DESIGN AND SIGNALIZATION

OF

HIGH-TYPE INTERSECTIONS

by

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INTRODUCTION

The High-Type Signalized Intersection

The at-grade intersection is one of the most critical elements of an urban street system since it exerts the greatest single influence upon 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. Since 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.

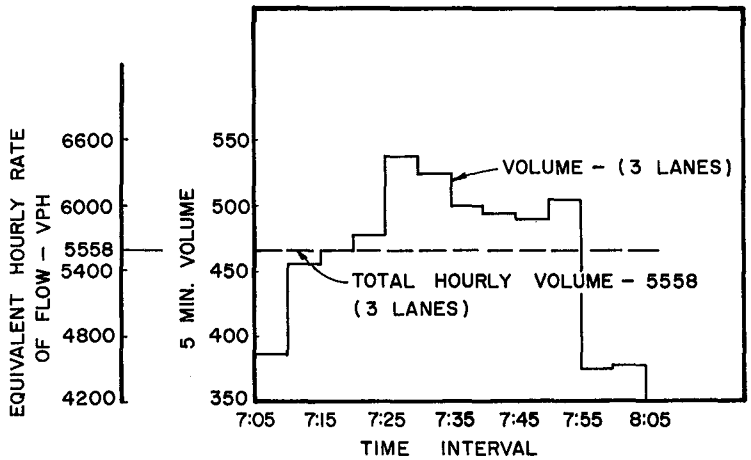
The purpose of this report is to discuss factors affecting the design and signalization of the "high-type" intersection and to develop procedures for designing and signalizing such facilities.

Designing For Peak Flows

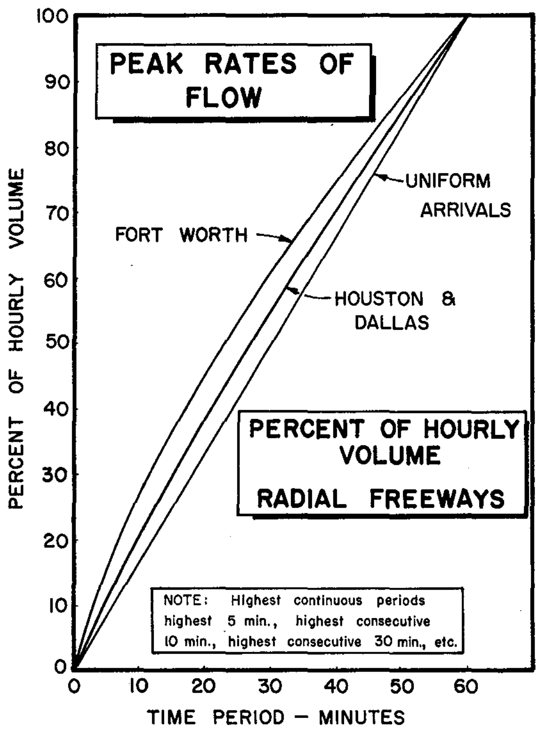
Efforts to increase operational efficiency at signalized intersections have gained impetus in recent years, and the divergency of treatment are testimonials to the many manifestations of the problem. A report (1) prepared for the Bureau of Public Roads presents an analysis of the impact of some 48 variables 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.

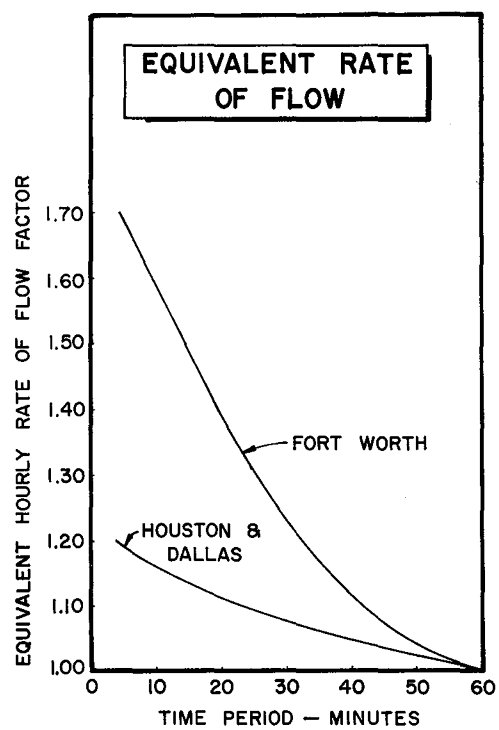
Capelle and Pinnell (3) , in developing a workable formula for determining the capacity of diamond interchanges, reported the presence of a peak period within the peak hour which complicated the signalization analysis and indicated the need for specific study of this factor. The Design Manual (4) used 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 (Figure 1).

Since 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, since the cycle length is not known, the highest one-minute volume should be multiplied by 60 to obtain a design figure. This, of course, raises a serious question in the case of new facilities where no one-minute volumes are available for expansion.



TYPICAL PEAK HOUR 5 MIN. TRAFFIC FLOWS
3 – LANE FREEWAY





PEAKING CHARACTERISTICS ON URBAN RADIAL FREEWAYS

FROM DESIGN MANUAL FOR CONTROLLED ACCESS HIGHWAYS (13)
FIGURES 1-201.4A, B, & C

FIGURE 1

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. Since it is obvious 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 in order 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 which 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 the Peak Period

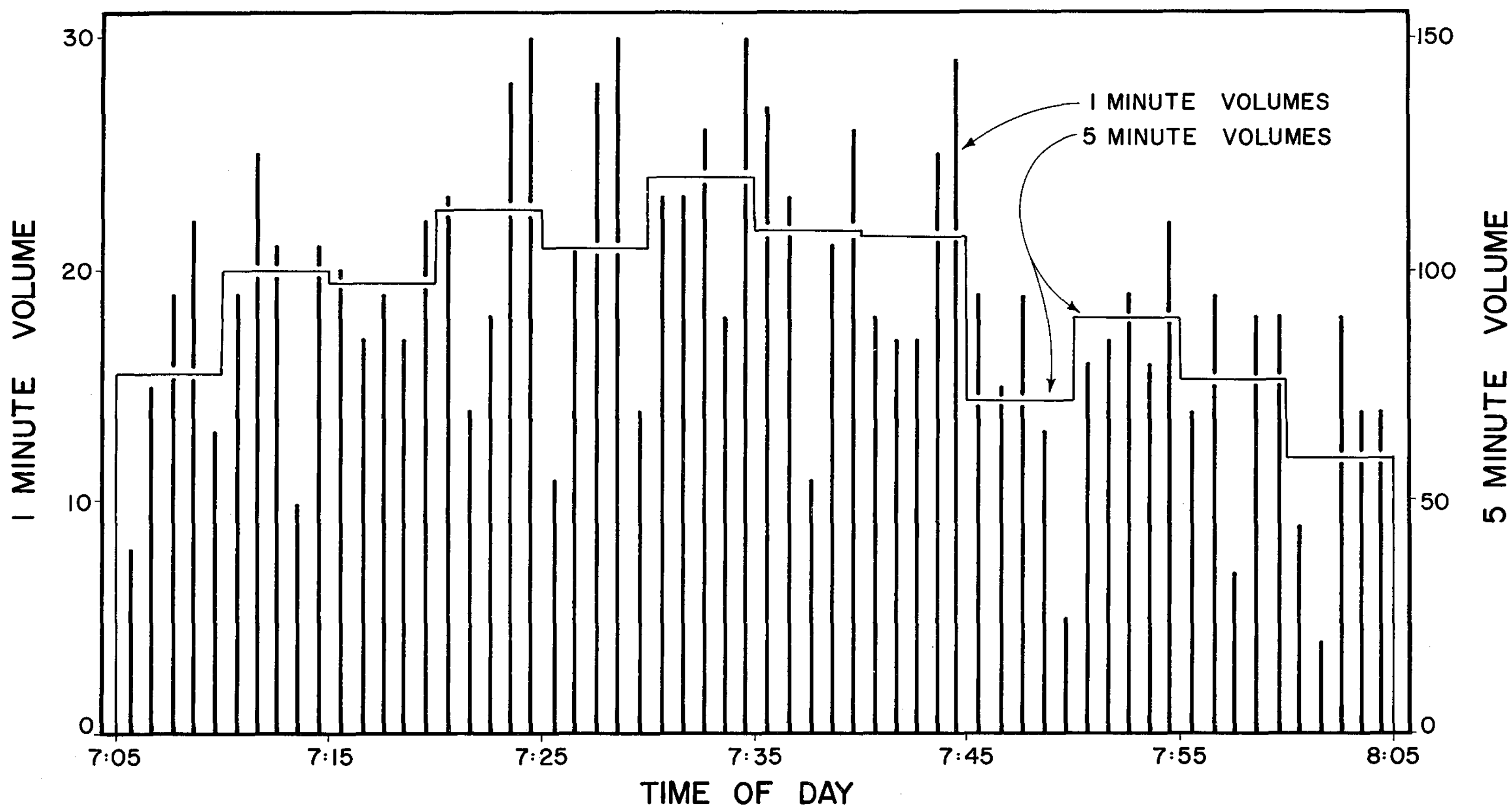
In order to define the peak period within the peak hour, it was necessary to utilize short time intervals for counting traffic demand. Intersection approach volumes recorded in one-minute intervals showed a marked fluctuation and little promise as a practical method of identifying the peak period. Therefore, a five-minute interval was arbitrarily chosen as a basis for grouping (Figure 2). Still, a distinct peak period was not apparent. However, by superimposing the average peak hourly volume on the graph of five-minute volumes (Figure 3) it was apparent that from 7:10 a.m. to 7:45 p.m. the average hourly rate of flow was exceeded. If the mid-points of the five-minute 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 five-minute intervals, exceeded the average hourly rate. The duration of the peak period was approximated either graphically (Figure 3) or algebraically to the nearest minute. The peak hour was simply taken as that 60-minute interval composed of the 12 highest consecutive five-minute volumes.

