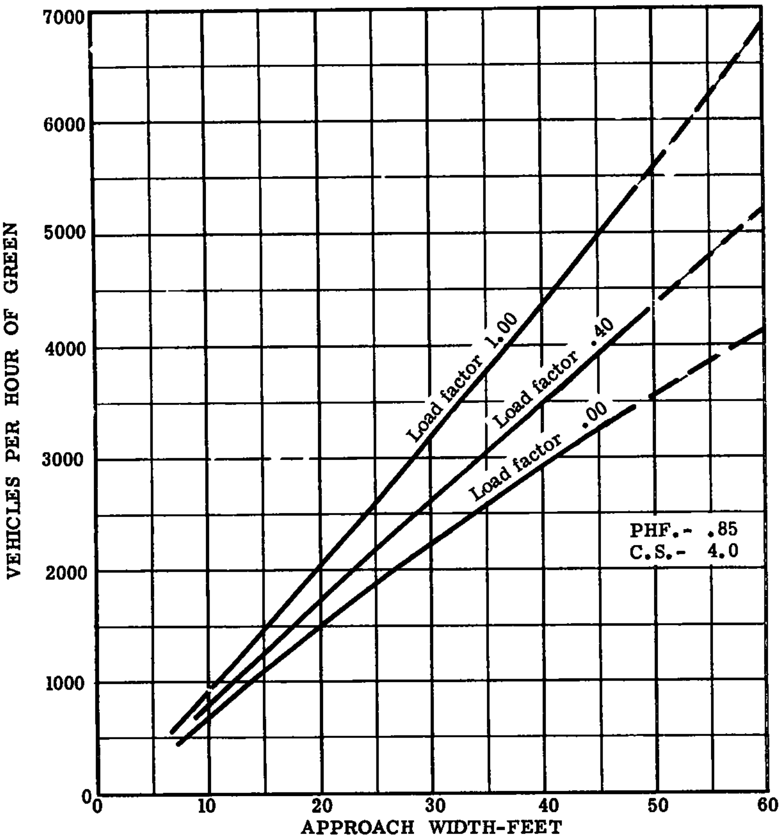

Figure 30. Effect of load factor on hourly intersection capacities, two-way streets, no parking.

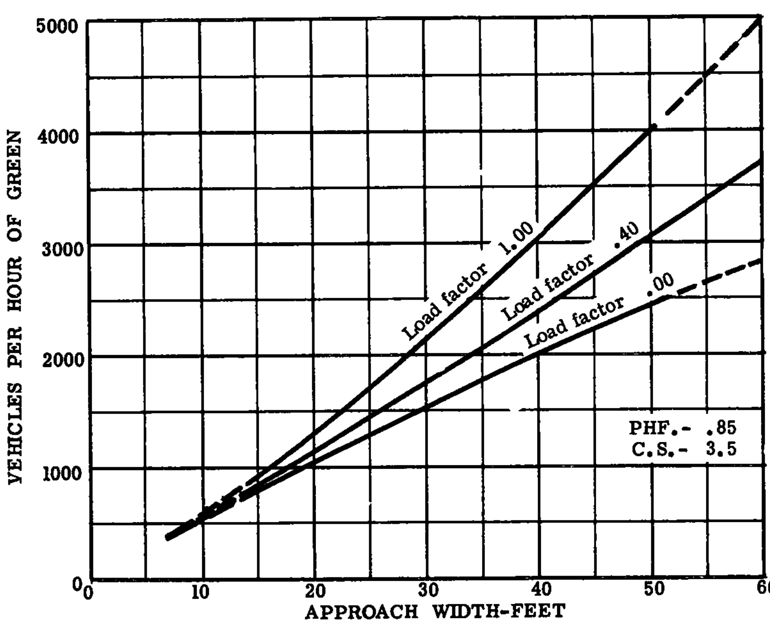

Figure 31. Effect of load factor on hourly intersection capacities, two-way streets with parking.

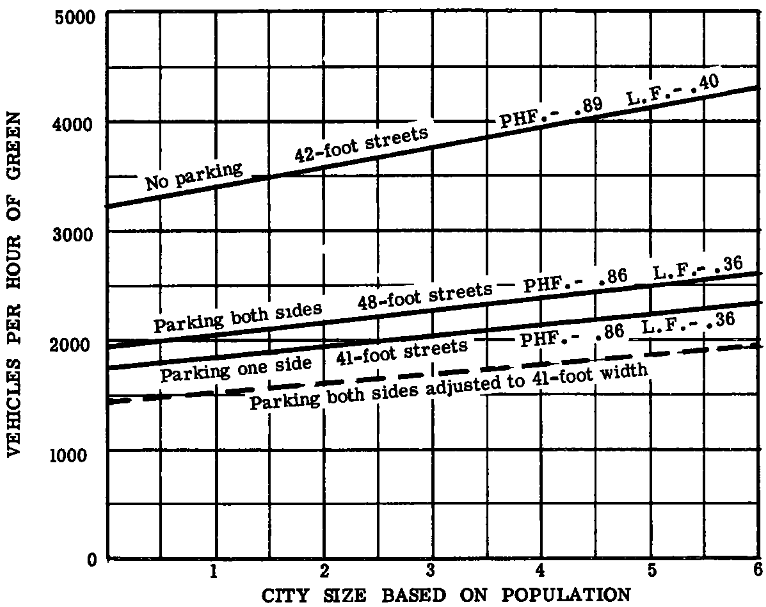

Figure 32. Effect of city size on hourly intersection capacities, one-way streets.

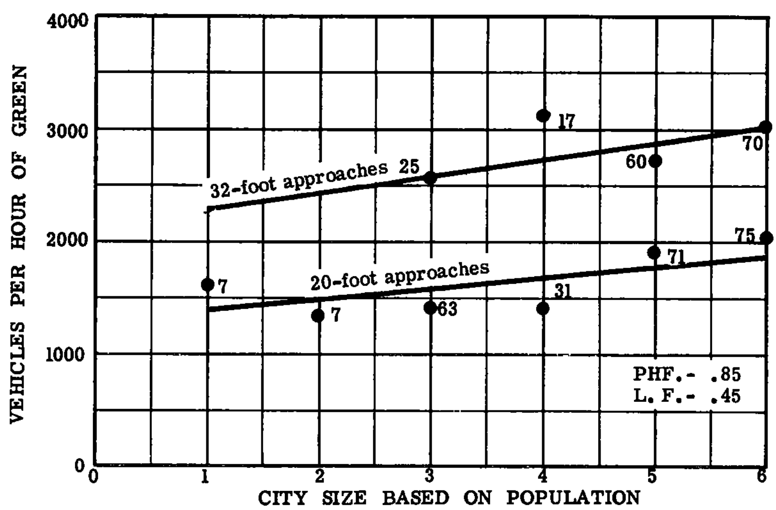

Figure 33. Effect of city size on hourly intersection capacities, two-way streets, no parking.

The curves representing a load factor of 1.00 (Fig. 30 and 31) also represent the maximum traffic flow that the various approach widths will accommodate regardless of the total traffic delay. In most cases, a load factor of 1.00 or approaching 1.00 can only be obtained with a continuous backlog of vehicles at the approach during the peak hour. With a properly coordinated signal system, fully responsive to the variations in traffic flow, load factors approaching 1.00 can be obtained without a continuous backlog of vehicles and with little more delay for the average vehicle than at lower traffic volumes. This is seldom accomplished at the present time. In fact, at the present time the most heavily loaded intersections selected for this study and scattered in cities

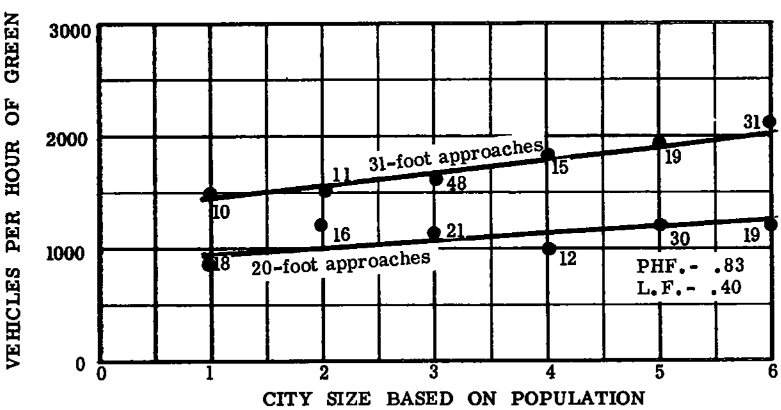

Figure 34. Effect of city size on hourly intersection capacities, two-way streets with parking.

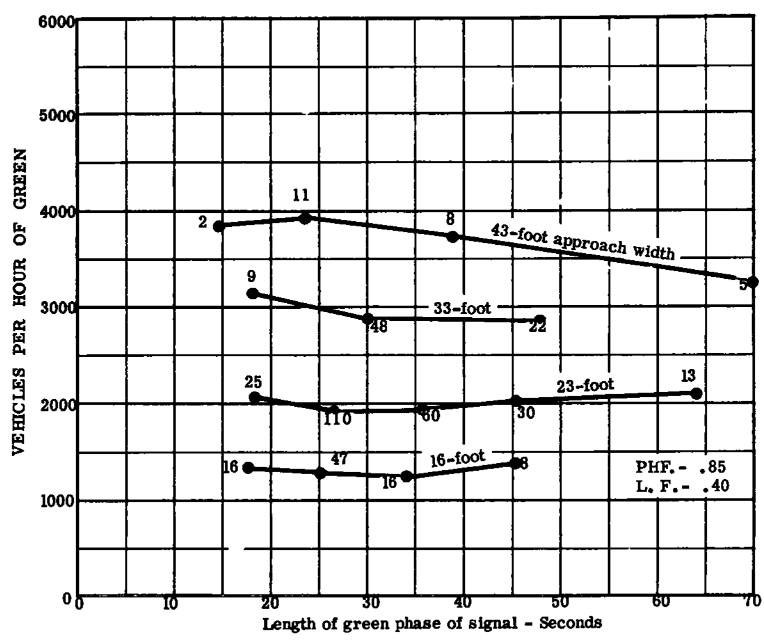

Figure 35. Effect of length of green phase on intersection capacity, two-way streets, no parking.

throughout the Nation were operating during the peak period at an average load factor of 0.40 which is the reason this specific curve was shown in Figures 30 and 31. Unless some major breakthrough occurs in traffic control, this curve certainly represents traffic volumes as high or higher than those that should be selected for design purposes if there is to be any improvement in traffic conditions in urban areas.

A whole series of curves similar to those in Figures 30 and 31 can be developed for different peak-hour factors and cities of different sizes with a knowledge of the effect of these factors on traffic flow at signalized intersections.

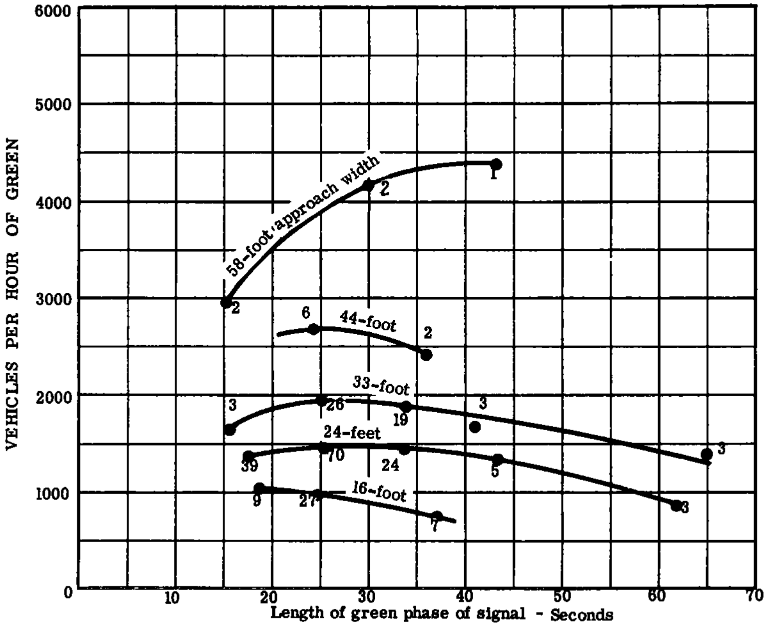

Figure 36. Effect of length of green phase on intersection capacity, two-way streets with parking.

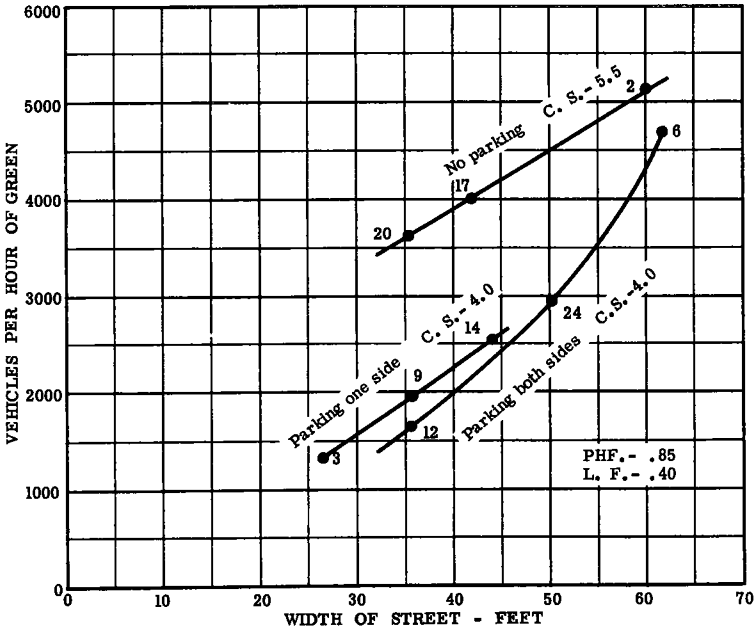

Figure 37. Capacity of intersection approaches on one-way streets.

