Gap Availability Studies

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•THE EXPRESSWAY Surveillance Project was established in April 1961 as a part of the research program of the Illinois Division of Highways, under the supervision of the Bureau of Research and Planning. The project is being financed with Highway Planning Survey funds made available through the United States Federal-Aid Highway Acts with Illinois, Cook County, and Chicago contributing the necessary matching funds. An advisory committee, consisting of representatives of the four cooperating agencies, was appointed to review the progress of the project and to advise on recommendations for future experimental work.

The immediate objective of the project is to develop, operate and evaluate a pilot network information and control system to reduce travel time and to increase traffic flow. Successful progress could lead eventually to a centralized information and control system for the entire Chicago Metropolitan Expressway and major street network system.

The approach being used in accomplishing this objective is frequently referred to as a case study or pilot study approach. A typical portion of the Metropolitan Chicago highway network system was selected as the laboratory or demonstration area, and a pilot detection system is being used to measure the existing traffic patterns from which control plans can be developed. Through experimentation with control, the pilot detection system gradually will be converted to a pilot network information and control system. When complete, the system will be evaluated in terms of road-user benefits and system costs. Adminstrative decisions can then be made as to the possible extension of the system.

The first major phase of the project was the development of a pilot detection system consisting of operational studies, surveillance equipment evaluation, system design and installation. The pilot detection system became operational in October 1962 and the work leading to this development has been described by May et al. (1). During 1963, a series of studies and field experiments was undertaken to control the amount of ramp traffic and to maintain the maximum flow on the expressway. Initial experimentation with manual ramp closure and metering was undertaken in May and June, and initial experimentation with automatic ramp metering began in September. The results of this work were reported by May (2).

The flexibility for control and the public acceptance of the traffic-adjusted ramp metering scheme, combined with the task of metering more critical ramps in the near future, has stimulated the project staff to attempt to improve the existing ramp metering techniques. This paper describes one approach being undertaken for doing this.

INITIAL EXPERIENCE WITH AUTOMATIC RAMP METERING

The results of the initial experience with automatic ramp metering were most gratifying. More than 97 percent of the ramp users operated their vehicles in a satisfactory manner and complied with the regulations pertaining to the ramp metering operations. The traffic operations in the freeway ramp merging area were improved as indicated by a reduction in the number of stopped vehicles in the merging area. The queue of vehicles on the ramp was normally less than ten vehicles and did not interfere with surface street operations.

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The initial configuration of the ramp metering equipment is shown in Figure 1, and photographs of the installation are shown in Figure 2. A schematic diagram of the basic steps in controlling the ramp metering device (Fig. 3) indicates that a measurement of occupancy (or volume) is obtained on the expressway just upstream of the on-ramp and transmitted to the computer center for computation. If no control is required (free-flow conditions, occupancy less than 15 percent), the ramp signal rests in green. In the event of impending congestion, one of five metering rates is selected, depending on the occupancy level. The relationship between mainline occupancy and the permitted metering rate originally employed is given in Table 1. In the event of expressway congestion, it was originally the intent that the ramp would be manually closed, and when the vehicles already on the ramp were cleared, the ramp signal would be locked in red.

MODIFICATIONS TO AUTOMATIC RAMP METERING

Since September 16, 1964, the ramp metering device at First Avenue has been in operation each weekday afternoon (except holidays) from approximately 3:30 until 6:30 PM. A number of operational and equipment studies have been conducted to determine if the ramp metering could be further improved.

In December and January, several changes were made at the ramp. The stop line was moved from a point 4 ft in advance of the traffic signal to the location of the traffic signal because a few vehicles each day were stopping short of the first detector loop and the traffic signal remained red. Two additional detectors were installed at the top and bottom of the ramp for detecting when a queue existed behind the ramp signal and when a vehicle had slowed down or stopped in the merging area. The detector at the top of the ramp is also used to facilitate changing the ramp signal from resting in green to resting in red when metering is begun. An advanced flasher was installed at the top of the ramp to replace the manually changed sign for the purpose of automating the ramp operation while still warning the ramp users when metering is in operation. A counter has been installed at the ramp to count the number of vehicles which violate the ramp signal operation.