As was 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 the following equation:

$$\text{magnitude} = \frac{\text{average rate during peak period}}{\text{average rate during peak hour}}$$

This magnitude factor will be greater than 1.000; and if the above expression 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.

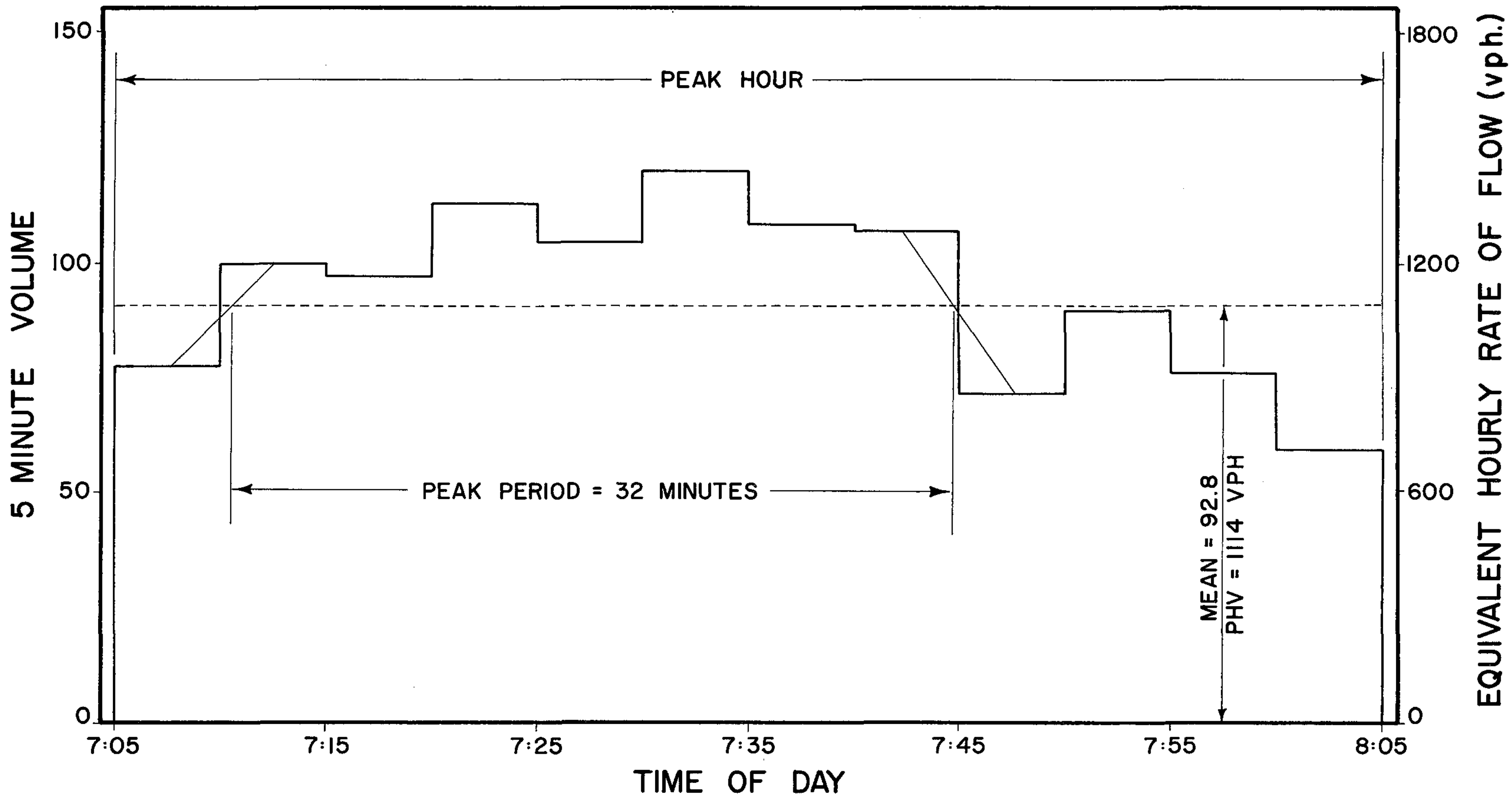


PEAK HOUR I MINUTE TRAFFIC DEMAND

HEIGHTS STREET APPROACH AT SIXTH STREET

HOUSTON, TEXAS, DEC. 20, 1960

FIGURE 2



PEAK HOUR 5 MINUTE TRAFFIC DEMAND

HEIGHTS STREET APPROACH AT SIXTH STREET

HOUSTON, TEXAS, DEC. 20, 1960

FIGURE 3

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 five-minute 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, turning movements, etc. 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:

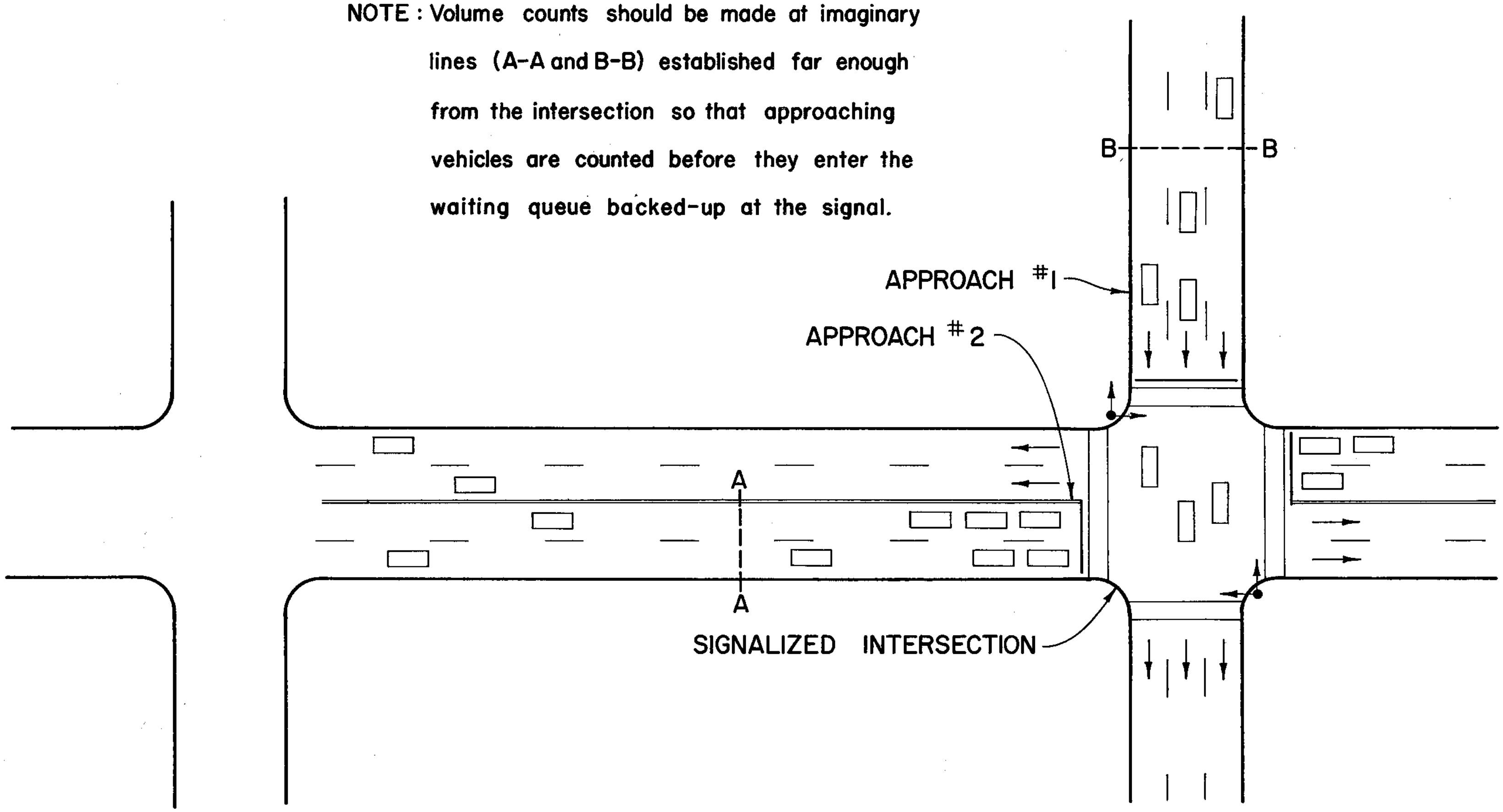
(1) The population of the city in which the intersection is located.

(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 traffic 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 (Figure 4). Since the queue length on the approaches increased greatly during the peak period, counting devices which depended upon road tubes or other stationary sensing devices were too inflexible. It was found, however, NOTE: Volume counts should be made at imaginary lines (A-A and B-B) established far enough from the intersection so that approaching vehicles are counted before they enter the waiting queue backed-up at the signal.



FIELD MEASUREMENT OF DEMAND

FIGURE 4

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 (populations 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)

Since time and financial limitations precluded personal execution of the field work, letters were sent to the traffic engineers of the above 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

Since volume counts recorded at five-minute 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 which approximated the average cycle length at a signalized urban intersection was needed to give an indication of the distribution of arrivals per cycle. A one-minute counting interval corresponding to a minimum

60-second cycle during the morning and afternoon peaks seemed reasonable.

A statistical test of significance was still to be considered. The Chi-square Test seemed appropriate (8), (9), (10). Two restrictions imposed by the Chi-square analysis helped set a maximum limit in the determination of a volume counting interval. Since 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 Chi-square Test. Because peak periods of 15 minutes were conceivable, intervals greater than one minute 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 one-minute intervals.

ANALYSIS OF DATA

Multiple Regression Analyses of Peak Factors

The data obtained from five-minute 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 five-minute 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 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 Central Business District 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 - A.M. and P.M. These combinations are $X_1X_2X_3Y$ (P.M.);

$X_1X_2X_3Y'$ (P.M.); $X_1X_2X_3Y$ (A.M.); $X_1X_2X_3Y'$ (A.M.); $X_1X_2'X_3Y$ (P.M.);

TABLE J

SUMMARY OF PEAK VARIABLES AND PEAK FACTORS

P.M. PEAK						P.M.	CITY	A.M.	A.M. 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	0.43	650	16.3	1.263	1	AMARILLO	1	1.311	21.1	850	0.43	2.1	137
137	2.1	0.34	1101	25.2	1.202	2		2	1.197	26.4	632	0.58	2.1	137
137	3.3	0.62	996	25.0	1.241	3		3	1.380	23.2	551	0.50	2.7	137
137	3.3	0.62	489	17.4	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	0.63	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	0.67	1595	22.3	1.086	7		7	1.146	24.5	1150	0.67	1.9	585
585	1.1	0.11	1292	20.9	1.082	8		8	1.131	25.5	1175	0.10	1.1	585
170	1.9	0.56	1155	16.9	1.196	9	AUSTIN	9	1.120	25.3	779	0.47	3.1	170
170	0.5	0.26	799	29.0	1.157	10		10	1.217	28.3	2171	0.48	3.1	170
170	3.1	0.48	2363	21.5	1.278	11		11	1.084	36.0	1267	0.15	1.0	170
170	3.1	0.47	742	28.1	1.088	12		12	1.168	23.5	717	0.31	1.0	170
941	4.2	0.49	2636	28.3	1.119	13	HOUSTON	13	1.123	32.2	1114	0.44	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	27.9	1.115	15		15	1.104	33.9	1581	0.45	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	0.29	2.3	101
101	1.7	0.20	1120	33.6	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	0.28	1.7	101
101	2.6	0.43	931	21.4	1.159	20		20	1.262	23.1	821	0.28	1.7	101
680	3.3	0.35	907	13.6	1.163	21	DALLAS	21	1.126	25.8	1390	0.42	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	0.61	1264	27.9	1.104	26		26	1.129	31.1	1183	0.61	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								

$X_1X_2'X_3Y'$ (P.M.); $X_1X_2'X_3Y$ (A.M.); and $X_1X_2'X_3Y'$ (A.M.). 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. Since 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).

Chi-Square 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 one-minute 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 one-minute 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 Chi-square 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 Chi-square 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 .54 (from tables of the Poisson distribution). Similarly, during any consecutive pair of one-minute intervals, the probability of 10 or more arrivals for both intervals is .54 x .54. The probability of less than 10 arrivals for both intervals is .46 x .46. Two additional possibilities remain: 1) 10 or more in the first interval and less than 10 arrivals in the second, and 2) the reverse, or less than 10 arrivals in the first one-minute interval and 10 or more in the second. The probabilities of either of these combinations is .54 x .46. The point is that a run of consecutive one-minute arrivals above the mean followed by a similar run of arrivals below the mean might not yield a significant Chi-square 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: 1) determining the observed combinations of arrivals for consecutive intervals, and 2) comparing these with the expected combinations of arrivals for consecutive intervals assuming a Poisson distribution. This latter comparison was also affected by the Chi-square Test.

STUDY RESULTS

Magnitude of the 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 A.M. and P.M. 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 than (X_2), the distance between the intersection and the CBD.

Substituting in the general regression equation, the magnitude of the A.M. peak period becomes

$$\hat{Y}' = \bar{y}' + b_{Y1.23} (X_1 - \bar{x}_1) + b_{Y2.13} (X_2' - \bar{x}_2') + b_{Y3.12} (X_3 - \bar{x}_3)$$

where $\bar{Y}' = 1.1795$

$\bar{x}_1 = 424.3$

$\bar{x}_2' = .4546$

$\bar{x}_3 = 1193.9$

b = (see Appendix D)

$$Y' \text{ (A.M.)} = 1.222 - .000125X_1 + .09X_2' - .000027X_3 \qquad \text{Eq. (1)}$$

Similarly, the following values for the P.M. peak:

$\bar{Y}' = 1.1644$

$\bar{X}_1 = 392.3$

$\bar{X}_2' = .4394$

$X_3 = 1257.9$

b = (see Appendix D)

$$\hat{Y}' \text{ (P.M.)} = 1.228 - .000145X_1 - .11X_2' + .000033X_3 \qquad \text{Eq. (2)}$$

If Equations (1) and (2) are averaged, term by term, the result is a common expression for both peaks, except for one sign. (A.M., positive; P.M., negative).

$$\hat{Y}' = 1.225 - .000135X_1 \pm (0.1X_2' - .00003X_3) \qquad \text{Eq. (3)}$$

where X_1 = population of city / 1000

$$X_2' = \frac{\text{distance between intersection and CBD}}{\text{distance from CBD to City Limits}}$$

X_3 = peak hourly volume per approach (PHV)

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

Since by definition . . . .