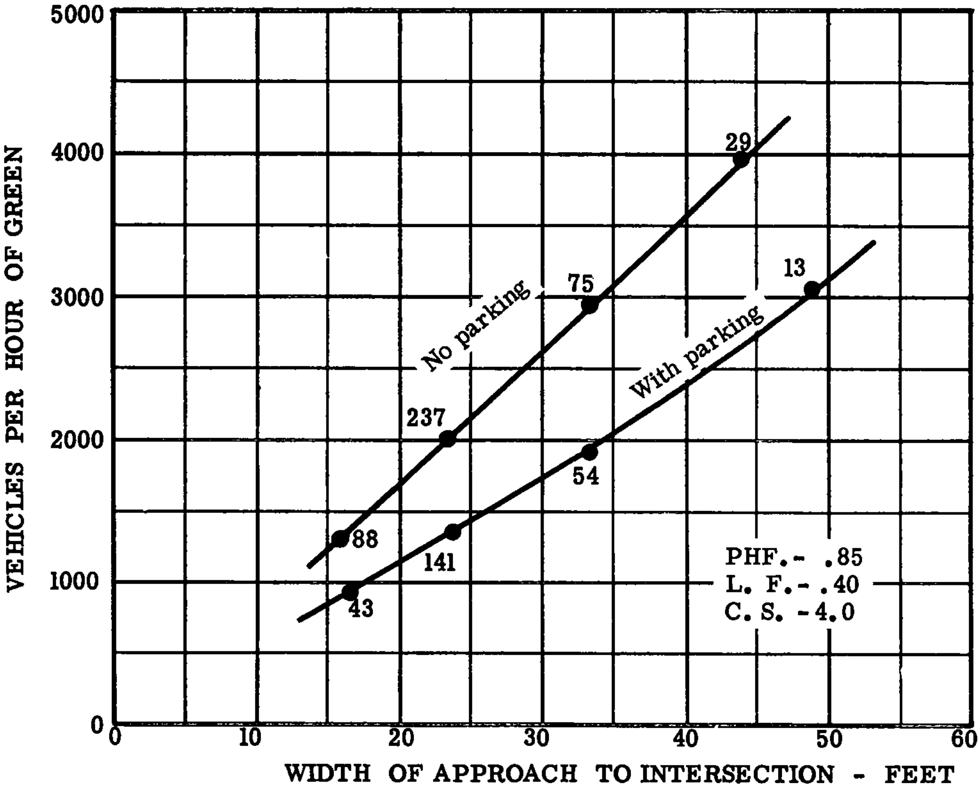

Figure 38. Capacity of intersection approaches on two-way streets.

EFFECT OF CITY SIZE

The effect of city size on the traffic-carrying capacity of an intersection located in that city was the most difficult of the primary variables to determine because the other primary variables (including the street width, peak-hour factor, and load factor) are also related to some extent at least to size of city. The effect of size as shown by Figures 32 through 34 is, therefore, over and above the effect that these other variables have on intersection capacities.

Size has been designated by numbers ranging from 1 through 6. These numbers represent the following city sizes:

Number	Population
1	Under 50,000
2	50,000 to 99,999
3	100,000 to 249,999
4	250,000 to 499,999
5	500,000 to 999,999
6	1,000,000 or more

It would probably have been more appropriate to use semilog paper for Figures 32 through 34 if the actual size of the city had been entered on the punch cards. The

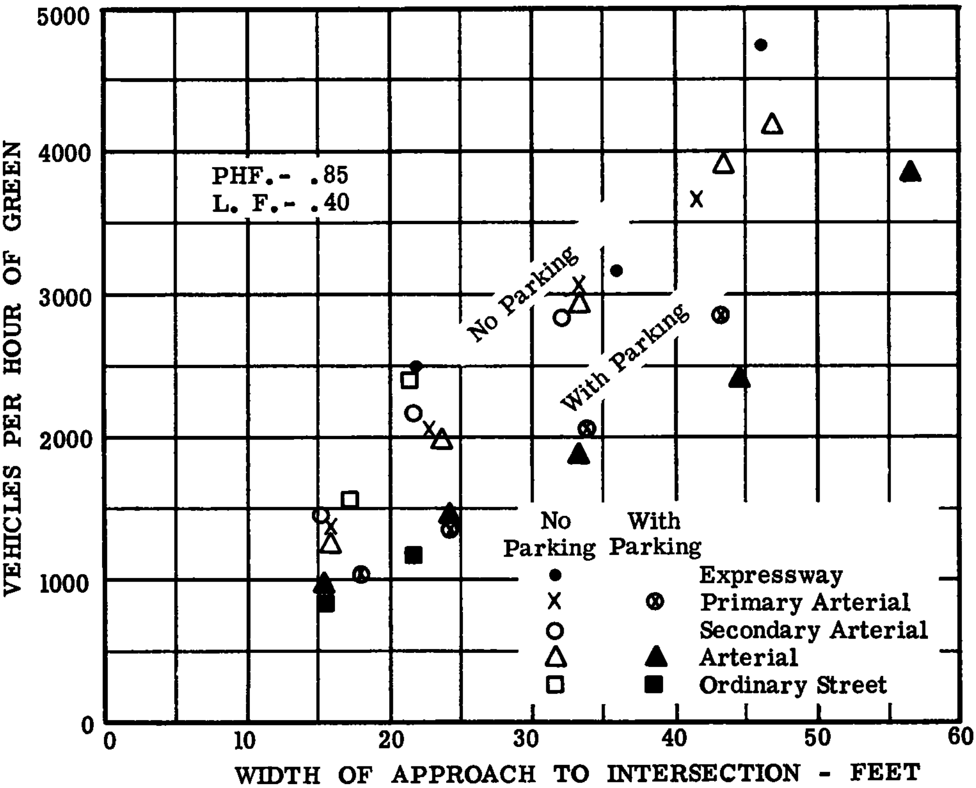

Figure 39. Capacity of intersection approaches, two-way streets by type of street.

average size of the cities as grouped, however, closely follows a logarithmic scale. In either case, the size of the city does have a very substantial effect on the traffic volumes that intersections on all types of streets will accommodate. The exact reason as to why the intersections in the larger cities accommodated more traffic than those in the smaller cities is not definitely known but the more common assumptions are (a) there are generally better traffic and pedestrian control measures in effect in the larger cities, and (b) drivers in the larger cities are more experienced in coping with high densities and congested traffic conditions than the drivers in the smaller cities.

The traffic volume that can be handled on an intersection in one city during the peak hour compared to that for an intersection in a larger or smaller city when all other conditions are the same, may be calculated by using the following equation:

$$VPH_2 = \left(\frac{14 + CS_2}{14 + CS_1}\right) VPH_1$$

in which

VPH_1 = known hourly volume for intersection in first city;
VPH_2 = hourly volume in second city;

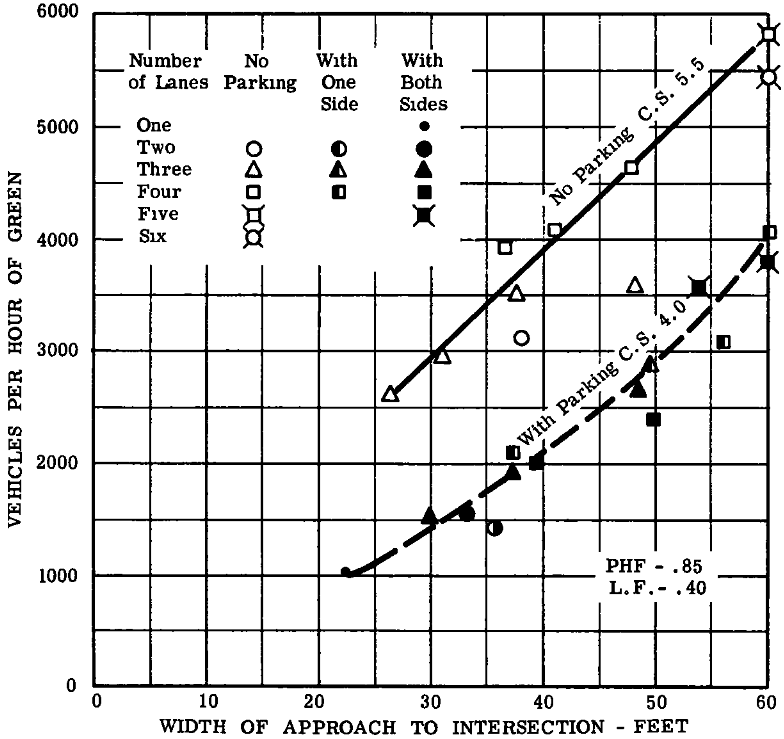

Figure 40. Effect of number of lanes, one-way streets.

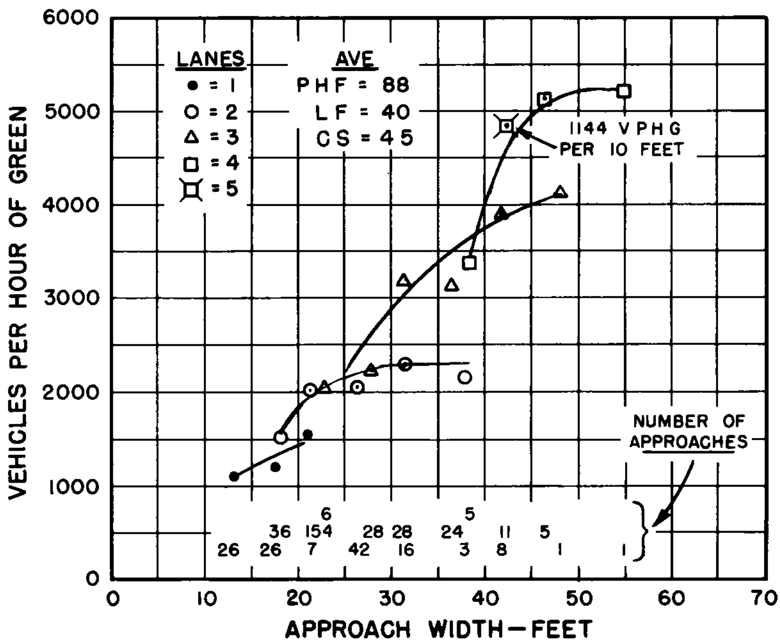

Figure 41. Effect of number of lanes, two-way streets, no parking.

CS_1 = size of first city;
CS_2 = size of second city (both city sizes being in terms of the code numbers used for this study).

For example, if an intersection approach in a city with a population of 162,000 can handle 500 vehicles per hour, an intersection with the same geometric features can be expected to accommodate about 560 vehicles per hour in a metropolitan area with 750,000 population $\left(\frac{14+5}{14+3} \times 500\right)$ providing traffic and other conditions are also the same.

EFFECT OF LENGTH OF GREEN PHASE

Figures 35 and 36 show for two-way streets the effect of the length of the green phases of the traffic signals on the traffic flows through intersection approaches in terms of the number of vehicles per hour of green time. The rate of flow per hour of green was obtained by expanding the flow as recorded during the green phases included in the peak 60 min to a full hour of green time.

On the two-way streets without parking (Fig. 36), there is little change in the rate of flow with green phases of different lengths. Any increase or decrease is not consistent between the different approach widths. On the two-way streets with parking, there is a general tendency for the rate of flow to increase as the length of the green phase is increased from 10 or 20 sec to 25 or 30 sec, depending on the street width, and then to decrease with any further increase in the length of the green phase. The exception is the curve for approaches 58 ft wide which continues to show an increase up to a green phase of 45 sec. This curve and also the curve for the 44-ft approach width are based on too few data to indicate a tendency that would be reliable enough to be applicable to other locations.

The results of this study are somewhat unexpected in view of the generally accepted practice of increasing the signal cycle to obtain higher capacities during peak traffic periods. These results do not necessarily condemn such a practice because some decrease in the flow during the green phases can be tolerated to reduce the percentage of amber time during the hour. For example, a peak of 1,450 vehicles per hour of green occurred on the 24-ft approach width (Fig. 36) when the green phase was 30 sec. With a green phase of 40 sec the vehicles per hour of green decreased to 1,400 VPHG. If a 60-sec cycle is assumed in the first case, a 77-sec cycle must be assumed in the second case to have two 4-sec amber periods and for the same ratio of green time in both cases between the intersecting roadways. With the 60-sec cycle, the total volume during a clock hour on the 24-ft approach would be 700 vehicles with 1,800 sec of green time, whereas with the 77-sec cycle the corresponding figure would be 727 vehicles with 1,867 sec of green time. The total delay to traffic at the intersection would depend on the peak-hour factor, the load factor, and the total traffic volume approaching the intersection during the peak hour. If the traffic volume approaching the intersection during the peak hour were under 700 vehicles, the total delays would be considerably greater with the 40-sec green period and 77-sec cycle than with the 30-sec green period and 60-sec cycle. At some approach volume considerably above 700 vehicles per hour, the total delay during the peak hour would under certain conditions become less for the 77-sec cycle than for the 60-sec cycle.

From the results of this study, it appears that the principal advantage of the use of green phases longer than 20 or 30 sec at individual locations results from the reduction in the percentage of the total time devoted to the amber phases and "all red" or "overlapping red" periods when they are necessary to clear the intersection of pedestrians or vehicles between the green phases. The longer green phases are also necessary at times to obtain the proper progression of traffic through a system of interconnected or coordinated signals. The disadvantages of the longer green phases as compared to the shorter green phases are (a) increased delays to traffic during periods when good progressive movement is not obtained and (b) fewer opportunities during the peak hour for vehicles that block the traffic movement in a lane to clear the intersection.

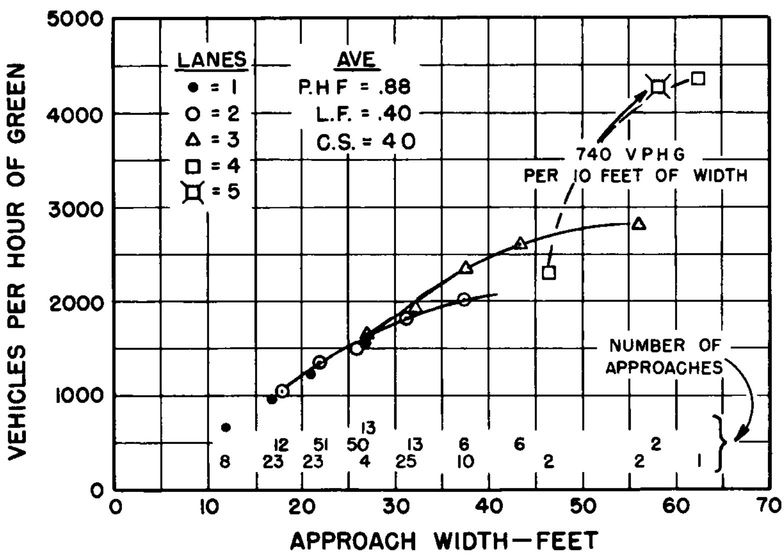

Figure 42. Effect of number of lanes, two-way streets with parking.

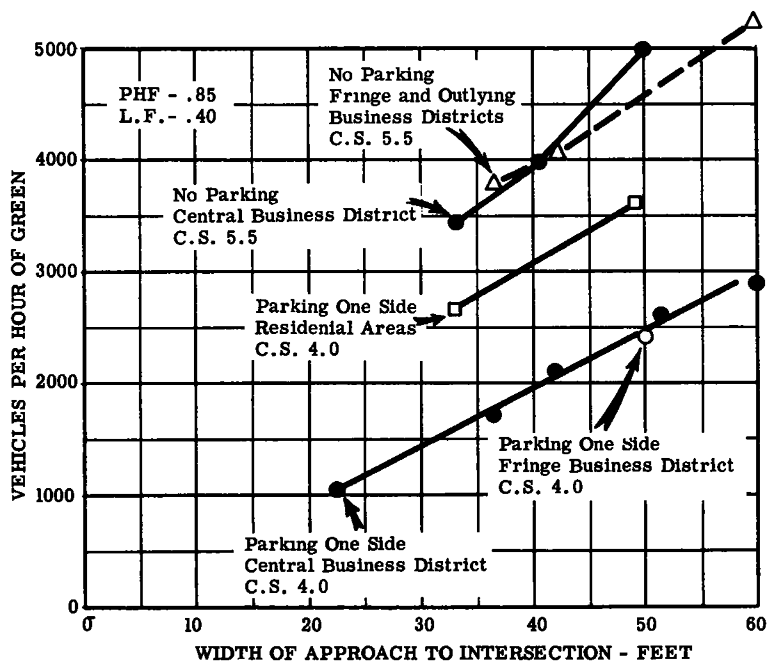

Figure 43. Capacity of intersections by area of city, one-way streets, no parking and parking one side.