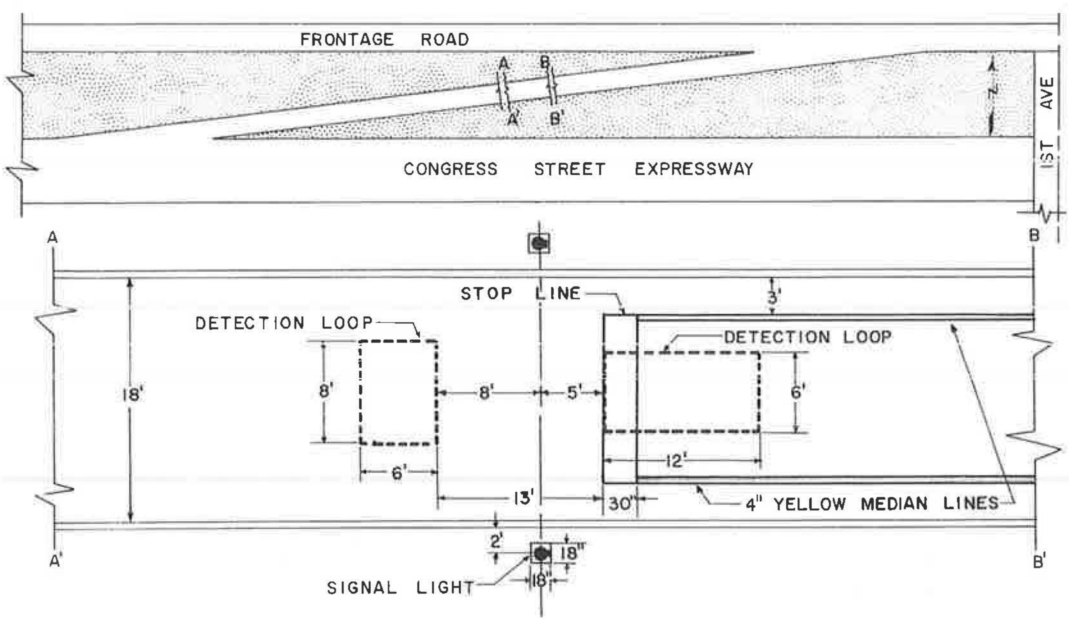

Figure 1. Configuration of ramp metering equipment.

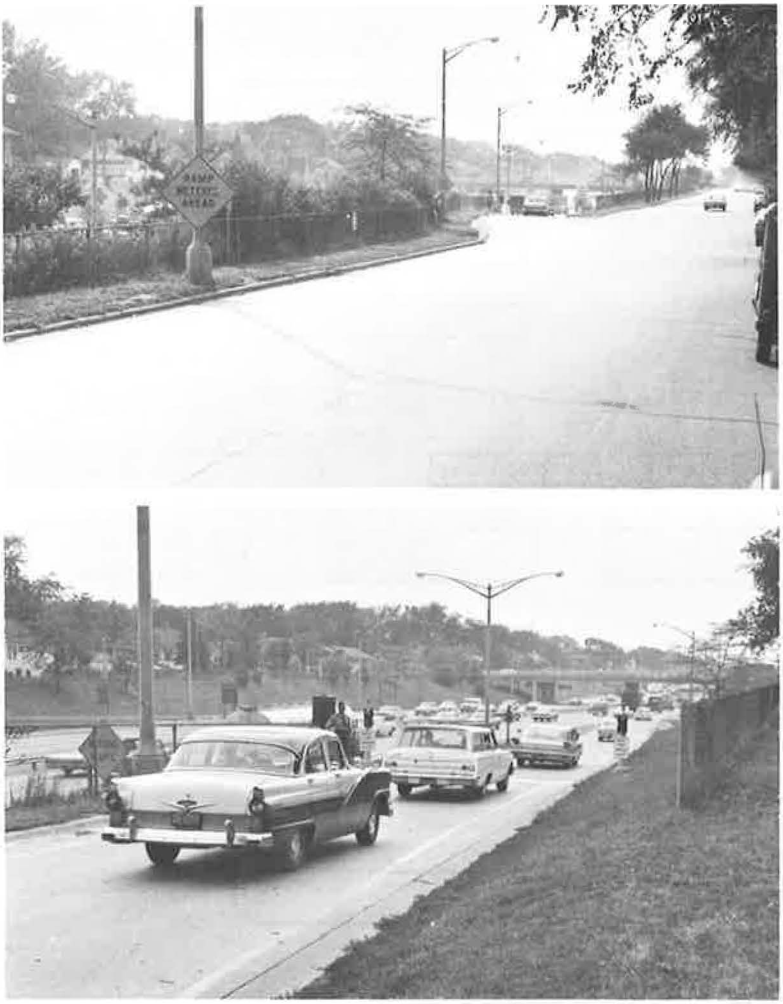

Figure 2. Ramp metering equipment.

STUDIES FOR IMPROVING AUTOMATIC RAMP METERING

In October, a comparison was made of two metering schemes: one based on center lane occupancy and the second on a combination of total directional volume and center lane occupancy. Before this study, efforts were made to experiment with several plans for each of the two schemes to compare the best plan devised for each scheme. The center lane occupancy scheme employed is given in Table 2, and the combined directional volume and center lane occupancy scheme employed is given in Table 3.

Both schemes worked satisfactorily and there was little difference between the two methods. It should be pointed out, however, that although the mainline flow is heavy (5,200 to 5,400 vph approaching the ramp), congestion does not normally occur at this location and, therefore, the schemes were not critically tested.

In January, a series of experiments at First Avenue was proposed for the purpose of evaluating the use of measurements in different lanes and with different response times. This study is still under way, but the preliminary results indicate that the measurement of center lane occupancy with a response time three (an averaging period of approximately 45 sec), is as good or better than other schemes tested to date.