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

where $m' = X_3/C'$ (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.

$$m = \frac{X_3}{C'} [1.125 - .000135X_1 \pm (.1X_2' - .00003X_3)] \qquad \text{Eq. (4)}$$

TABLE 2[†]

RESULTS OF ANALYSES FOR MAGNITUDE OF PEAK PERIOD (Y')

Analysis	"F"-test	"t"-test				R^2
		X_1	X_2 or	X_2'	X_3	
P.M. Peak (X_2')	**	**		N	N	36.8%
A.M. Peak (X_2')	*	*		N	N	31.0%
P.M. Peak (X_2)	*	**	N	N	N	29.9%
A.M. Peak (X_2)	*	*	N	N	N	28.0%

TABLE 3[†]

RESULTS OF ANALYSES FOR DURATION OF PEAK PERIOD (Y)

Analysis	"F"-test	"t"-test				R^2
		X_1	X_2 or	X_2'	X_3	
P.M. Peak (X_2')	N	N		N	N	14.3%
A.M. Peak (X_2')	N	N		N	N	8.7%
P.M. Peak (X_2)	N	N	N		N	14.3%
A.M. Peak (X_2)	N	N	N		N	9.2%

† Results taken from calculations in Appendices D and E

N Non-significant relationship

* Significant relationship

** Highly significant relationship

R^2 Percentage of variance explained by the multiple regression analysis

Duration of the 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. It is noteworthy that although this regression was not significant, 14.3 per cent of the variance of the duration of the P.M. peaks could be attributed to these three factors - population, location, and hourly volume. Thus, the best estimates of the duration of either an A.M. or P.M. peak period were their respective means.

The estimates for the duration of the A.M. and P.M. peak periods are obtained as follows:

$$\hat{Y} = \bar{y} \pm (t_{.05}\, S_{\bar{y}})$$

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

$\bar{y}$ = the mean of the sample durations

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

Thus, $\hat{Y}$ (A.M.) $= 26.69 \pm (2.052)\ (.92) = 26.69 \pm 1.89$ Eq. (5)

$\hat{Y}$ (P.M.) $= 25.04 \pm (2.042)\ (1.18) = 25.04 \pm 2.41$ Eq. (6)

Distribution of Arrivals

The Chi-square 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 .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 .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 indentical with the postulated (Poisson) distribution.

TABLE 4

SUMMARY OF CHI-SQUARE TESTS FOR THE DISTRIBUTION OF ARRIVALS†

Study No.	PEAK HOUR Chi-square	d. f.	Probability	PEAK PERIOD Chi-square	d. f.	Probability
1	13.1	7	.10>P >.05	0.9	3	P > .07
2	25.2**	7	P < .001	10.6*	3	.02>P >.01
3	64.3**	6	P < .001	2.1	2	.50>P >.30
4	17.0*	7	.02>P > .01	2.2	1	.20>P > .10
5	14.2*	6	.05>P > .02	4.2	4	.50>P > .20
6	4.0	7	P > .70	2.7	2	.30>P > .20
7	22.7**	6	P < .001	3.2	2	P = .20
8	24.3**	7	P = .001	0.4	1	P = .50

* 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.

† See Appendix F

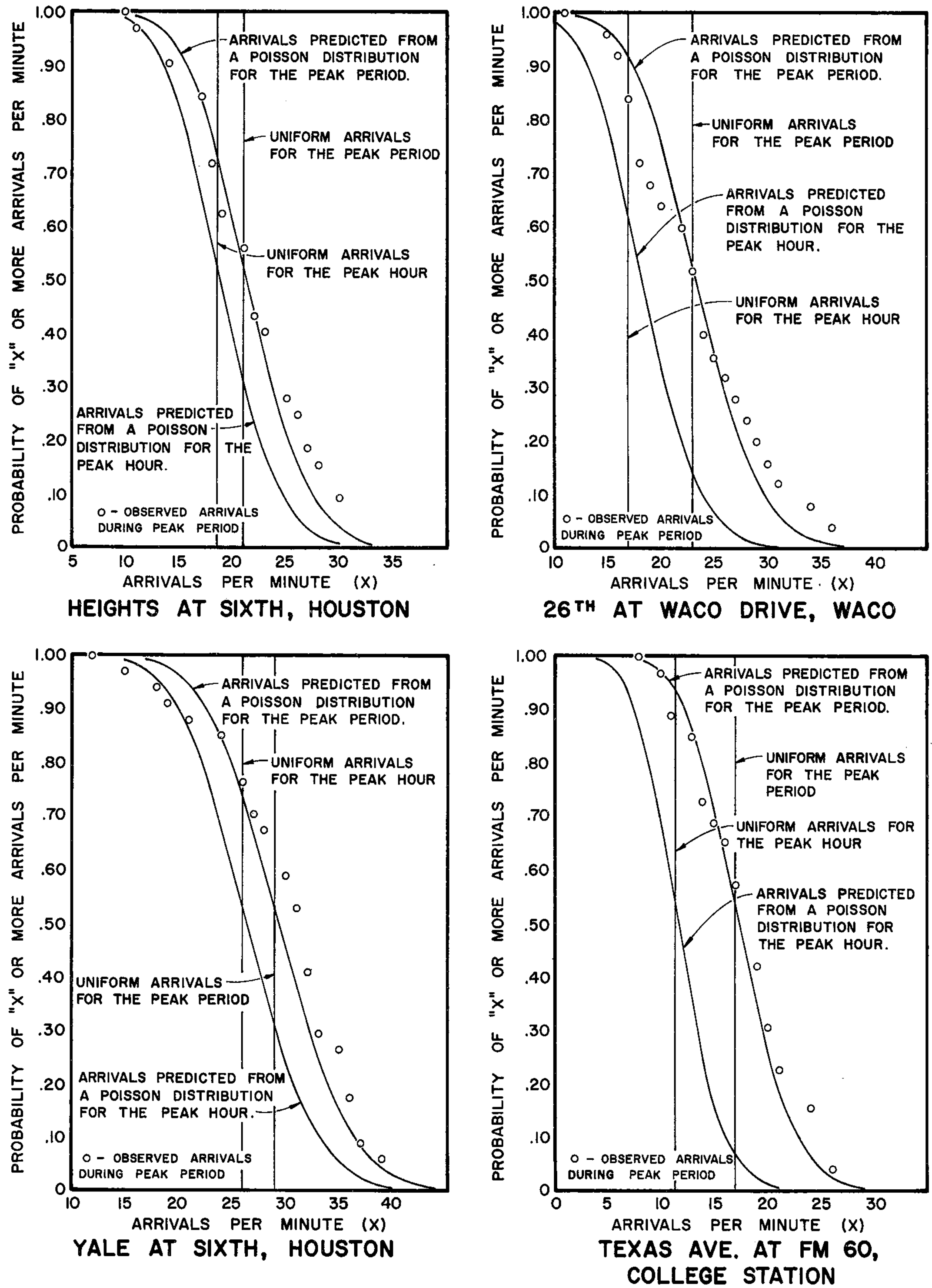
The check for the independence of arrivals for successive one-minute intervals proved to be academic (Appendix H). The observed variations in arrivals in consecutive intervals agreed almost exactly to that expected for a Poisson distribution. This property of recovery - or the tendency of fluctuations 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: 1) Poisson arrivals throughout the peak hour, or 2) 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 distribution during the peak period, while 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.