EFFECT OF ON-STREET PARKING

The figures that have been presented thus far can be used to determine the effect of parking on one-way and two-way streets, but Figures 37 and 38 are more appropriate for this purpose. It is rather evident from Figure 37 that the sample of one-way streets

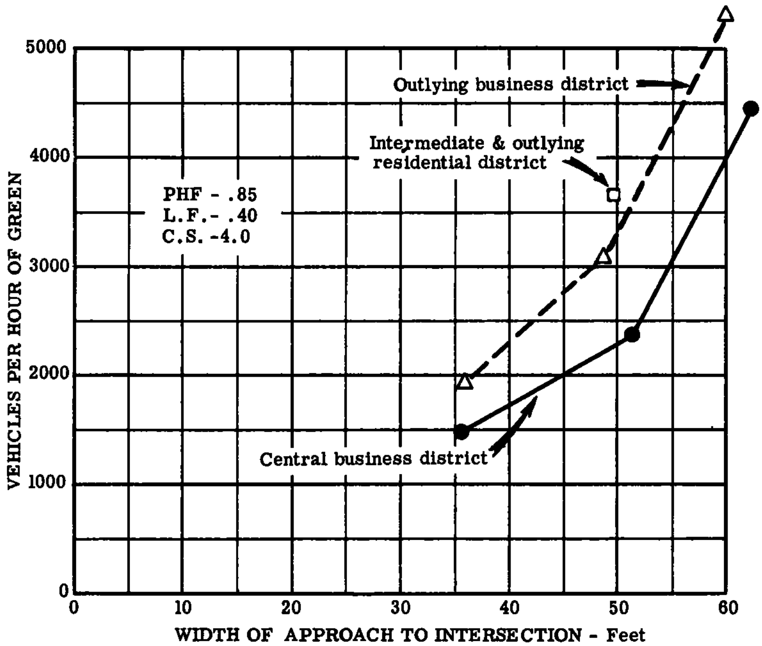

Figure 44. Capacity of intersections by area of city, one-way streets, parking both sides.

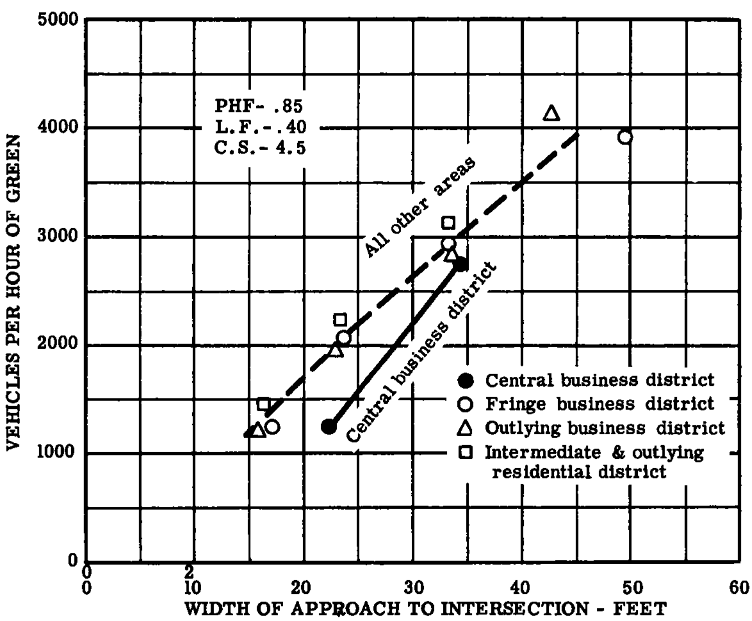

Figure 45. Capacity of intersections by area in city, two-way streets, no parking.

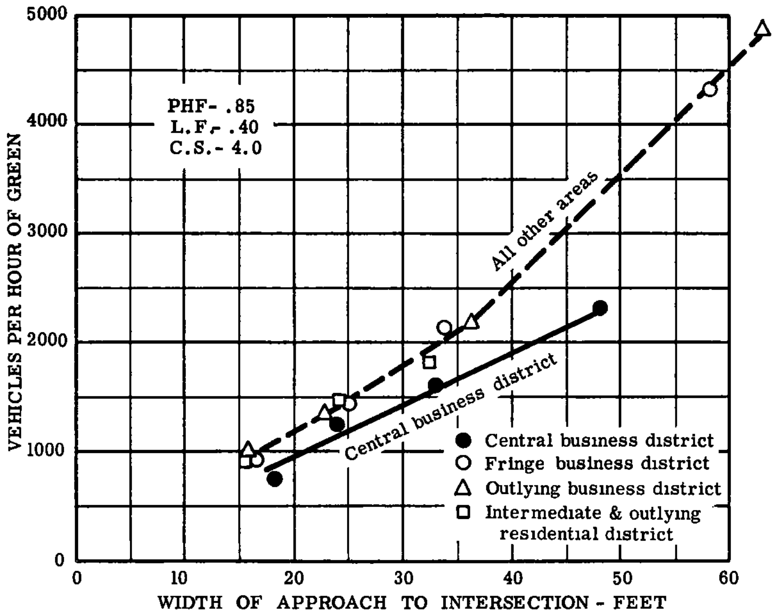

Figure 46. Capacity of intersections by area in city, two-way streets, parking both sides.

included in this study was too limited to obtain accurate values except for street widths within a range of 35 to 45 or 50 ft. The most accurate comparison can be made between the 40-ft widths which is as follows:

Parking	VPHG
None	3,550
One side	2,250
Both sides	2,000

Parked vehicles on one side of a one-way street 40 ft wide reduce its capacity 33.5 percent. The corresponding figure for parking on both sides is 43.6 percent. Fron another viewpoint, eliminating parking on one side of a one-way street 40 ft wide will increase its capacity only 12.5 percent whereas eliminating parking from both sides will increase its capacity 77.5 percent. Comparisons for other one-way street widths are not reliable because the data are not adequate to make such a comparison.

It is also evident that the one-way streets, especially those with parking on one side and the wider streets without parking, were not being operated in such a manner as to obtain anything like their potential capacities. Can it be true that the same effort is not being made through known traffic control procedures to obtain the potential capacities on these streets as on other types and widths of streets? Or is it too often assumed that one-way operation will solve a traffic problem and the street is then left to fare for itself? Two things are certain from the detailed studies that have been made of the data obtained for one-way streets: (a) there is a greater range in the traffic volumes carried by one-way streets with similar geometric and traffic characteristics when loaded to the same degree than for two-way streets, and (b) there is little or no advantage to one-way operation from a capacity viewpoint unless the one-way operation extends for a sufficient distance to obtain full utilization of the street's capacity. One-way operation for a few blocks may solve some of the problems at the intersections for the cross-streets but in such cases, the one-way streets cannot be expected to operate efficiently. There was an abnormal number of one-way streets included that were only a few blocks long that were connected at one end or the other with two-way streets of the same or a similar width.

The effect of parked vehicles on the capacity of two-way streets is shown by Figure 38. The results are considered reliable for approach widths of 15 to 45 ft. Parking reduced the capacity an average of about 30 percent regardless of the street width. It should be remembered, however, that parking is usually eliminated for some distance back from the crosswalk on most streets with parking and that more of the approaches of certain widths had the parking eliminated for a considerable distance to provide an additional usable lane than the approaches of other widths. The effect of this variable is covered later.

TYPE OF STREET BY SYSTEM

It was considered reasonable to assume that there might be some difference in the capacity of identical intersections on different types of streets. The type of street on which each intersection approach was located was therefore recorded during the field studies. Figure 39 indicates, however, that if the type of street or the street system made any difference, this fact was not apparent from the available data either for streets with or without parking, except possibly for the expressways at grade which show a slight tendency to be able to accommodate higher traffic volumes at the intersections than other facilities of the same width.

EFFECT OF NUMBER OF LANES

Traffic at intersection approaches of equal width sometimes operates in a different number of lanes at one location than at another. This is shown by Figures 40, 41, and 42.

One-Way Streets

Figure 40 shows that at the one-way streets with no parking the following obtained:

1. When traffic operated in four lanes on streets between 35 and 40 ft wide, the street accommodated, on an average, about 400 more VPHG than when the traffic was in three lanes, and about 800 more VPHG than when traffic was in two lanes.
2. Streets between 45 and 50 ft wide accommodated 1,050 more VPHG or nearly one-third more traffic when the vehicles were in four lanes at the intersection than when they were in three lanes.
3. For widths of 60 ft, five lanes accommodated somewhat more traffice than six lanes.

In considering the effect of the number of lanes for one-way streets with parking (Fig. 40), the elimination of parking ahead of the crosswalk must be considered. It is quite obvious that three lanes of traffic on a 30-ft street or four lanes of traffic on a 40-ft street could not have been accommodated at an intersection approach unless parking had been eliminated for a considerable distance ahead of the crosswalk. The data for the one-way streets were too meager to arrive at any extensive conclusions, but in general the streets where parking had been eliminated only near the intersection to permit traffic to operate in one additional lane did not, in most cases, accommodate substantially higher volumes than other streets of the same width but with traffic in one fewer lane on the approach. The curves in the figure represent, however, the minimum volumes that should be accommodated if the streets of specific width are divided into the most appropriate number of lanes.

Two-Way Streets

Sufficient data were recorded for the two-way streets to develop some interesting facts relative to effective street widths and their division into lanes. The results for two-way streets without parking (Fig. 41) show that streets of various widths accommodate more traffic when they operate with the following number of traffic lanes than with any other number of lanes:

Approach Width (ft)	Number of Traffic Lanes
Below 14	1
15 to 22	2
23 to 35	3
36 to 50	4

The traffic accommodated by the more efficient approach widths under average conditions with a peak-hour factor of 0.88 and a load factor of 0.40 on two-way streets without parking may be expressed by the following equation:

$$\text{VPHG} = (\text{Approach width in feet} - 5\text{ ft})\ 130$$

The average rate was considerably lower than this for approaches that were 15 ft, and 35 to 40 ft wide regardless of the number of lanes. There is some doubt, therefore, that these approach widths should be constructed or provided through line markings, except for unusual traffic conditions such as when there are either no commercial vehicles or an exceptionally large percentage of commercial vehicles during the peak hours. Approach widths of 35 to 40 ft, for example, might be very efficient when operating as four lanes with no commercial vehicles or as three lanes with an exceptionally large number of commercial vehicles. Lane lines must be well marked to obtain even reasonably efficient operation under the following conditions: (a) two lanes of traffic on widths under 20 ft; (b) three lanes of traffic on widths under 30 ft, and (c) four lanes of traffic on widths under 40 ft. A more detailed discussion of the effect of well-marked lane lines is presented later.

The intersection approaches on two-lane streets with parking that were of the more efficient widths accommodated average traffic volumes during the peak hours which may be expressed by the following equation when the peak-hour factor is 0.88 and the load factor 0.40 (Fig. 42):

$$\text{VPHG} = (\text{Approach width in feet} - 5\text{ ft})\ 78$$

This is 60 percent of the traffic accommodated by streets of equal width without parking. The number of lanes in which traffic was operating on the approach had a far less effect on the total peak-hour volume than for two-way streets without parking. This suggests that the midblock conditions have a very substantial effect on traffic flow, regardless of the number of lanes, on the intersection approach. For example, approach widths between 25 and 30 ft wide accommodated an average of about 1,500 vehicles per hour of green regardless of whether traffic entered the intersection from one, two, or three lanes. To obtain three-lane operation with widths of 25 to 30 ft, parking was prohibited on the approach for some distance ahead of the crosswalk, whereas this was not necessary for one- or two-lane operation. Likewise, parking had to be eliminated ahead of the crosswalk to obtain two-lane operation on widths under 25 ft. The data available also indicate that approaches between 40 and 48 ft wide on streets with parking are less efficient per foot of width than the wider or narrower approaches. Because approaches of this width generally occur only on two-way streets wider than 80 ft, the sample of such intersections included in this study was too small to be able to place any reliability in a general conclusion based on this statement.

LOCATION WITHIN A CITY

Each intersection included in this study was classified by the area of the city in which it was located. The five different location classifications were as follows:

1. Central business district;
2. Fringe of central business district;
3. Outlying business district;
4. Intermediate residential area; and
5. Outlying residential area.

Some intersections on two-way streets with and without parking were located in all of

TABLE 2

EFFECT OF RAIN ON INTERSECTION CAPACITIES (FROM DIRECT COMPARISONS)

Two-Way Streets: No Parking, Approach Width (ft)	No Parking, VPH of Green, Clear	No Parking, VPH of Green, Rain	No Parking, Percent Change	With Parking, Approach Width (ft)	With Parking, VPH of Green, Clear	With Parking, VPH of Green, Rain	With Parking, Percent Change	One-Way Streets with Parking, Approach Width (ft)	One-Way, VPH of Green, Clear	One-Way, VPH of Green, Rain	One-Way, Percent Change
16	1,310	1,100	-16	17	1,070	810	-24	36	1,460	1,490	2
17	1,180	990	-16	18	1,260	1,240	- 2	36	1,190	1,230	3
20	2,380	2,200	- 8	24	950	900	- 5	36	1,050	1,040	- 1
20	3,460	2,750	-21	24	1,490	930	-38	36	1,700	1,530	-10
21	1,980	1,780	-10	24	1,910	1,580	-17	36	1,440	1,470	2
21	2,720	2,420	-11	38	3,790	2,410	-36	36	1,520	1,190	-22
24	1,060	1,130	8	38	1,560	1,370	-12	50	1,870	1,720	- 8
24	1,970	1,670	-15					50	1,630	1,590	- 3
24	2,750	2,010	-27					50	1,860	1,790	- 4
24	2,500	1,360	-46					50	2,300	1,990	-15
30	4,740	2,990	-37					50	2,480	2,040	-18
30	3,790	1,910	-50					50	1,740	1,730	- 1
Total	29,840	22,310	-249		12,030	9,240	-134	50	1,450	1,130	-22
Avg			-20 8				-19 1		21,690	19,940	-97
Weighted avg			-25 3				-23 2				- 7 5
											- 8 1

TABLE 3

AVERAGE VALUES FOR INTERSECTION APPROACHES WHERE EFFECT OF RAIN WAS STUDIED BY DIRECT COMPARISONS

Type of Average Value	Two-Way Streets, With 16- to 24-ft Approach Width, No Parking, Clear	No Parking, Rain	With Parking, Clear	With Parking, Rain	With 30- to 38-ft Approach Width, No Parking, Clear	No Parking, Rain	With Parking, Clear	With Parking, Rain	One-Way Streets with Parking, With 36-ft Approach Width, Clear	Rain	With 50-ft Approach Width, Clear	Rain
No of approaches	10	10	5	5	2	2	2	2	6	6	7	7
Width (ft)	21	22	21	21	30	30	38	38	36	36	50	50
Left turns (%)	7	6	12	7	21	0 5	2	0 5	11	8	8	15
Right turns (%)	13	11	2	8	20	9	36	0 5	11	15	12	4
Commercial (%)	6	8	5	10	10	8	10	7	11	7	5	7
Peak-hour factor	0 86	0 87	0 91	0 84	0 84	0 87	0 76	0 84	0 87	0 84	0 89	0 81
Load factor	0 32	0.40	0 57	0 41	0 74	0 22	0 41	0 18	0 14	0 12	0 17	0.17
VPH of green	2,131	1,741	1,340	1,090	4,270	2,450	2,680	1,890	1,390	1,330	1,900	1,710
Percent change	--	-18 3	--	-18 6	--	-42 6	--	-29 5	--	-4 3	--	-10 0

TABLE 4

EFFECT OF LOCAL BUSSES ON TWO-WAY STREETS

Type of Parking	Load Factor	Peak-Hour Factor	City Size	No of Lanes at Crosswalk	Avg Approach Width (ft)	Without Busses, No of Approaches	Without Busses, Avg per Hour Vol (VPHG)	With Busses, No of Approaches	With Busses, Avg. per Hour Vol (VPHG)	With Busses, Avg No Busses	Bus Equiv. in Passenger Cars
Without	0 40	0 85	5 0	1	16	29	1,300	29	1,180	24	6 0
				2	22	72	2,100	161	1,950	39	4 8
				3	32	29	3,007	65	2,957	47	2 1
				4	42	4	4,083	17	4,030	64	1 8
With[1]	0 40	0 85	3 5	1	20	22	1,250	36	1,200	24	3 1
				2	26	25	1,624	123	1,490	30	5 5
				3	32	8	2,170	30	1,845	31	11 5

[1] Parking eliminated only at bus stop

these areas but no data were available for one-way streets under the following conditions:

1. No parking in residential area;
2. Parking on one side in outlying business district; and
3. Parking on both sides at fringe of business district.