Importance of Findings

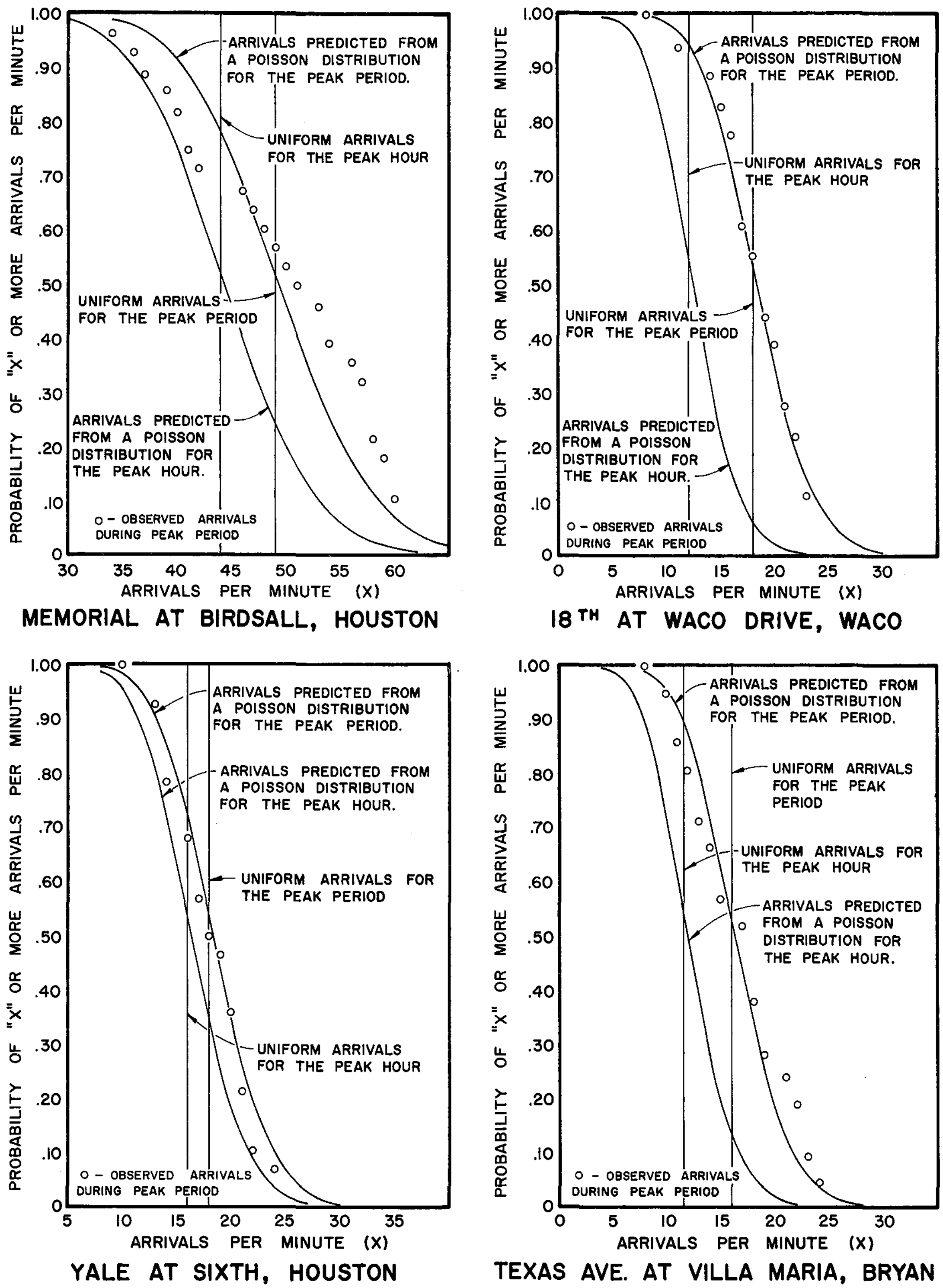
Referring again to Figures 5 and 6, by definition, the average of arrivals during the peak period divided by the average of arrivals per peak hour is actually the magnitude factor (Y') evaluated through the multiple regression analysis. This magnitude factor serves a function analogous to the conversion 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



RELATIONSHIP BETWEEN OBSERVED AND PREDICTED ARRIVALS DURING A.M. PEAK PERIODS

FIGURE 6



RELATIONSHIP BETWEEN OBSERVED AND PREDICTED ARRIVALS DURING P.M. PEAK PERIODS

FIGURE 6

new facility this factor would be estimated by Equation (3). 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 chapter.

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. Since 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.

In order to visualize intersection performance, it is convenient to plot the conditions on a time-space diagram as illustrated 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 the following equation:

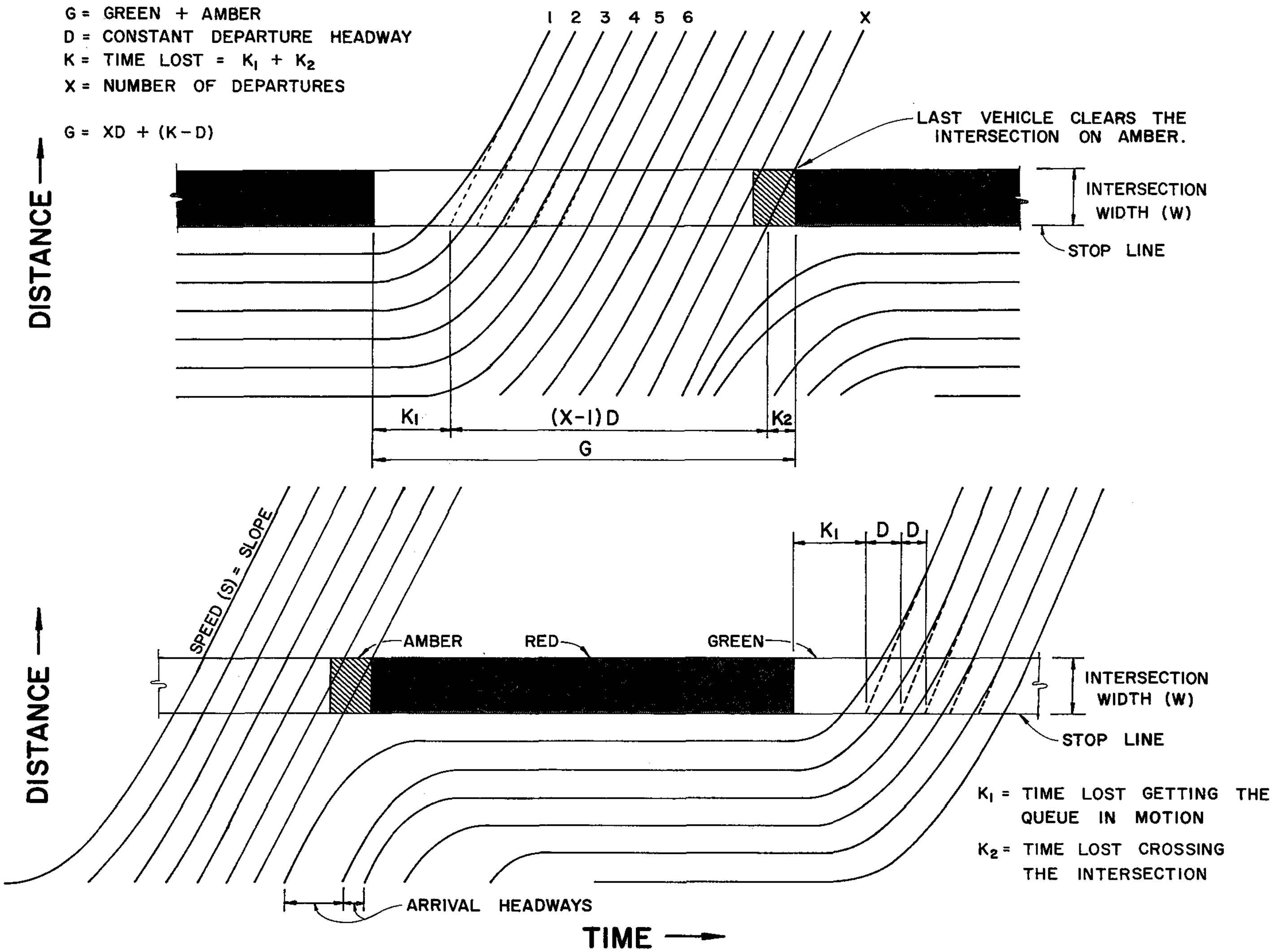
$$\text{average minimum headway} = \frac{\text{Time}}{\text{Volume}} \quad \text{or}$$

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

where $K = K_1 + K_2$ and where K_1 is the starting delay for getting the entire queue into motion and K_2 is the time necessary for the last vehicle to cross the intersection. Since the last vehicle is legally allowed to cross on amber, the "G" value equals green plus amber time. Rewritten, the expression becomes:

$$G = (x-1)\,D + K \quad \text{or,} \qquad \text{Eq. (7)}$$

$$G = xD + (K - D) \qquad \text{Eq. (8)}$$



TIME-SPACE RELATIONSHIPS FOR 2 PHASE SYSTEM

FIGURE 7

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 departure headway and a starting delay.