One-Way Streets

For the one-way streets without parking (Fig. 43), there was no definite indication that intersection capacities were significantly different in central, fringe, or outlying business districts. There were no data for these streets in residential areas.

For the one-way streets with parking on one side (Fig. 43), intersection capacities were about the same in the central and fringe business districts. In both of these areas, the capacities of such streets were much lower than in the residential areas. In the central and fringe business districts they handled 34 to 40 percent less traffic than in residential areas with the greater difference percentage-wise being on the narrower streets. No data were available for one-way streets with parking on one side in outlying business districts.

For one-way streets with parking on both sides (Fig. 44), intersection capacities, on an average, were about 10 percent lower in the outlying business districts than in the residential areas. In the central business districts, they were 25 to 30 percent lower than in the residential areas.

Two-Way Streets

Intersection capacities for two-way streets, both with and without parking (Figs. 45 and 46), were, on an average, about 20 percent lower in the central business districts than in other areas of the cities. There was also some tendency for the two-way streets without parking to accommodate more traffic in the residential areas than in fringe or outlying business districts, but the difference was too small to make a distinction between these areas in traffic capacity determinations for two-way streets.

There are several reasons for the lower capacities in the central business districts than in other areas of the city. Two of the more important ones are (a) a greater frequency of vehicles stopping to load or unload passengers and (b) more pedestrians causing interferences to vehicular traffic. The latter cause can be further investigated with the data available. This will be in conjunction with the use and effect on capacity of separate pedestrian signals, separate pedestrian phases, and the "scramble" system in a subsequent report.

The fact that intersections with like geometric features are able to accommodate considerably higher peak-hour volumes when located in certain sections of a city than when located in the central business district makes it especially important that the curves thus far presented be modified in an effort to obtain a more accurate comparison of the relative capacities of one-way and two-way streets. This can be accomplished only after the effect of most of the other variables has been investigated.

EFFECT OF RAIN ON INTERSECTION CAPACITIES

Some 200 intersection approaches were studied during inclement weather conditions including periods while it was raining or snowing or while the streets were wet or covered with snow. None of these data has thus far been used in this analysis.

A detailed review of data for inclement weather conditions revealed that for 32 of the locations where rain occurred during the peak hour, repeat studies were conducted during fair weather conditions. The results obtained by comparing the hourly volumes through each of the 30 intersections during the rainy periods with the fair weather conditions are given in Table 2. Two-way streets with and without parking, and one-way streets with parking are included. No direct comparisons were obtained for one-way streets without parking.

The intersections on the two-way streets carried an average of 20 percent less traffic when rain occurred during the peak hour than for the fair weather condition. The reduction, on an average, was about the same for the two-way streets with parking as for those without parking. The corresponding figure for the one-way streets was 7.5 percent with only 5 of the 13 one-way streets being affected to any appreciable extent. There was also a tendency for the intersections on the wider streets, both one-way and two-way, to be affected more on a percentage basis than the narrow streets. The reduction due to the rain was therefore somewhat greater based on the weighted averages (by street width) than for the unweighted averages; 24 percent for the intersection approaches on two-way streets and 8 percent for those on the one-way streets.

There was a large variation in the effect of rain at the different locations but this variation was probably no greater than the difference in the rainy conditions that occurred. These varied from a light drizzle or wet pavement for a few minutes during the peak hour to a continuous light rain for the entire hour. Accurate information as to the exact conditions during the rainy periods is not available but there is no indication that a heavy downpour occurred for any extended period of time at any of the locations while the studies were in progress. A heavy downpour over an extended period of time would probably have caused a much greater reduction in the traffic flow. Also (Table 3), the lower volumes accommodated during the rainy periods as compared with the fair weather conditions were not the result of a lower traffic demand or a difference in other conditions (such as an increase in turning movements or in the percentage of commercial vehicles) which would also have had a tendency to reduce the traffic flow during the peak hours. A further analysis of the data for all the locations where inclement weather occurred might be desirable, but, if so, this can only be done on a basis of comparing average values for similar intersections, using the entire mass of data for each weather condition.

EFFECT OF LOCAL BUSSES

Local busses were operating on about 70 percent of the two-way streets on which the intersection approaches included in this study were located. The local bus volume at most of these locations was in the neighborhood of 2 percent of the total traffic during the peak hours. The number of local busses varied from an average of 24 per hour in the one direction on the narrower streets to 64 per hour on the wider streets with four traffic lanes for the one direction of travel. No attempt has been made to determine the effect of local busses on one-way streets or to relate the change in bus equivalents with a change in the number of busses on specific widths of streets in view of the limited data for this purpose.

Table 4 gives the results of the study to determine the effect of local busses in terms of the equivalent number of passenger cars. The bus equivalent varies for the two-way streets without parking from 6.0 when there is only only traffic lane to 1.8 when there are four traffic lanes. On the two-way streets with parking, the bus equivalent increased with an increase in the number of lanes; 3.1 on the streets with one lane to 11.5 for streets with three lanes for traffic in the one direction. Parking was always estimated at the bus stop on the streets with parking. This accounts in a large measure for the differences between the bus equivalents on the two-way streets with and without parking, especially when the following conditions are considered:

1. A bus while loading is usually out of the normal traffic lane on a street with parking and at least some of the right-turning vehicles can use the bus stop when no bus is present, thus providing an added street width part of the time. This is not true for streets without parking and only one lane for each direction of travel.
2. On the wider streets with parking, a bus in entering and leaving a bus stop interferes with traffic in lanes other than the one the bus occupies. This is not necessary on the streets without parking.

Locations where no local busses stopped during the peak hour to load or unload passengers, as well as near- and far-side bus stops and intersection approaches with both near- and far-side bus stops, were included in the preceding analysis. The number and percentage of locations for each each of these conditions are given in Table 5.

The data for the two-way street intersections that had two traffic lanes for the one direction of travel contained the largest sample and were therefore used to determine the relative advantage of near- and far-side bus stops. The results are given in Table 6 for the two-way streets. These results show that the busses cause less interference to other traffic if the stop is located at the far side on the streets without parking and at the near side on streets with parking.

TABLE 5

DISTRIBUTION OF LOCATION OF LOCAL BUS STOPS

Bus Stop	Intersection Approaches for Two-way Streets			
	With No Parking		With Parking	
	Number	Percent	Number	Percent
None [a]	75	27.5	52	27.5
On near side	142	52.3	109	57.7
On far side	44	16.2	22	11.6
On both sides	11	4.0	6	3.2
Total	272	100.0	189	100.0

[a] No passenger stop made by local busses during peak hour.

OTHER FACTORS AFFECTING CAPACITY

There still remain for analysis several variables that have an extremely important effect on intersection capacities. These include right turns, left turns, commercial vehicles, type of signal control; effect of separate pedestrian signals and pedestrian intervals; and the use of three- and four-phase control together with scheduling the movements during each phase and the sequence of the different phases. Some exploratory work has been done in all these areas using the extensive data obtained during this study, but the preliminary results in some cases contradict established traffic engineering practices to such a degree that further analyses are needed or desirable before their publication. As a few examples of the less controversial items, the preliminary analyses show that under certain conditions the following obtain:

1. An increase in the right-turning movements will improve the traffic flow through an intersection, especially when there are three traffic lanes on the approach.
2. At many intersections where three phases are being used, the third phase is not only unnecessary but hinders rather than improves the smooth and safe flow of traffic.
3. Traffic lane lines in good condition are far more necessary at certain locations than at other locations. In fact, in certain instances, even when applied in the most correct manner, they reduce capacities without improving safety.
4. The "scramble system" for pedestrians not only reduces the time available for vehicular movement but also increases pedestrian delays and pedestrian inter-

TABLE 6

EFFECT OF BUS STOP LOCATION ON BUS EQUIVALENTS

Item	No Parking		With Parking	
	Near-Side Stop	Far-Side Stop	Near-Side Stop	Far-Side Stop
No. of traffic lanes at crosswalk	2	2	2	2
Approach width (ft)	22	22	26	26
No. of locations	85	24	78	16
Traffic volume during PH (VPHG)	1,854	2,119	1,499	1,406
Avg. no. of busses during PH	41	34	40	37
Bus equivalent in passenger cars	7.0	1.0	4.1	7.0

ferences to traffic so that less traffic capacity is available during the shorter available periods for traffic movement.

There is such a wealth of information available in the data that have been compiled for this project that every effort should be made to analyze it to the maximum extent possible in an effort to obtain reliable information on which to base scientific traffic engineering practices for improving transportation in urban areas. There must also be developed a new basic family of curves for use in the design of intersections and for capacity determinations. These apparently will not invalidate any previous work that has been based on the Highway Capacity Manual published in 1950 but will place the entire procedure on a more scientific basis.

USE OF LOAD FACTOR RATIOS IN DETERMINING EFFICIENCY OF SIGNAL OPERATION

The most efficient movement of traffic and the least total delay occurs at an intersection when the two approaches carrying the major cross-movements are loaded to their same relative capacities. An excessive delay should not be encountered by traffic on one approach while there is little or no delay to traffic on the intersecting approach or approaches. The load factors obtained for the various approaches for the capacity analysis offer a means of determining the efficiency of a traffic signal in allocating time between the intersecting flows. By dividing the highest load factor for any of the approaches at an intersection into the highest load factor for the intersecting street, a ratio may be obtained which is called the peak-hour "load factor ratio" between the approaches. This ratio cannot exceed 1.00 but increases in magnitude with an increase in the efficiency of the signal in allocating the time between the two approaches. This may not be true at locations where a major street intersects a minor street because in such a case the least delay occurs if the signal is set to favor the major facility. Nearly all (95 percent) of the locations included in this study were, however, at the intersection of two major arterials.

TABLE 7

DISTRIBUTION OF LOAD FACTOR RATIOS FOR ALL TYPES OF INTERSECTING STREETS

Load Factor Ratio Between Approaches		Intersection with Data Available		Peak-Hour Factor	
Group	Avg	No	Percent	Avg Highest at Intersection	Avg Ratio Between Approaches
0 01 - 0 09	0 02	43	16 0	0 87	0 92
0 12 - 0 18	0 15	16	6 0	0 90	0 93
0 20 - 0 29	0 24	17	6 3	0 87	0 91
0 30 - 0 39	0 34	27	10 1	0 87	0 92
0 40 - 0 49	0 45	30	11.2	0 88	0 93
0 50 - 0 59	0 54	24	9 0	0 86	0 93
0 60 - 0 69	0 65	31	11 6	0 86	0 94
0 70 - 0 79	0 75	21	7 8	0 88	0 93
0 80 - 0 89	0 85	26	9 7	0 91	0 92
0 90 - 0 99	0 95	33	12 3	0 90	0 94
Total		268	100 0		

There were 268 intersections included for which complete data regarding the intersecting movements are available. At the other intersections, both streets did not carry traffic volumes of sufficient magnitude to load at least one approach on each street so as to produce a load factor of about 0.10 or higher. In such cases only the data for the one approach with a load factor of 0.10 or above were included and therefore the "load factor ratio" for the intersection cannot be calculated. In certain instances, the signal cycle was also changed from its normal setting for this study in order to obtain a high load factor on one of the approaches. The traffic volumes on the approaches carrying the cross-traffic were in such instances too low to be used for the capacity analysis so these intersections are also not included in the 268 for which complete data for the cross-movements are available.

Table 7 shows that the load-factor ratios for the 268 intersections were almost uniformly distributed over the widest possible range. There were just as many intersections with a poor adjustment of the signals for peak-hour traffic, resulting in a load-factor ratio under 0.09, as there were intersections with the best adjustment of the signals. A load-factor ratio of 0.09, for example, means that eleven times as many of the signal cycles on one approach were loaded as on another approach carrying cross-traffic. A further condensation of this table shows that at 37 percent of the intersections the load-factor ratio was 0.4 or less, at another 37 percent it was between

TABLE 8

DISTRIBUTION OF LOAD FACTOR RATIOS BY TYPE OF INTERSECTING STREETS

Load Factor Ratio Between Approaches		Intersection with Data Available		Avg. Highest Load Factor at Intersection	Peak-Hour Factor	
Group	Avg.	No.	Percent		Avg. Highest at Intersection	Avg. Ratio Between Approaches
Intersection of One-Way Streets						
0.00 - 0.07	0.01	18	41.9	0.16	0.88	0.94
0.10 - 0.15	0.15	1	2.3	0.13	0.88	0.90
0.20 - 0.28	0.25	4	9.3	0.35	0.90	0.89
0.32 - 0.34	0.34	3	7.0	0.27	0.84	0.96
0.41 - 0.49	0.44	3	7.0	0.34	0.86	0.92
0.50 - 0.56	0.53	3	7.0	0.32	0.92	0.93
0.66 - 0.69	0.67	3	7.0	0.68	0.94	0.94
0.71 - 0.77	0.74	2	4.6	0.46	0.88	0.94
0.81 - 0.89	0.85	4	9.3	0.59	0.91	0.91
0.92 - 0.99	0.92	2	4.6	0.30	0.93	0.94
Total	0.32	43	100.0			
Intersection of Two-Way Streets						
0.00 - 0.09	0.02	22	11.6	0.32	0.85	0.90
0.12 - 0.18	0.14	12	6.3	0.60	0.91	0.94
0.20 - 0.29	0.23	11	5.8	0.52	0.87	0.90
0.30 - 0.39	0.34	17	9.0	0.51	0.87	0.93
0.40 - 0.49	0.45	24	12.6	0.56	0.88	0.92
0.50 - 0.59	0.54	15	7.9	0.68	0.85	0.94
0.60 - 0.69	0.64	26	13.7	0.65	0.84	0.94
0.71 - 0.79	0.75	17	8.9	0.61	0.88	0.92
0.80 - 0.89	0.85	18	9.5	0.71	0.91	0.92
0.90 - 0.99	0.96	28	14.7	0.74	0.89	0.94
Total	0.53	190	100.0			
Intersections of a One-Way and a Two-Way Street						
0.00 - 0.09	0.02	3	8.6	0.50	0.92	0.89
0.10 - 0.15	0.15	3	8.6	0.50	0.88	0.93
0.26 - 0.29	0.26	2	5.7	0.61	0.81	0.97
0.30 - 0.38	0.34	7	20.0	0.45	0.88	0.90
0.42 - 0.49	0.46	3	8.6	0.51	0.90	0.98
0.50 - 0.58	0.52	6	17.1	0.39	0.84	0.90
0.67 - 0.69	0.68	2	5.7	0.73	0.93	0.93
0.72 - 0.77	0.74	2	5.7	0.70	0.89	0.94
0.81 - 0.85	0.83	4	11.4	0.62	0.92	0.94
0.91 - 0.96	0.93	3	8.6	0.71	0.91	0.93
Total	0.48	35	100.0			

0. 4 and 0. 8, and at only 26 percent of the locations was the load-factor ratio 0. 8 or higher during the peak hours.

These figures illustrate the tremendous possibility of improving traffic flow or reducing delays at intersections within urban areas through methods and equipment which will give a better allocation of the green signal time between traffic on intersecting streets. The results would have been even more astonishing had not most of the intersection approaches where load factors under 0. 10 were recorded been excluded from this capacity analysis. There is general agreement that it is easier to achieve the proper allocation of the green signal time at the intersection of one-way than two-way streets. Table 8 (cols 1 and 4), however, shows that such a possible achievement was not accomplished in actual practice. The fifth column does show, however, that there was some tendency to obtain a better allocation of the green time between approaches at the most heavily loaded intersections. All intersections selected were heavily loaded.

Columns 6 and 7 in Table 8 also show that the "peak-hour factors" and the "ratio" between the peak-hour factors on intersecting approaches at the same intersection did not have a tendency to change with a change in the load factor ratio. This means that an improvement in the allocation of green time between intersecting approaches may have changed the magnitude of the two traffic flows through the intersection but did not change the patterns of the flows during the peak hour.

It is of interest to investigate the peak-hour load factor ratios by the type of traffic signal system inasmuch as the data for this study included the most heavily loaded intersections in all areas of the United States. It is believed that the sample was fairly representative for the various areas because each selected intersection was generally at the location of the worst congestion on a street or highway, or system of streets or highways. It is not purported, of course, that the sample includes the most heavily loaded intersections in the United States as a whole.

Tables 9 and 10 give the distribution of signal types and the average load-factor ratio for each of the signal types separated by isolated signal locations in coordinated systems. The figures in these tables indicate that during periods of peak flow, at least on an average, fully actuated signals are either not being operated properly or do not have the type of performance that they are normally expected to have. The results, however, confirm the advantage obtained by the increased use that is being made of flexible progressive systems. The analysis to determine the effect of the type of signal system on the capacity of various types of streets has not, as yet, been completed. It is expected that the results of the study will be extremely useful in further improving the efficiency of traffic flow in the United States. There is still plenty of room for improvement.

STABILITY OF PEAK-HOUR FACTORS

The results of published studies on the 30th highest hourly volume during a year is an extremely reliable index for use in the design of highway facilities. It does change with time and with increases in traffic volume but these changes can be fairly accurately predicted. If the peak-hour factor is likewise to be a useful index for the design of intersections or for predicting future traffic volumes that they can accommodate, it is necessary to know more about the variables that tend to cause changes in the magnitude of the peak-hour factor. Although this study was not designed for this specific purpose,

TABLE 9

DISTRIBUTION OF TRAFFIC SIGNAL TYPES AND LOAD FACTOR RATIOS AT ISOLATED SIGNAL LOCATIONS

Type of Operation	Distribution (percent)	Average Load Factor Ratio
Fixed time	80	0 54
Pre-timed program	3	0 64
Semi-actuated	6	0 66
Fully actuated	11	0 36

TABLE 10

DISTRIBUTION OF SIGNAL TYPES AND LOAD FACTOR RATIOS FOR COORDINATED SYSTEMS

Type of System	Distribution (percent)	Average Load Factor Ratio
Simultaneous	7	0 43
Alternate	10	0 34
Simple progressive	63	0 42
Flexible progressive	20	0 61

TABLE 11

DISTRIBUTION OF RATIOS BETWEEN PEAK-HOUR FACTORS FOR THE TWO HEAVIER CROSS-MOVEMENTS AT EACH INTERSECTION

Peak-Hour Factor Ratio		Intersection with Data Available		Avg. Highest Peak-Hour Factor	Avg. Highest Load Factor	Avg. Load Factor Ratio
Group	Avg.	No	Percent			
0.54 - 0.74	0.64	4	1.5	0.85	0.59	0.45
0.75 - 0.79	0.77	8	3.0	0.87	0.42	0.36
0.80 - 0.84	0.82	12	4.5	0.86	0.33	0.40
0.85 - 0.89	0.87	33	12.3	0.85	0.49	0.42
0.90 - 0.94	0.92	79	29.5	0.89	0.54	0.53
0.95 - 0.99	0.97	132	49.2	0.88	0.57	0.50
Total		268	100.0			

the data do lend themselves to a few preliminary results that will help to guide future studies.

Table 11 shows, for example, that at about 50 percent of the intersections the peak-hour factors for the two heaviest cross-movements were within 5 percent of one another, and at nearly 80 percent the difference was less than 10 percent (cols 1 and 4). Also, the magnitude of the highest peak-hour factor at an intersection did not have a tendency to be greater where the difference between the two peak-hour factors was large than where the difference was small (col 5). Furthermore, there is only a slight, if any, tendency for the highest load factor at an intersection, or the load-factor ratios, to be greater at locations where the difference in the peak-hour factors for the two heavier cross-movements are large than where they are small (col 6 and 7). These are rather important findings if verified by more extensive studies under a larger variety of conditions.

There were only 48 intersection approaches that were studied twice during peak hours on clear days where the traffic volume during one study was appreciably higher than during the other study. The peak-hour factors and peak-hour factor ratios have been summarized in various forms in relation to the traffic flow rates and load factors for these 48 locations in Tables 12, 13, and 14.

TABLE 12

VARIATION IN PEAK-HOUR FACTOR AT SAME APPROACH

Peak-Hour Factor Ratio			Approach Studied Twice		Highest Peak-Hour Factor		Highest Flow Rate (VPHG)		Ratio of Traffic Flow Rates	
Group	Total	Avg.	No.	Percent	Total	Avg.	Total	Avg.	Total	Avg.
0.76 - 0.84	736	0.82	9	18.8	7,952	0.88	17,070	1,895	5.65	0.63
0.85 - 0.89	1,051	0.88	12	25.0	10,760	0.90	24,940	2,078	9.08	0.76
0.90 - 0.94	1,109	0.92	12	25.0	10,936	0.91	30,260	2,522	7.57	0.63
0.95 - 0.99	1,455	0.97	15	31.2	13,140	0.88	35,480	2,365	10.64	0.71
Total or avg.	4,351	0.906	48	100.0	42,788	0.891	107,750	2,245	32.94	0.686

TABLE 13

VARIATION IN PEAK-HOUR FACTOR BY MAGNITUDE OF PEAK-HOUR FACTOR

Highest Peak-Hour Factor		Approach Studied Twice		Avg Ratio Between Peak-Hour Factors	Highest Flow Rate at Approach (VPHG)	Avg Ratio Between Flow Rates
Group	Avg.	No	(%)			
0.75 - 0.79	0.77	4	8.3	0.88	2,420	0.62
0.80 - 0.84	0.82	5	10.4	0.91	2,046	0.68
0.85 - 0.89	0.87	13	27.1	0.94	1,983	0.70
0.90 - 0.94	0.927	22	45.9	0.90	2,447	0.68
0.95 - 0.99	0.964	4	8.3	0.88	2,056	0.75
Total		48	100.0			

TABLE 14

VARIATION IN PEAK-HOUR FACTOR AT SAME APPROACH COMPARED TO VOLUME CHANGE

Ratio Between Traffic Flow Rates		Approach Studied Twice		Avg Highest Traffic Flow Rate (VPHG)	Avg Highest Peak-Hour Factor	Avg Peak-Hour Factor Ratio
Group	Avg	No	(%)			
0.26 - 0.39	0.33	7	14.6	3,410	0.89	0.93
0.43 - 0.48	0.45	5	10.4	3,030	0.92	0.85
0.51 - 0.69	0.62	9	18.8	1,940	0.86	0.90
0.72 - 0.78	0.76	12	25.0	1,960	0.89	0.88
0.84 - 0.89	0.86	4	8.3	1,530	0.89	0.92
0.90 - 0.99	0.94	11	22.9	1,980	0.90	0.94
Total		48	100.0			

Table 12, which gives items by the magnitude of the ratios between the two peak-hour factors, shows that the average difference between the two traffic flows at the same locations were no greater, nor the peak-hour factors higher, where the larger changes in the peak-hour factors occurred than where the smaller changes occurred. Likewise, Table 14, which gives locations by the magnitude of the difference in traffic volume during the two studies, shows that the higher of the two peak-hour factors (col 5) and the ratio of the two peak-hour factors (col 6) do not increase or decrease with an increase in the difference between the traffic flow rates (col 1).

The two peak-hour factors for the same location determined during two different days will, on an average, be within 10 percent of one another even though the traffic volume on one day is triple the traffic volume on another day. The peak-hour factor at a given location apparently does not change with a change in the total flow during the peak hour. This is an extremely important traffic characteristic in relation to intersection design and capacity determinations.

CONCLUSIONS

There is little doubt but that the improvement of the efficiency of traffic movement at intersections is one of the more important, if not the most important, urban transportation problems. This study indicates that there is a lot of room for improvement. The study also develops new criteria in use for improving traffic flow through increased efficiency at intersections regardless of whether this improvement will come about through the use of present traffic control equipment, additional electronic equipment on the car or in the roadway; or the use of new equipment employing radar, infrared, or sonic detection with centralized control employing extensive high-speed computer systems to handle predetermined as well as feed back information.

Much remains to be done in translating the results of this study to a coordinated set of usable charts and tables and in completing the analysis of factors for which only preliminary results are available. What needs to be done, however, is clearly evident and not too involved. The terms "making better use of city streets," "coordinating street and expressway systems," and "the application of more scientific technology to urban transportation problems" are time-worn phrases that no one has completely understood or been able to put in practice to the extent desired. A continuation of this investigation is certain to produce new criteria and information that will go a long way toward the realization of these goals.

Intersection Capacity

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This paper has a twofold purpose: (a) presenting a newly proposed concept for determining capacity and (b) presenting capacity determinations for channelized left turn intersections.

Three types of capacity have been known to be utilized: basic, practical, and possible. Using capacity per se the degree of capacity is specified on a percentage basis.

From this last thought a new method of capacity determination was conceived and a preliminary analysis made thereof. The method used an ogive curve with ordinate values of percent volume and abscissa values of percent maximum volume. A second abcissa scale was superimposed on the graph representing capacity as a percent of total capacity.

The capacity study involved the analysis of 20 intersection approaches. The results obtained from the ogive curve were compared to capacities as determined by the average starting headway and the Highway Capacity Manual methods. The results indicated that there may be some relationship between an ogive curve and capacity.

On this basis the channelized intersection complexes were studied and capacities determined. The results indicate that intersections with the channelized left turn (New Jersey Left Turn) is significantly more efficient than regular intersections.

•WHEN TWO desire lines of travel cross, conflicts between these desires develop. This is what occurs at the intersection of two roads. The conflicts may be eliminated by space (a grade separation) or by time (a traffic signal). The number of conflicts may be reduced by making the desire lines of travel in one direction only—one-way streets. Likewise, the channelization of a left turn maneuver is a method of reducing serious conflicts.

Many studies have been made to determine the effect of left turn maneuvers on the capacity of an intersection. However, little has been done in the way of capacity studies on the channelized left turn because of the lack of such intersections. (The type of intersection referred to is the "New Jersey Left Turn" as developed by W.R. Bellis of the New Jersey State Highway Department.)

The term "capacity" is technically supposed to express the ability of a given roadway to accommodate traffic. However, there have been certain conditions imposed on this term that have produced the terms basic, practical, and possible capacities. The traffic engineer must juggle these to fit the problem at hand. A more reasonable step would be to eliminate the three offshoot terms and use just the term "capacity". Inasmuch as capacity is generally utilized in discussions involving designs or existing conditions, the expression of this term as degree of capacity, given as a percentage, might provide a more useful tool in the analysis of traffic flow.

This paper presents a preliminary study of a proposed capacity determination method for special intersection designs. The ogive capacity curve was the basis for this study and in determining the capacity of the "New Jersey Left Turn" type intersection. The ogive capacity curve also fulfills the definition of capacity in its per se form.

The theoretical capacity of the channelized intersection is extremely difficult to determine through normal procedures of the Highway Capacity Manual (1). The channelized left turn is really a compound intersection with three separate intersections and with at least two of the three being signalized. With the lack of knowledge about capacity of this specialized intersection and other specialized intersections a need has arisen for a capacity study determination and a method for determining the capacity of these or any other intersections. Further, this study involves the relating of capacity to an ogive curve with ordinate values of percent volume and an abscissa of percent maximum volume. A second abscissa scale is superimposed on the curve for determining capacity as a percent of total capacity.

OGIVE CAPACITY CURVE

Quite frequently it is desired to show in diagrammatic form the cumulative frequency above or below a given value. For example, it may be desirable to read off from a chart the number or proportion of cars (volume) whose quantity does not exceed some stated value. Charts of this type are known variously as cumulative frequency diagrams, more-than or less-than curves, and ogives.

For this particular study the ogive curve used is the less-than type where abscissa values are percent of maximum volume. The ordinate values are the cumulative frequencies of the maximum volume values of the abscissa. The plot produces the general S-type of cumulative frequency curve, or ogive.

Theory of the Ogive Curve

The ogive capacity curve was derived from the much-used speed cumulative frequency curve. This speed curve has long been a tool in the study of traffic flow; because of this, the possibility of a similar curve for volumes was considered.