It is noteworthy that in this treatment of capacity, amber time does not directly affect the phase lengths or cycle lengths, and hence capacity. Since 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 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 two seconds to four seconds on an intersection approach theoretically should not reduce the capacity, since 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 utilized. The measurements which are summarized in Appendix D were reduced by methods outlined in Table 5 and then classified as to movements (Table 6). Thus, values of D = 2.0 seconds and K_1 = 4.0 seconds seem representative. K_2 equals approximately 2.0 seconds (based on an intersection width of 50 feet, a speed of 30 mph, and an allowance of 30 feet for the length of vehicles).

TABLE 5

TYPICAL REDUCTION OF TIME INTERVALS BETWEEN SUCCESSIVE VEHICLES INTO DEPARTURE HEAD-WAYS AND STARTING DELAYS †

	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	.1
	3-4	2.1	2.0	.1
	4-5	2.0	2.0	0.
	5-6	2.0	2.0	0.
Summations:	6	13.6	10.0	3.6

D (departure headway) = 2.0 seconds

K_1 (starting delay) $= \sum(I-D) = 3.6$ seconds

TABLE 6

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

Type Movement	D	K_1
Through Movement	2.0	4.0
Left Turn	2.0	3.9
Right Turn	2.0	4.1
Side by Side Turns		
Inside Lane	2.2	4.7
Outside Lane	2.4	5.3

† See Appendix D for data.

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.

$$V = \left(\frac{3600}{C}\right) x$$

where $x = \dfrac{G - (K-D)}{D}$ from Equation 8.

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

$$\Sigma V = \left(\frac{3600}{C}\right) \Sigma x$$

$$\Sigma V = \left(\frac{3600}{C}\right) \frac{\Sigma G - \phi\,(K-D)}{D}$$

where $C = \Sigma G$

ϕ = number of phases

$$\Sigma V = \left(\frac{3600}{C}\right) \frac{C - \phi\,(K-D)}{D} \qquad \text{Eq. (10)}$$

$$\Sigma V = \left(\frac{3600}{D}\right) - \frac{3600\,\phi\,(K-D)}{CD}$$

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

$$\Sigma V\ (\phi = 3) = 1800 - \frac{21,600}{C} \qquad \text{Eq. (11)}$$

$$\Sigma V\ (\phi = 4) = 1800 - \frac{28,800}{C} \qquad \text{Eq. (12)}$$

Tabulations of (ΣV) for various cycle lengths (C) appear in Table 7. It is seen that as C approaches infinity, ΣV approaches 1800 vehicles per hour per lane.

Last, Equation 10 may be solved for cycle length (C):

$$C = \frac{3600\,\phi\,(K-D)}{3600 - D\,\Sigma V} \qquad \text{Eq. (13)}$$

TABLE 7

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

Cycle Length C Seconds	Summation of Hourly Critical Lane Volumes $1800 - \frac{7200\phi}{C}$	
	$\phi = 3$	$\phi = 4$
40	1260	1180
50	1368	1224
60	1440	1320
70	1491	1389
80	1530	1440
90	1560	1480
100	1584	1512
110	1604	1538
120	1620	1560
∞	1800	1800

It should be remembered that Equations 7 through 11 are based on the assumption of uniform arrivals for every cycle over a 60-minute 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.

Since it has been established (Figures 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. Equation 14 is the cumulative 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.

$$P_{(x+1)(m)} = \sum_{x+1}^{\infty} \frac{m^{x+1} e^{-m}}{(x+1)!} \qquad \text{Eq. (14)}$$

where $m = V \div (3600/C)$

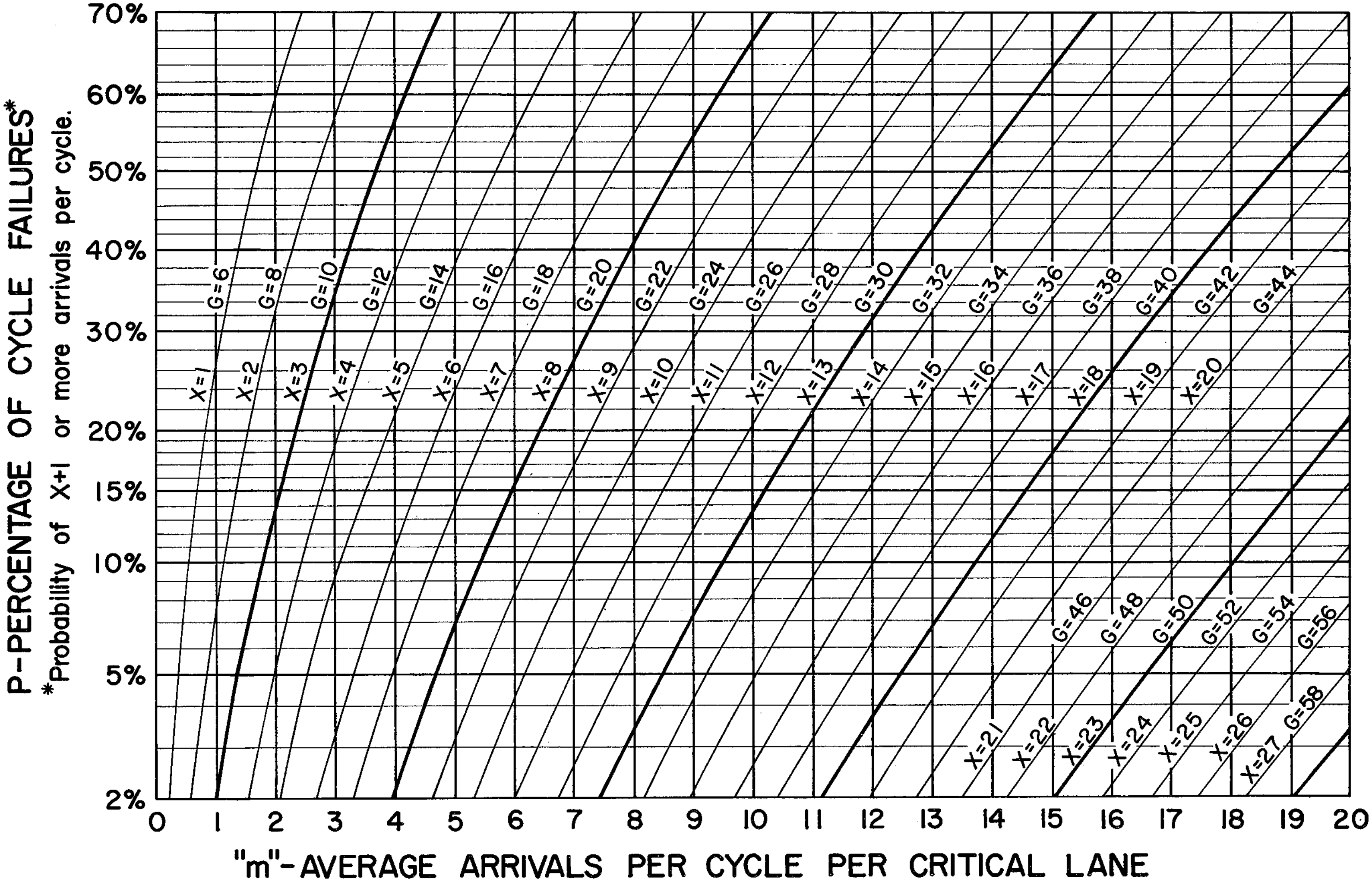
$$x = \frac{G - (K-D)}{D} \quad \text{from Equation 8}$$

$$C = \Sigma G$$

These four equations can be reduced by successive approximations. This reduction is greatly facilitated through the use of a graph of cumulative Poisson curves (Figure 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 illustrate seven steps to be followed in the design and signalization of a future high-type intersection. These steps are as follows:



GREEN REQUIREMENTS FOR NON-CONFLICTING MOVEMENTS

FIGURE 8

STEP 1: LIST CONDITIONS

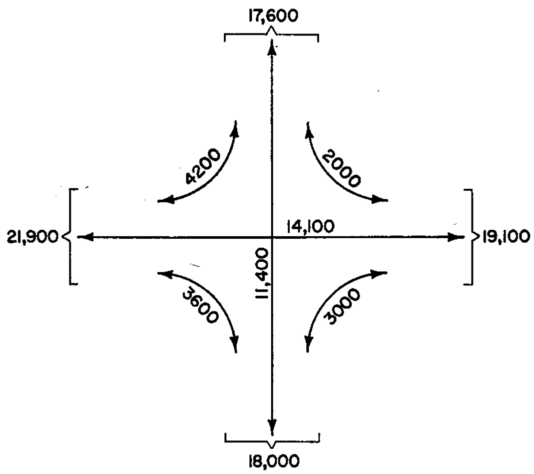
(1) High-Type Intersection

(2) Population of City (1981)=280,000

(3) Location of Intersection (1981)

distance from CBD = 4.0 miles

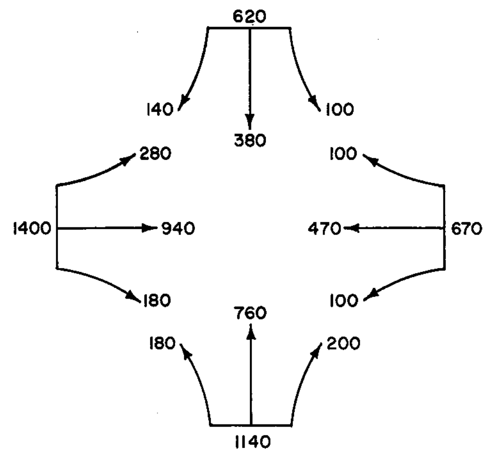
distance from City Limits = 2.6 miles



STEP 2: FIND PEAK HOUR VOLUMES

K=10 % (Peak Hour Factor)

D=67% (Directional Distribution)



NOTE: Only the P.M. peak is considered for the purpose of this example.

STEP 3: FIND PEAK MAGNITUDE FACTOR FOR EACH APPROACH

$$\hat{Y}' = 1.225 - .000135X_1 \pm (0.1X_2' - .00003X_3)$$

Where X_1 (pop.÷1000) = 280

X_2' (ratio dist.) = 4.0÷(4.0+2.6)=6.1

X_3 (south approach) = 1140

X_3 (west approach) = 1400

X_3 (north approach) = 620

X_3 (east approach) = 670

P.M. Peak:

$\hat{Y}$ (P.M.) = $1.225 - .038 - .016 + .00003X_3$

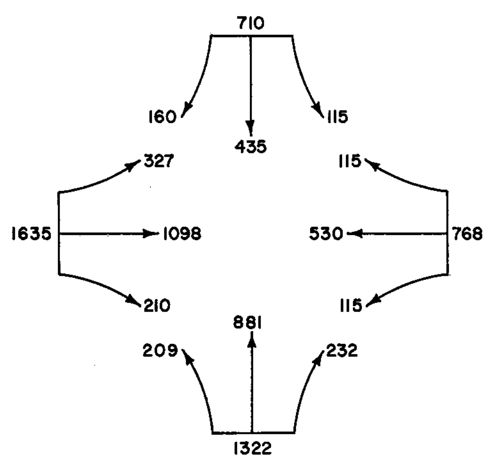
$\hat{Y}'$ (south approach) = 1.160

$\hat{Y}'$ (west approach) = 1.168

$\hat{Y}'$ (north approach) = 1.145

$\hat{Y}'$ (east approach) = 1.146

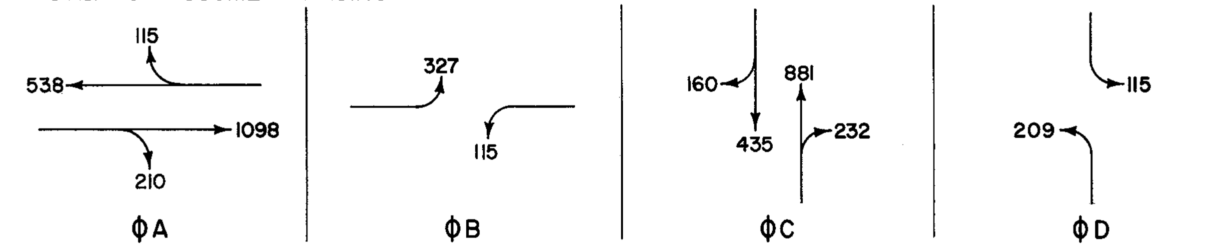
STEP 4: SHOW ADJUSTED HOURLY RATES OF FLOW FOR PEAK PERIOD



CAPACITY-DESIGN PROCEDURE
(STEPS 1 TO 4)

FIGURE 9

STEP 5: ASSUME PHASING

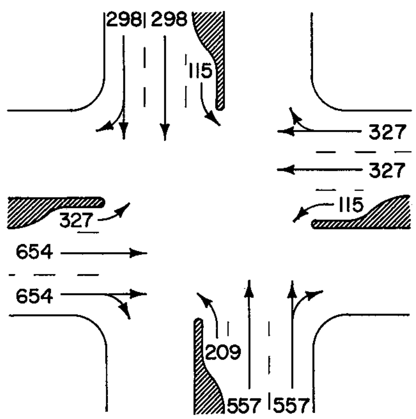


STEP 6: ASSUME VARIOUS LANE COMBINATIONS. DETERMINE CRITICAL LANE VOLUMES (V) AND MINIMUM CYCLE LENGTHS (C).

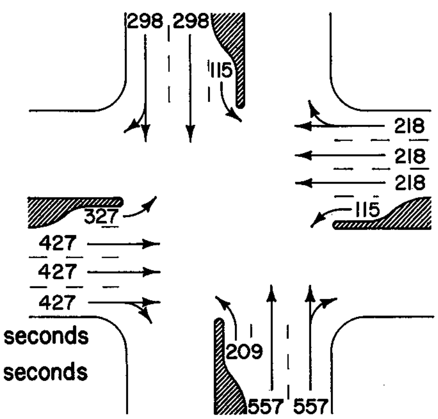
$$C = \frac{3600\,\phi(K-D)}{3600 - D\Sigma V} \quad \text{where } \phi = 4,\ K = 6.0,\ D = 2.0$$

$$C = \frac{57{,}600}{3600 - 2.0\Sigma V}$$

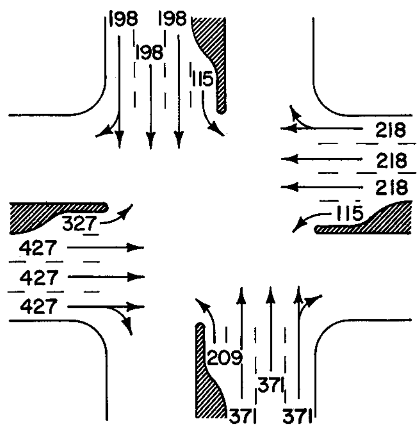
NOTE: Equation assumes uniform arrivals for the Peak Period.



$V_A = 654$
$V_B = 327$
$V_C = 557$
$V_D = 209$
$\Sigma V = 1747$



$V_A = 427$
$V_B = 327$
$V_C = 557$
$V_D = 209$
$\Sigma V = 1520$
C = 106.2 seconds
say 100 seconds



$V_A = 427$
$V_B = 327$
$V_C = 371$
$V_D = 209$
$\Sigma V = 1334$
C = 63.0 seconds
say 60 seconds

CAPACITY-DESIGN PROCEDURE
(STEPS 5 AND 6)

FIGURE 10

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.

CAPACITY - DESIGN PROCEDURE - STEP 7

FIGURE 11

Step 1 - Determine the three conditions (population, location, and volumes) which affect the magnitude of peak period.

Step 2 - Since the volumes are given in terms of ADT, they must be converted to Peak Hourly Volumes. In an actual problem the A.M. peak would also be checked.