It is known that, as the slope of the tangent portion between the lower and upper portions of the curve increases, the range of speeds decreases, and flow is more uniform. Likewise, as the slope of the tangent decreases, the range of speeds increases, and flow is likely to be more unstable and susceptible to large volume changes. Because capacities and volumes have the same units, the belief arose that a similar cumulative frequency plot using volumes instead of speeds would also give some explanation about the character of traffic flow. Similarly, the speed range had to be replaced by some range of volumes. Because capacity has the same units as volume, the speed range was replaced with a capacity measure as a supplemental abscissa. Therefore, the changes in slope of the volume-frequency curve could then be reflected as a change in the percent of capacity. Percent of capacity is defined as the ratio of sample hour volume to the maximum capacity of a given facility. This relation is expressed in the following equation:

$$C_p = \frac{V_s}{C}$$

in which

C_p = percent of capacity;

V_s = the sample volume; and

C = the capacity of the facility.

In the next section, percent capacity is determined from the ogive curve, the volume sample is the field data, and the unknown term is the capacity.

Development of the Curve

The development of the ogive curve used in this study is based on the distribution of volume groups and their relation to maximum volume values. These relations are plotted vertically as percent of volume (a cumulative volume distribution) and horizontally as percent of maximum volume.

In the development of the curve and preliminary testing, data were used that had been gathered on freeways flowing at or near capacity. One-min volume counts were taken by individual lane for AM, midday and PM periods of flow in the Edsel Ford and John Lodge freeways in Detroit. The one-min volumes were grouped and the frequencies of the various volumes determined. (See Table 1 for a sample set of data.)

Volume groups are those 1-min volumes or cycle volumes that may occur in any given period of time. That is, during 1 hr there would be 60 1-min volumes. Some of these one-min volumes occur more than once and are put into volume groups. Each volume group could then potentially contain from 0 to 60 repetitions (frequencies). Frequency is therefore the number of times any one volume group occurred during the 1-hr period. Cumulative frequencies are the accumulation of all frequencies starting from the smallest volume group and adding the successive frequencies to the preceding accumulative

TABLE 1

SAMPLE DATA FOR THE DEVELOPED OGIVE CAPACITY CURVE

Volume Group	Tab	f	Cum f	Percent Volume	Percent Max. Vol.	Total Vol./Gp.
11	X	1	1	1.7	25.0	11
12		0	1			
13		0	1			
14		0	1			
15		0	1			
16	X	1	2	3.4	36.4	16
17		0	2			
18		0	2			
19	XX	2	4	6.7	43.2	38
20	X	1	5	8.4	45.4	20
21	XX	2	7	11.7	47.7	42
22	XX	2	9	15.0	50.0	44
23	X	1	10	16.7	52.5	23
24		0	10			
25	X	1	11	18.4	56.8	25
26	X	1	12	20.1	59.1	26
27	XX	2	14	23.4	61.4	54
28	X	1	15	25.1	63.6	28
29		0	15			
30	X	1	16	26.8	68.2	30
31	XXXXO	5	21	35.1	70.5	155
32	XXXX	4	25	41.8	72.7	128
33	XX	2	27	45.1	75.0	66
34	XXXX	4	31	51.8	77.3	136
35	XXXXO	5	36	60.1	79.5	175
36	XXXXOXXXXO	10	46	76.8	81.9	360
37	XXXX	4	50	83.5	84.1	148
38	XX	2	52	86.8	86.4	76
39	XXX	3	55	91.8	88.6	117
40	XX	2	57	95.1	90.9	80
41	XX	2	59	98.4	93.1	82
42		0	59			
43		0	59			
44	X	1	60	100.0	100.0	44
						1926

value. The cumulative frequency for the last volume group should total to 60 for a 1-hr period of 1-min volume groups.

The value for percent volume is determined by taking the smallest cumulative frequency and dividing it by the largest. In this example, it would be X / 60 = percent volume. The value for the percent of maximum volume is determined by taking the smallest volume group and dividing it by the maximum volume group. This is done for each of the volume groups including the last, which is obviously 100 percent. A plot of percent volume vs percent maximum volume yields a series of points and when they are connected an ogive curve is formed.

The secondary abscissa, percent capacity, is superimposed on the percent maximum volume abscissa but with its origin shifted to the right of the ogive curve origin. The origin coincides with the tangency of the ogive curve with the abscissa. The upper end of the capacity scale and maximum volume scale coincide at 100 percent. These characteristics of the ogive curve are shown in Figure 1.

Application of the Curve

The application of the ogive curve is the same as in the development of the basic ogive curve. The calculations made are identical to those previously discussed. The plot of data is the same except that, for ease in plotting and to minimize the work involved, the grouping of volume groups with a class interval or 3 accomplishes an end result with little error.

With the points plotted on the ogive capacity curve, straight lines are drawn between the points. The slope of the lower portion of the curve ($\overline{AB}$) (see Fig. 2) is transferred to the ogive capacity curve. At the point of tangency a line is extended vertically downward to the capacity line (abscissa). This is also done for the upper portion of the curve.

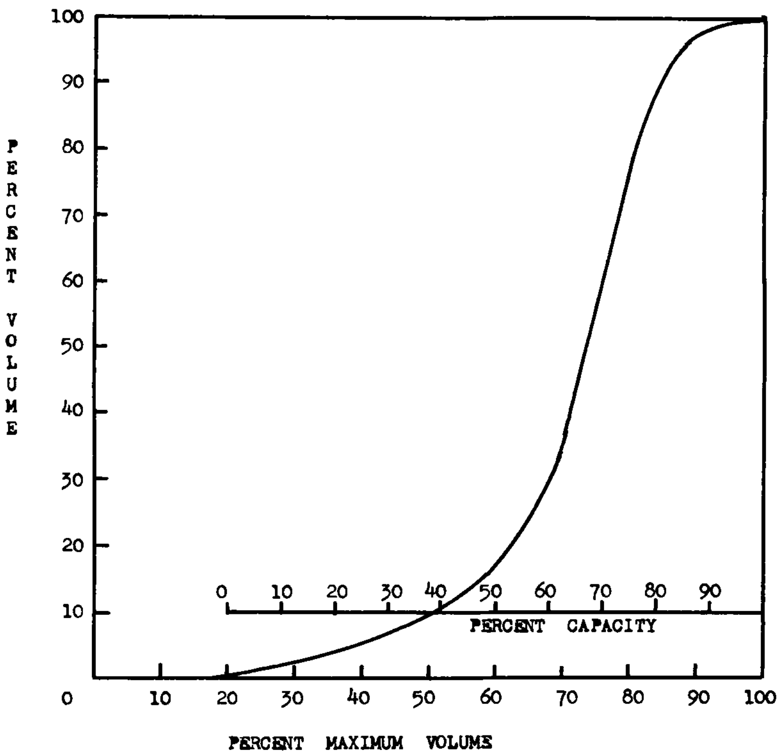

Figure 1. Developed ogive capacity curve.

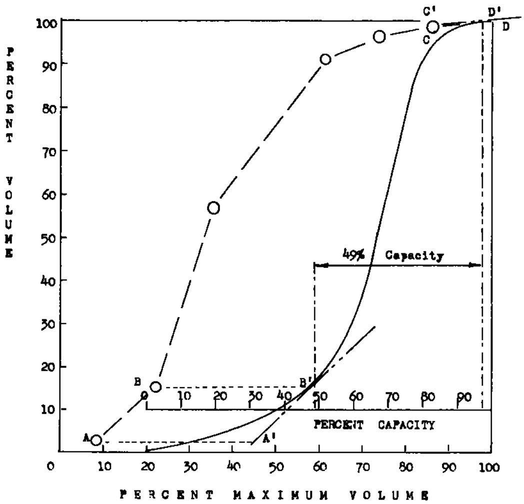

Figure 2. Example ogive capacity curve determination. Dashed line is cumulative frequency curve plot of volumes obtained at intersection with signal cycle length of 50 sec. Data given in Table 2.

The line ($\overline{CD}$) is transferred to the upper portion of the ogive capacity curve and at the point of tangency a second line is extended vertically downward to the capacity line. The difference between the two percentages read from the percent capacity scale is that portion of capacity for the given set of conditions.

NEW JERSEY LEFT TURN

For more than three decades it has been recognized that the intersection is the big bottleneck in traffic flow. In 1923, the Bronx River Parkway was opened to traffic and with it the door opened to a new era of road design with its grade-separated intersections.

From the first grade separation to today's complex multilevel interchanges the goal has been to improve the flow of traffic through the intersection area. The ability to improve the flow for at-grade intersections, however, has not kept pace with the interchange, but one of the more important steps was the introduction of the New Jersey Left Turn (2), or the Channelized Left Turn.

Due to the relative newness and paucity of the special left turn type of intersection little has been done in the way of capacity analyses. Being of special breed this type of intersection presents many problems for conventional methods of analysis. Because of this, the ogive capacity curve method was developed to overcome some of the older method's shortcomings.

Theory of the New Jersey Left Turn

The theory underlying the design of the New Jersey Left Turn is the removal of the left turn conflict from the intersection. The left turn maneuver is removed at least

TABLE 2

SAMPLE TABULATIONS OF VOLUME DISTRIBUTIONS, PERCENT VOLUME AND PERCENT MAXIMUM VOLUME VALUES NEEDED TO DETERMINE CAPACITY BY OGIVE CAPACITY CURVE METHOD

Volume Group	Tab		Class Mark	Mid-Mark	Cum f	Percent Volume	Percent Max Vol
1		0	1 - 3	2	2	2 8	8 7
2		0	4 - 6	5	11	15 3	21 7
	(2)						
3	XX	2	7 - 9	8	41	57 0	34 8
4		0	10 - 12	11	55	76 5	47 8
5	XXX	3	13 - 15	14	66	91 8	60 8
	(9)						
6	XXXXOX	6	16 - 18	17	70	97 2	73 8
7	XXXXOXXXXOXXX	13	19 - 21	20	71	98 4	86 8
8	XXXXOXXXX	9	22 - 24	23	72	100 0	100 0
	(30)						
9	XXXXOXXX	8					
10	XXXXOXX	7					
11	XXXXO	5					
	(14)						
12	XX	2					
13	XXXXOX	6					
14	XXXX	4					
	(11)						
15	X	1					
16	X	1					
17	XX	2					
	(4)						
18	X	1					
19	X	1					
20		0					
	(1)						
21		0					
22	X (1)	1					
	Vol = 699						

300 ft from the main intersection and cuts diagonally across one of the quadrants. The left turn lane is complemented by an adjacent median separated right turn lane (see Fig. 3 for the general intersection layout). An additional signal is needed for the efficient movement of traffic through the left turn merge area. Progressive timing of this signal gives a minimum delay to all vehicles and yet the left turn can be made without conflict to normal traffic flow throughout the intersection area. Also aiding in the smooth flow of traffic are the shadowing effect produced by the main intersection signal for the left turn diverge and there would not be right turning vehicles from the main intersection crossing the diverge maneuver. Thus, this theory of design provides more safety, comfort, and convenience to the drivers using the facility.

Method of Study

When this study was being conceived, it was planned that more than just the special left turn intersection should be studied. It was planned to use a control intersection for comparative purposes with two of the New Jersey Left Turn intersections. Agreement, then, between capacities of the ogive curve and the presently accepted Highway Capacity Manual method, for the control intersection would give some basis for valid capacity values for the left turn intersections.

Test Sections.—The two New Jersey Left Turn intersections chosen for the study were somewhat different in geometric design of the entire intersection, but the special left turn element of design was the same (see Figs. 4, 5, and 6 for the intersection layouts). It is quite interesting to see at-grade intersections as complicated as these and yet to know that there is very little conflict between movements.

One of the sites selected for the New Jersey Left Turn was in the City of Saginaw, Mich. (Fig. 4). The intersection consists of two intersecting State highways, M-13 and M-46, in an urban area. To the north side of the intersection lies a city park and to the south lies a residential area. The north-south street is a main route into the

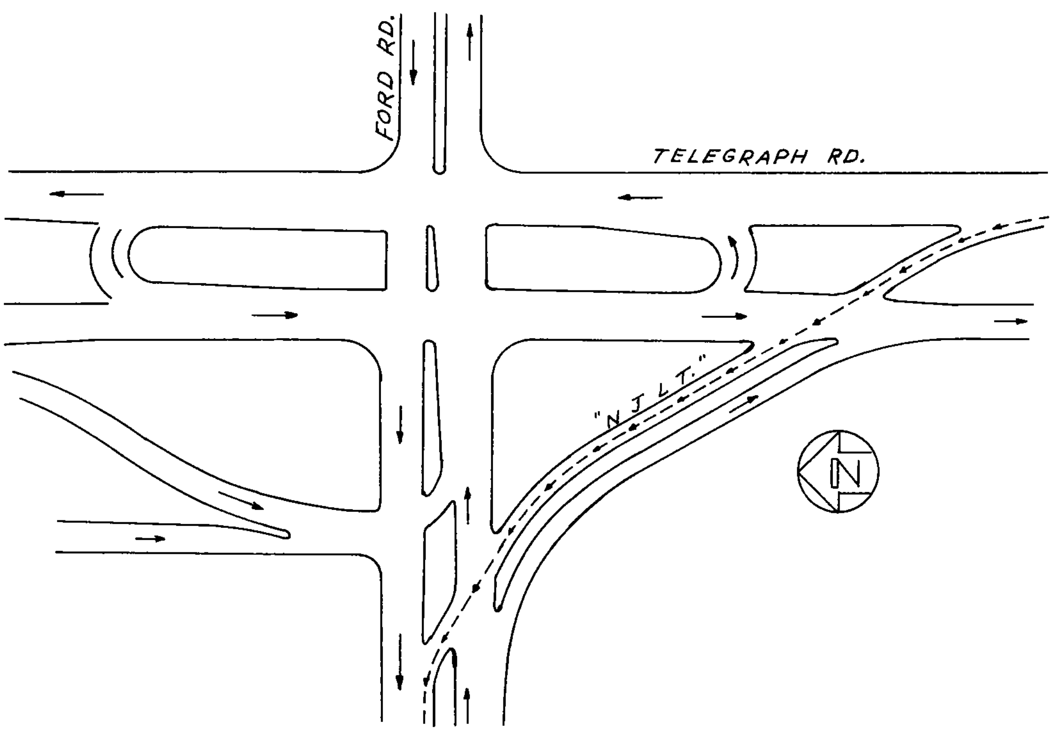

Figure 3. Example of New Jersey Left Turn movement.

central business district. The east-west street serves an industrial area to the east and suburban residential area to the west. There are two special left turn lanes which are the result of desired traffic movements. Intersection 1 in the figure does not have signal but does have a yield sign for the left turn diverge movement. Intersections 2, 3, and 4 have signals and are timed progressively as a closed network. Thus, all major movements are free from other conflicting maneuvers.

The other similar intersection selected was in suburban Detroit (Fig. 5). The intersection consists of two high-volume intersecting State highways—Ford Road (M-153) and Telegraph Road (US 24)—in a commercially developed area. The prominent feature of the intersection is the left turn lane from the south to west. The operation is such that intersection 1 is not signalized but has a yield sign for movement control. Intersections 2 and 3 are progressively timed, which allows all major movements to be made with little conflict. The operation is such that the "jug-handle" also has an opportunity to maneuver with a minimum of conflict.

The control intersection is on the same north-south highway (US 24), as the intersection just described (Fig. 6). This intersection lies 4 mi south in an urban area with commercial developments in the four quadrants. The intersection is signalized without separate phases for left turn movements which must necessarily cause conflicting maneuvers. The left turning vehicles generally have long delays and those that do get through generally do so on amber or red. The south to west and east to south left turn movements are prohibited.

Field Work and Data Collected.—The data collected were volume, speeds, delays, traffic composition, signal timing, and general geometrics of the intersections. To complement the field data an observer was used to record various changes in flow; such as accidents, congestion, or any other type of disruption.

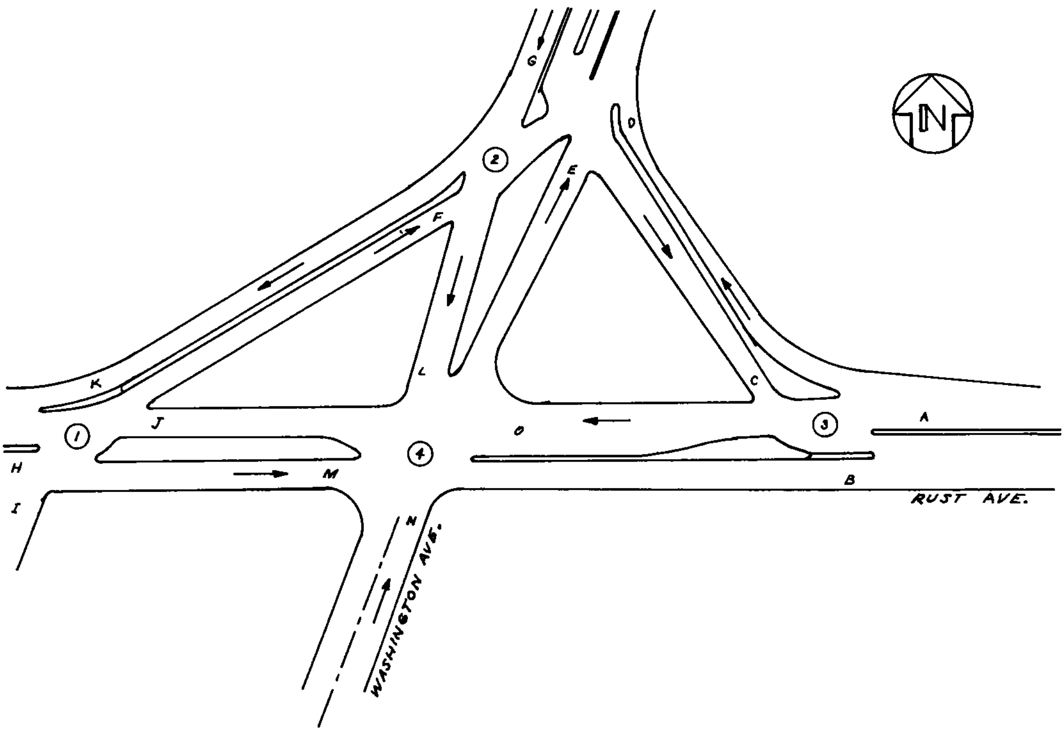

Figure 4. Rust-Washington Ave. intersection layout and approach designations.

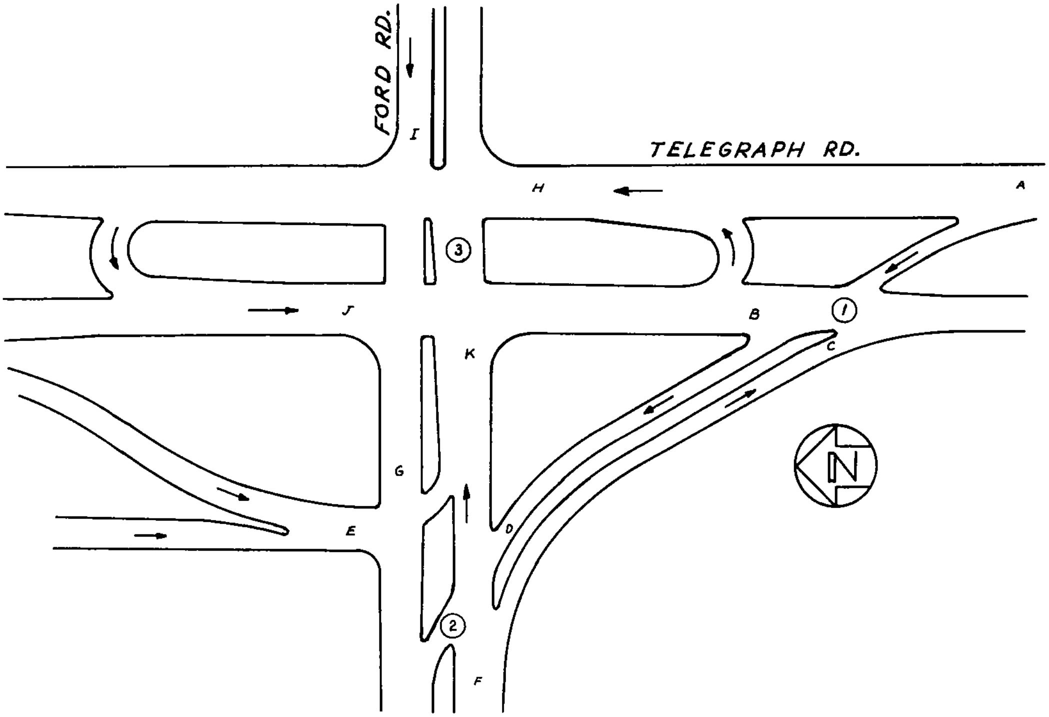

Figure 5. Ford-Telegraph Road intersection layout and approach designations.

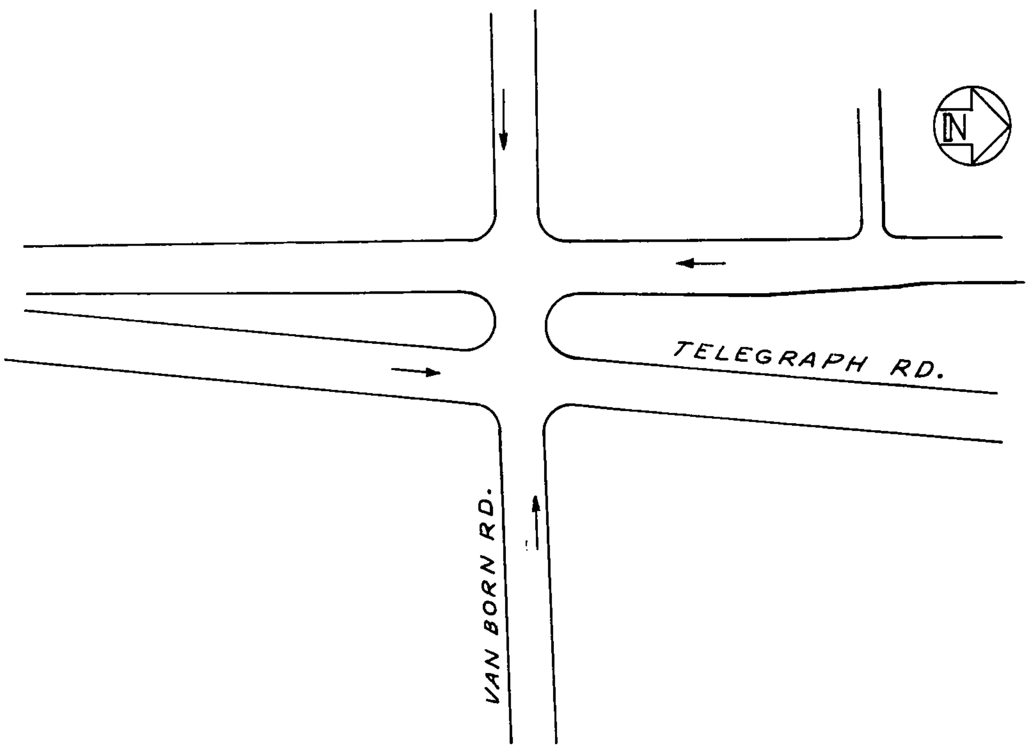

Figure 6. Van Born-Telegraph Road intersection layout and approach designations.

The data were taken manually for the most part. Speeds were obtained by stopwatch or radar speedmeter. Volumes were taken by accumulative hand counters. Vehicle delays were determined by manual counts.

Volumes were collected for 3-hr periods during the AM, midday, and PM hours for 1-min intervals. The traffic was further classified according to passenger car or truck. The volumes were taken by lane in all instances.

Speeds were recorded for all the approach movements and left turn lanes. The speeds were sampled at the rate of 25 per approach per study period. The speed samples are speed estimates of a typical vehicle in a platoon rather than the speed of an individual vehicle.

Delays were obtained for left turns only. Delays were determined by counting the number of vehicles delayed for one red phase, the number that went through on the next green phase, and the number of vehicles that did not make it through on that green phase. This information was obtained for each complete cycle of the study.

Signal timings were obtained periodically throughout the day to check the signal operation for uniformity and changes in timing. Offsets were determined also for the additional signals at the left turn merge.

Method of Analysis. —Capacities were determined in accordance with the procedures described in the first part of this study. Capacity calculations were made by the ogive curve method, by the Highway Capacity Manual method, and by use of average starting headways.

The element of time used in the ogive capacity curve determinations was the cycle. This made little difference, though, in the process as the frequency of each volume and the percent of each volume in relation to the maximum volume were calculated. Capacities were determined for the AM peak, midday hour, and PM peak for some 28 approach elements.

Capacity determinations were also made for each approach element according to the Highway Capacity Manual (1), pp. 6-104 procedure. The third series of capacity calculations were made by the average starting headway method as described by Matson and McGrath (3).

Analysis of Data

Delays to Left Turns. —The Van Born-Telegraph Road intersection has only two left turns permitted—from the north and from the east. There was little delay for this maneuver from the north as a left turn lane was provided and there was an ample median to shadow stored cars. However, over 60 percent of the vehicles turning from the west were delayed for at least one cycle. It is likely that the delays are a result of no left turn lane or storage area and that the left turning vehicles had to cross a through movement of just under 1,100 vehicles during the peak hour.

The main intersection at Ford and Telegraph Roads have left turn restrictions for all approaches. A modified left turn maneuver is made at the diverge to the New Jersey Left Turn channel but, there is little delay here as the turning vehicles only have to yield to through movement. The merge maneuver of the left turn lane is controlled by a signal and is the only place where appreciable delay could occur. There was less than one vehicle delayed per cycle or the average delay per delayed vehicle, using one-half the red time as average delay, is just under 9 sec.

Delays were also kept to a minimum for the New Jersey Left Turn portions of the Rust and Washington Street intersection. There were no delays at the merge for the north-to-east movement. Those vehicles delayed at the diverge signal were delayed only the normal amount caused by random arrival of vehicles at a red indication. Delays at the diverge to the west-to-north channelized left turn lane were only those that occurred as a result of not being able to find an acceptable gap in the through traffic lane as the movement is controlled by a yield sign. Delays at the merge end of this left turn channel were only those caused by random arrival of vehicles at a red signal indication. The only time that a vehicle did not make it through on the first cycle was when there was a backlog of eight vehicles in front.

The east-to-south left turn movement was very light. Averages were about one vehicle per cycle during peak and about one-half vehicle per cycle for the entire study period, which did not yield enough data for developing good conclusions. The left turn was made from a separate lane in the median. The south-to-west left turn was made from a through lane causing delay to through vehicles as well as left turn vehicles. The greatest backlog of vehicles was 15 with an average of 20 percent of the vehicles being delayed for more than one cycle.

Capacity Analysis: Van Born Road at Telegraph. —Capacities were calculated for each approach to the intersection and are given in Table 3. Capacities were determined for the hours of 7 to 8 AM, 11 to 12 AM, and for the peak hour of 4 to 5 PM.

Approaches from the north and south have similar geometrics and, likewise, similar capacities. The average ogive capacity was just under 1,900 vph or about 540 vph per through lane. This is an average of 7.5 vehicles per cycle, which is one less than that for average starting headways. Calculations using the Highway Capacity Manual (HCM) approach were 1,531 vph total or 431 vehicles per through lane. This yields an average of 5.8 vehicles per cycle.

The east-west approaches also have similar geometrics and capacities. The ogive curve capacity was determined to be 504 vph per lane or an average of 7 vehicles per cycle. Average starting headways would indicate that 8.5 vehicles per cycle are possible. The HCM capacities were calculated to be 316 vph per lane or an average of 4.4 vehicles per cycle.

For this intersection of generally standard geometrics, there are indications that the HCM capacities and probable capacities for average starting headways differ—4.4 as compared to 9 vehicles per cycle. The ogive curve compares 7 and 7.5 vehicles per cycle 9 for average starting headways. It would therefore appear that the ogive determination is a reasonable indication of the capacity per lane with a 50-50 split of a 50-sec cycle.

TABLE 3

SUMMARY OF VAN BORN APPROACH CAPACITIES[1]

Item	Approach			
	North	West	South	East
No of lanes	4	2	4	2
Avg. lane width (ft)	12	10.5	12	10.5
Peak hr vol. (no.)	947	544	1,146	953
% peak hr is of ogive capacity	48	54	81	102
Ogive cap. 7-8A	1,975	980	1,400	920
Ogive cap. 11-12	1,780	1,040	1,120	850
Ogive cap. 4-5P	1,940	1,005	1,710	1,020
Ogive cap. avg.	1,898	1,008	1,410	930
Ogive cap. /hr green	3,796	2,016	2,820	1,860
HCM cap.	1,531	632	1,036	668
HCM cap. /hr green	3,062	1,264	2,072	1,336
Ogive cap. /lane	540	504	353	465
HCM cap. /lane	431	316	259	334
Green time (sec)	22	22	22	22

[1] All capacities listed are per hour for the given green indications except those specified as being per hour of green time.

Capacity Analysis: Ford Road at Telegraph.—This intersection is a New Jersey Left Turn intersection. The special left turn lane and complementary right turn lane produce different intersection characteristics. The intersection is really comprised of three intersections and each should be considered separately. (Fig. 7 shows numbering of intersections.)

Intersection 1 was unsignalized with a yield sign controlling the left turn diverge movement. The movement was, however, shadowed by the signal at Intersection 3 so that there was a period of time for relatively free movement. The peak volume for this movement was 309 vehicles which occurred with little delay. All other movements were free maneuvers and subject to continuous flow. A comparison of capacity values for the north-south movements is given in Table 4.

Intersection 2 is the merge of the left turn lane and the diverge of the complementing right turn lane. The west-to-east and left turn merge maneuvers are controlled by traffic signals while the east-to-west movement is uncontrolled.

The left turn merge peak hour volume was 309 vehicles or an average of 4.3 vehicles per 50-sec cycle (55-45 split). Although the approach was seldom fully pressurized, there was an indication that the peak movement may be about 8 vehicles per cycle or about 576 vph. The ogive curve capacity was determined as 707 vph with an average of 9.8 vehicles per cycle. This is a little more than one vehicle less than could be attained if average starting headways prevailed. The capacity as determined by the HCM indicated about 8 vehicles per cycle, which is about the same as indicated from observing peak flow.