Step 3 - The Peak Magnitude Factor ($\hat{Y}'$) for each approach is calculated using the regression equation.

Step 4 - The Peak Magnitude Factors ($\hat{Y}'$) are applied to the Peak Hourly approach volumes to arrive at an hourly rate of flow equivalent to the arrivals during the peak period.

Step 5 - Consistent with the assumption of a high-type facility, all conflicting movements must be separated by the signal phasing.

Step 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 distribution during the peak period. The maximum lane volume required to move on a given phase is called the <u>critical lane volume</u>. The sum of these critical lane volumes for all phases provides the basis for calculating the <u>minimum cycle length</u> either by use of Equation 13 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.

Step 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 (Figure 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.

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 utilized.

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 minutes for the P.M. 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 A).

Table A

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"	10.7-minutes	10.0-minutes

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 per cent during the peak period represents a practical design level (remembering that this would be only about 10 to 15 per cent of the peak hour). Step 7 for Designs B and C could be repeated assuming longer cycle lengths (say 120 seconds and 80 seconds, respectively) to obtain a lower level of failure. The excessive cycle length of 120 seconds required for Design "B" might preclude its use. However, since the conditions utilized in the calculations are for projected volume data, design alternative B might possibly offer 15 years of desirable operation and thus still merit consideration.

SUMMARY

Conclusions

It is apparent that a problem exists in deciding what volumes should be used either 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 minutes) within the design hour. In many locations, this represents an intolerable situation. Furthermore, since there are two peaks each day (morning and evening), ten each week, and 520 a year, an intersection could very conceivably be underdesigned well over 200 hours a year. Thus, the "Thirtieth 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:

   $$Y' = 1.225 - .000135X_1 \pm (0.1X_2' - .00003X_3)$$

   where the factors within the parentheses are to be added for the A.M. peak and subtracted for the P.M. peak, and
   X_1 = population of the city ÷ 1000

   X_2' = location of the intersection as a fraction of the distance from CBD to the city limits

   X_3 = peak hourly volume (PHV) for the approach

3. The duration of the peak period was not significantly related to the variables - population, location, and volume. Therefore, the mean

for each period - A.M. and P.M. - represented the best available estimate. These values were 26.69 minutes for the A.M. peak based on 28 approaches and 25.04 minutes for the P.M. peak based on 32 approaches.

4. Chi-square tests made on arrivals during the A.M. and P.M. 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 since 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 (Figures 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 (Figure 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 percentage 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 seconds 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.

It should be noted that 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. Many 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 per cent for directional distribution and, say, 11 per cent for the "K" ratio of DHV to ADT, this gives an hourly volume of 9,000 x .67 x .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 A.M. and P.M. 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 A.M. peak and away from the CBD in the P.M. peak.

2. Counts should be made for at least 16 consecutive five-minute periods, so as to bracket the peak hour.

3. The five-minute 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 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 the above information, data forms (Figure 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:

DATA FORM FOR INTERSECTION STUDY

FIGURE 12

APPENDIX B
SUMMARY OF FIVE-MINUTE VOLUME COUNT STUDIES
Amarillo

A.M. Approach Number 1	A.M. Approach Number 2	A.M. Approach Number 3	A.M. Approach Number 4	Name of Street and Location		P.M. Approach Number 1	P.M. Approach Number 2	P.M. Approach Number 3	P.M. Approach Number 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		5 Minute Arrivals			
				A.M.	P.M.				
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 5	6	7	8			P.M. APPROACH NUMBER 5	6	7	8
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						A. M. APPROACH NUMBER			
9	10	11	12			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		5 Minute Arrivals			
				A.M.	P.M.				
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						P.M. APPROACH NUMBER			
13	14	15	16			13	14	15	16
Heights at Sixth	Heights at Eleventh	Yale at Sixth	Houston at Washington	Name of Street and Location		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						P. M. APPROACH NUMBER			
25	26	27	28			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		5 Minute Arrivals			
				A.M.	P.M.				
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	5 Minute Arrivals			
P.M.				
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.

APPENDIX C

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
$(C_{11})^{1/2} = .000767$		Decoded:	1	.000000589	-.00004044	-.000000099
$(C_{22})^{1/2} = .1455$			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	-.000000141
$(C_{22})^{1/2}$ = 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 ($X_1 X_2' X_3$)

	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
$\Sigma xy'$	-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
$\Sigma xy'$	-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 = .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 (A.M. PEAK)

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

* Identifies the beginning and ending of the peak hour.

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

SUMMARY OF ONE-MINUTE VOLUME COUNT STUDIES (P.M. PEAK)

Location	Time	4:30	4:40	4:50	5:00	5:10	5:20	5:30	5:40	5:50
18th Street at Waco Drive, Waco	:00	17*	5	11	5	14	11	6	15	7
	:01	10	8	14	5	18	16**	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**	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*	14	6	4
Yale Street at 6th Street, Houston	:00		26*	12	17	21	10	22		
	:01		8	19	16	13	9	16		
	:02		13	18**	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**	13	4*		
Texas Avenue at Villa Maria, Bryan	:00	4*	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**	19	14**	9*			
Memorial at Birdsell, Houston	:00	33	33*	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**	35	42		
	:06	41	42	64	50	52	41	54		
	:07	31	50	37	40	31	37	43		
	:08	31	59**	58	59	50	46	38		
	:09	33	34	60	40	34	30	41*		

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

APPENDIX G

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)			
$\leq$ 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; $.10 > P > .05$

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(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; $P < .001$

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(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

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(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

Arrivals per Minute X	Observed Frequency f_x	Predicted Frequency F_x	f_x^2/F_x
(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 Chi-square test (Appendix G), an investigation of the independence of arrivals for successive intervals was made. The data for the following example was taken from the one-minute volume count study for Heights Street at Sixth Street in Houston (Appendix F). A graph of these arrivals appears in Figure 2.

The analysis below consists of two parts: 1) the determination of the observed combinations of arrivals with respect to the mean for successive one-minute intervals, and 2) the comparison of the observed with the expected combinations by the Chi-square test assuming a Poisson distribution.

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

TIME A.M.	ARRIVALS PER MINUTE		COMBINATIONS 1	2	3	4
	x_1	x_2	$x_1 \geq m$ $x_2 \geq m$	$x_1 < m$ $x_2 \geq m$	$x_1 > m$ $x_2 < m$	$x_1 < m$ $x_2 < m$
7:12	21	10			1	
:13	10	21		1		
:14	21	19			1	

7:15	19	17				1
:16	17	19				1
:17	19	17				1
:18	17	22		1		
:19	22	23	1			
:20	23	14			1	
:21	14	18				1
:22	18	28		1		
:23	28	30	1			
:24	30	11			1	
:25	11	21		1		
:26	21	28	1			
:27	28	30	1			
:28	30	14			1	
:29	14	23		1		
:30	23	23	1			
:31	23	26	1			
:32	26	18			1	
:33	18	30		1		
:34	30	27	1			
:35	27	23	1			
:36	23	11			1	
:37	11	21		1		
:38	21	26	1			
:39	26	18			1	
:40	18	17				1
:41	17	17				1
:42	17	25		1		
:43	25	End of Peak				
Total Observed Frequency (f)			9	8	8	6
Expected Frequency (F)†			8.7	7.75	7.75	6.0
(f-F)			0.3	0.25	0.25	-0.8
$(f-F)^2/F$			.01	.01	.01	.09

$$\chi^2 = \Sigma (f-F)^2/F = .12$$ on 3 degrees of freedom; $P = .99$

Since a probability equal to or less than .05 is needed to reject the hypothesis that arrivals during the peak period are of a Poisson distribution, the hypothesis is definitely accepted.

† Expected frequencies (F) were determined by multiplying the total number of pairs of intervals (31) by the probability of each of the four combinations.

(continued)

Thus, assuming a Poisson distribution, 53% of the intervals would have equal to, or more than the mean number of arrivals ($P_{x \geq m} = .53$). The probability of both x_1 and x_2 exceeding m would be .53 times .53 which equals .28. Lastly, .28 multiplied by 31 gives the expected frequency (F=8.7).

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 (4-5 and 5-6)	18	2.0 (4-5 and 5-6)	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 (3-4 to 5-6)	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.