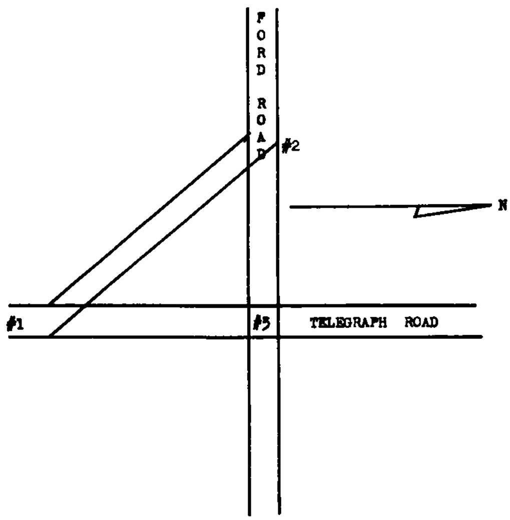

Figure 7. Line diagram of Ford-Telegraph Road intersection giving intersection numbering.

The capacity of the approach from the west as determined by the ogive curve method was 1,437 vph. Considering through lanes only this would be an average of 380 vph per lane or 5.3 vehicles per cycle. This is about 2 vehicles per cycle less than could be attained with average starting headways but about 1 vehicle more than that as determined by the HCM calculation. The apparent approach capacity by these latter methods would be about 2,100 and 1,265 vph, respectively. In light of the capacity determined for the left turn lane (600-600 vph), the value of 2,100 vph would not seem to be too bad an estimate of capacity. However, previous calculations have shown the capacity as determined by average starting headways to be 10 to 20 percent higher, which would imply that a capacity of about 1,500 to 1,700 would be more nearly the capacity estimate.

Intersection 3 is the main intersection of the complex and provides for the straight through movements and right turns. The capacity for the north-south approaches is about 2,500 vph and about 1,650 vph for the east-west approaches.

A comparison of the capacities as determined by the other methods indicates the HCM values to be lower and the average starting headway capacities higher than those determined by the ogive curve method. However, the difference between the ogive capacity and the average starting headway capacity could be accounted for by any slight delay to starting vehicles. If average starting headways prevailed for north-south movement with its indicated HCM capacity, nearly 40 percent of green time would not be utilized, but field data has indicated that as many as 12 vehicles may go through the

25-sec green interval in a single lane which would tend to disprove the low estimate of the HCM. Likewise, a similar analysis can be made for the east-west movement except that the unused green time is not as much.

Capacity Analysis: Rust at Washington. —The Rust-Washington Avenue intersection is a rather complex set of intersections comprised of three "junior intersections" and the main intersection. Three of the four intersections are signalized and for all practical purposes their operation is two-phase (Fig. 8 shows numbering of intersections.)

Intersection 1 is the diverge movement of the left turn lane and is unsignalized. Approaches from the west and east are free maneuvers, whereas the diverge and right turn merge are semi-free maneuvers in that they are controlled by yield signals. Capacity calculations by either the HCM method or average starting headway method are not applicable in this instance, but a capacity determination for any of the approaches could be made by the ogive curve method. This method indicated capacities of 675 vph for the right turn lane, 1,537 vph for the approach from the west, and 1,170 vph for the approach from the east. The ogive curve method does not consider signal time per se; however, the method does rely on the distribution and magnitude of volumes for any given period of time (see Table 5 for capacity calculations).

Intersection 2 is a complex of movements; a free right turn from the east, a free right turn to the west, a left turn diverge from the north to the east, and straight through movements from the south to north.

Although the right turn maneuver from the east is a free movement, ogive capacity determinations were made. The ogive capacity was 567 vph compared to 800 vph by the HCM method. As it has been pointed out previously, ogive capacities are dependent on prevailing traffic characteristics, and if these change enough, a change in the capacity could result. This point is brought out here because the AM ogive capacity was determined as 735 vph. This might therefore mean that the conditions during the AM period of study were very close to those used in the HCM capacity calculations.

The two-lane approach from the south indicated similar values for lane capacities. The HCM value was 803 vph per lane and the ogive value was 633 vph per lane. It might also appear that lane width has little effect on capacity as this approach has 12-ft lanes compared to the previously described approach which had a 16-ft lane.

TABLE 4

SUMMARY OF FORD ROAD APPROACH CAPACITIES[1]

Item	Approach									
	A	B	C	D	F	G	H	I	J	K
No. of lanes	4	4	1	1	4	3	4	3	4	4
Avg. lane width (ft)	12	12	16	16	12	12	12	12	12	12
Peak hr vol. (no.)	2,070	1,256	153	309	699	1,327	2,018	1,566	1,246	702
% peak hr is of ogive capacity	92	58	15	44	49	74	82	95	47	38
Ogive cap. 7-8A	2,322	2,130	1,220	758	1,595	2,010	2,740	1,556	2,250	1,680
Ogive cap. 11-12	2,150	1,990	880	705	1,290	1,920	2,210	1,710	2,870	1,735
Ogive cap. 5-6P	2,290	2,280	956	658	1,425	1,460	2,450	1,705	2,770	1,635
Ogive cap. avg.	2,254	2,133	1,019	707	1,437	1,797	2,467	1,657	2,630	1,683
Ogive cap. /hr green	3,965	3,750	1,019	1,245	3,320	4,155	4,340	3,830	4,630	3,900
HCM cap.	3,615	2,180	1,025	568	1,265	1,495	1,568	1,339	2,300	1,531
HCM cap. /hr green	6,360	3,840	1,025	1,000	2,930	3,450	2,750	3,095	4,050	3,540
Ogive cap. /lane	564	533	1,019	707	380	599	617	552	552	658
HCM cap. /lane	904	545	1,025	568	323	498	392	446	575	510
Green time (sec)	25	25	Free	25	19	19	25	19	25	19

[1] All capacities listed are per hour for the given green indications except those specified as being per hour of green time.

The approach from the north is geometrically two lanes but, due to the green arrow indication for the right-hand lane the through movement could be considered for just one lane. The capacity calculations would tend to bear this out as the HCM value is 492 vph and the ogive value is 407 vph.

The left turn merge lane from the west is a single 16-ft lane and the capacity calculations by both methods were about the same—483 and 485 vph.

Intersection 3 is the merge area of the east left turn lane. The approach from the east is controlled by the signal that controls the merge movement, whereas the approach from the west is a free movement through the intersection.

The through lane capacity is about the same for the HCM and ogive curve methods—378 vs 406 vph per lane. However, the average starting headway method yielded a value of 540 vph per lane. This difference cannot be accounted for except that there must be other conditions affecting the intersection flow.

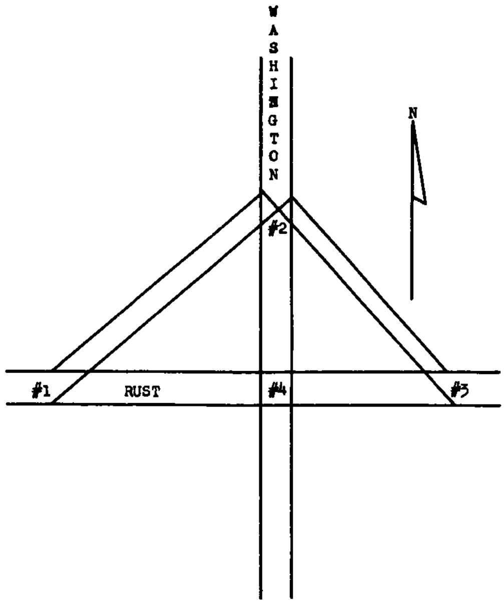

Figure 8. Line diagram of Rust-Washington Ave. intersections giving intersection numbering.

Calculations by both the HCM and ogive curve capacity methods are about the same—528 vs 499 vph per lane. These values are about 25 percent lower than the capacity determined from average starting headways. Again, there is an indication that average starting headways do not prevail.

Intersection 4 is the main movement area; however, some of the turn movements are prohibited. Basically the intersection is the junction of two four-lane roadways divided in the intersection area except for the south approach which is undivided. The approach from the east has a separate left turn lane, whereas the approach from the south does not.

The north and south approaches have similar calculated values in that the north is 455 vph per lane compared to 484 vph per lane by the ogive curve method. Similarly, HCM values are close (725 and 683 vph per lane), but these are almost 50 percent

TABLE 5

CAPACITY CALCULATIONS[1]

	Approach													
	A	B	C	D	E	F	G	H	J	K	L	M	N	O
No of lanes	3	2	1	1	2	1	3	3	2	1	2	2	2	3
Avg lane width (ft)	12	12	16	16	12	16	12	12	12	16	12	12	12	12
Peak hr vol (no)	790	355	169	235	411	139	652	501	619	194	367	344	579	548
% peak hr is of ogive capacity	69	43	34	41	33	29	73	43	40	29	40	33	60	43
Ogive cap 7-8A	995	955	585	735	1,315	548	870	1,260	1,690	507	905	1,170	890	1,440
Ogive cap 11-12	1,055	750	432	435	1,255	344	925	1,155	1,250	743	972	1,000	880	1,035
Ogive cap 3-4P	1,385	790	470	530	1,225	556	875	1,095	1,670	775	852	783	1,130	1,375
Ogive cap avg	1,145	832	499	567	1,265	483	890	1,170	1,537	675	910	1,051	967	1,283
Ogive cap /hr Green	2,430	1,770	942	567	2,390	1,025	1,685	1,170	3,270	675	1,720	2,240	1,825	2,730
HCM cap	1,090	903	528	800	1,605	485	1,072	2,539	980	1,020	1,450	747	683	1,087
HCM cap/hr Green	2,330	1,925	995	800	3,030	1,035	2,020	2,539	2,090	1,020	2,730	1,595	1,285	2,310
Ogive cap /lane	406	416	499	567	633	483	297	422	768	675	455	525	484	566
HCM cap/lane	378		528		803	485	357				725	373	342	468
Green time (sec)	23	23[2]	26	Free	26[2]	23	26	Free	23[2]	Free	26	23	26	23

[1]All capacities listed are per hour for the given green indications except those specified as being per hour of green time

[2]The effective green time.

greater than ogive capacities. The HCM values are about the same as 720 vph per lane determined by the average starting headway method.

The east and west approaches also have similar capacity values by both methods, but the ogive values are about 40 percent larger than the HCM values. The east has an ogive capacity of 566 vph per lane compared to 468 vph per lane for the HCM method. The west approach values are, respectively, 525 and 373 vph per lane. These values can all be compared to 590 vph per lane by the average starting headway capacity, but this value is higher than for either of the other methods. Again, average starting headway capacities are higher than HCM or ogive capacities.

INTERSECTION CAPACITY AS RELATED TO THE NEW JERSEY LEFT TURN

One of the objectives of this study was to evolve a capacity of the special left turn lane and compare this capacity with the capacity of a left turn at a standard intersection. The determination is based on three New Jersey Left Turn lanes and three standard left turn movements.

The intersection of Rust and Washington allows two normal left turns along with two separate New Jersey Left Turn movements. The left turn movement from the south had 136 vehicles turning during the peak hour, but this was with delays to all vehicles and to some a double signal delay. It would be somewhat optimistic to believe that the left turn volume would increase, especially if the volume of opposing traffic also were to increase. However, if the left turn volume did increase there would probably be some compensating volume change in the opposing flow. Also, this left turn is made from a regular through lane and must therefore affect the through capacity from the south.

The approach from the east has a separate left turn lane. Although the volume was quite low (64 vehicles during the peak hour), there was little delay to turning vehicles and little or no effect on through vehicles. Calculations indicate that the left turn movement could be 150 vph. This is not much higher than the present left turns from the south, but this would be across an opposing movement of 1,051 vehicles of which 15 percent are trucks.

The Telegraph-Van Born Road intersection also had two similar left turns—one with a lane and the other without a separate lane. The approach from the north has a separate left turn lane with a peak-hour volume of 215. The advantage of some 60 vehicles can be accounted for in the more adequate storage in the intersection median area for left turning vehicles. The cars can stack up three abreast and two deep and even on the yellow portion these cars could go through. It is further pointed out that these stored vehicles can actually move through on the green portion of the east-west movement without disrupting the normal movement appreciably. Without such an adequate storage area it is very likely that fewer cars would be able to cross the heavy northbound movement.

When the left turn movement is considered from the west approach where there is no separate left turn lane nor median area for storage, the volume drops off to 116 vph. This value is similar to those found at the Rust-Washington intersection.

The New Jersey Left Turn lane capacity shows quite a different picture. Both left turn lanes at the Rust-Washington intersection had low peak-hour volumes (169 vph for the east lane and 139 vph for the west lane), but the main point is that they had little delay. The estimate of capacity is also positive in that these lanes could handle nearly 500 vph or two or three times the traffic of the previously mentioned standard type of left turn lane.

The Ford-Telegraph intersection has a higher capacity than the Rust intersection by some 200 vph. The calculated capacity of 707 vph is more than twice the measured peak-hour flow of 309 vph. The 309 vehicles were delayed little or about the amount normally found in the random arrival of vehicles at a signalized intersection. There is definitely some significance in the ability to put 300 vehicles through a left turn with little or no effect on opposing traffic and with a minimum of delay to those vehicles making the turn.

An over-all intersection capacity analysis was made also to determine the relative degree of capacity at peak flow. The Ford-Van Born intersection indicated that during the peak hour it was flowing at 71 percent of capacity or at a total daily volume of

46,410 vehicles. The corresponding capacity is 65,500 vehicles per day. The Ford-Telegraph intersection, which is similar but contains the special left turn lane, indicated a peak-hour flow of 56 percent capacity or at a total daily volume of 61,866 vehicles. The corresponding capacity would be 110,500 vehicles per day. Finally, the Rust-Washington intersection indicated that peak-hour flow was at a low 44 percent of capacity or at a total daily volume of 25,837 vehicles. The corresponding capacity would be 58,800 vehicles per day. Comparing this intersection's eight approach lanes with Van Born's twelve, it would seem that the latter intersection could handle one-third more vehicles. However, the difference is only 10 percent and this may be some indication that with the special left turn lane capacity is increased considerably. This point is accented more in a comparison of Ford's 110,500 vehicle capacity to Van Born's relatively low 62,000 vehicle capacity.

In summary, the analyses of intersection capacity encompassed some 28 approaches. Of these, eight were either a free maneuver or semi-free maneuver and, hence, it was difficult to make a complete capacity analysis with accurate comparisons. The remaining 20 approaches indicated that the ogive capacity was similar to those determined by the HCM, though in most instances on the high side.

CONCLUSIONS

1. The preliminary investigation of the ogive curve as developed in this report has some relationship to capacity determinations.
2. The Highway Capacity Manual method of capacity analysis tends to be lower than those based on average starting headways but in general close to ogive capacities.
3. The capacity of the New Jersey Left Turn lane is about 550 vph but could range from 500 to 700 vph.
4. The capacity of a standard left turn lane at an intersection or maneuver ranges from 150 to 225 vph.
5. There is an indication that the New Jersey Left Turn lane increases the potential capacity of an intersection by a considerable amount.

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