A study now in the planning stage is the possible overriding of the basic metering

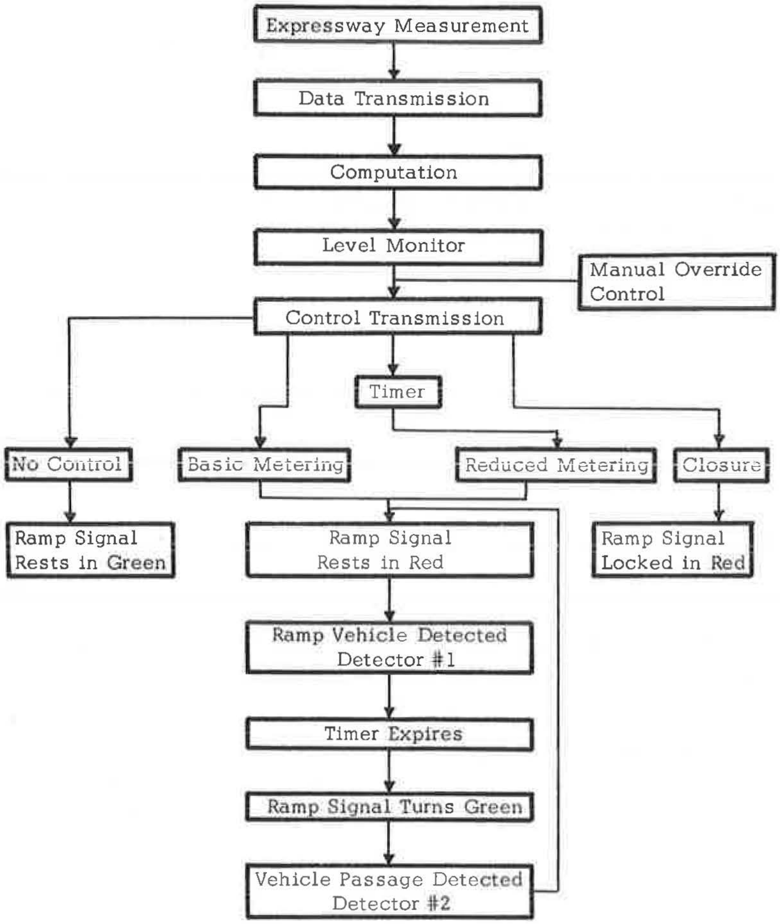

Figure 3. Schematic diagram of ramp metering operation.

TABLE 1

Mainline Occupancy (%)	Metering Rate (vpm)
15	12
16	12
17	12
18	8.5
19	8.5
20	8.5
21	7.5
22	6.5
23	5.0
24	5.0

TABLE 2

Center Lane Occupancy (%)	Metering Rate (vpm)
15 - 18	9.5
18 - 21	7.2
21 - 23	5.3
> 23	3.9

TABLE 3

Center Lane Occupancy (%)	Total Directional Volume (vpm)	Metering Rate (vpm)
≤ 22	< 86	13
	86 - 91	9.5
	91 - 93	7.2
	93 - 95	5.3
	> - 95	3.9
> 22	All volumes	3.9

scheme based on the occupancy levels of the queue detector (detector at the top of the ramp) and the merge detector (detector at the bottom of the ramp).

FUTURE PLANS FOR EXTENDING RAMP METERING

Plans for extending ramp metering to other adjacent ramps were presented to the project's advisory committee on April 29, and approval was given to the staff to proceed. In connection with the ramp metering, additional expressway surveillance detectors will be installed downstream of the present study section, and four changeable message display signs will be installed on surface streets leading to the expressway to inform motorists of conditions at nearby ramps and portions of the expressway.

Figure 4 shows the location of ramp metering, additional expressway detector stations, and surface street displays. These installations will be made during the summer and should be in operation by late summer or early fall. These plans call for extending the ramp metering to other ramps that are presently heavily congested during the afternoon peak traffic period. The present scheme for metering, although best of those tested, is not necessarily the best scheme for maximizing total expressway-ramp flows. Therefore, several different approaches are being considered, as well as further possible modifications of the present scheme.

The approach discussed in this paper is the combination of the gap availability and the gap acceptability distributions for various volume and occupancy levels to estimate the number of available acceptable gaps for different volume and occupancy levels. This type of analysis actually permits two approaches. First, the relationship between volume or occupancy and metering rate for each of the different ramps can be established before actual ramp metering, and basically, the approach now being used can be refined. A second approach would be to meter individual ramp vehicles on the basis of measuring available acceptable gaps approaching the merge area. The ramp could be thought of as a launching pad with the release of the individual ramp vehicle at the proper time to "hit" an available gap. The analysis proposed permits some insight into the feasibility of such an approach.

PREVIOUS WORK IN GAP AVAILABILITY AND GAP ACCEPTANCE

Gap Availability Studies

One of the first references containing measurements of time headways or gap availability was the Highway Capacity Manual (4). The objective of the time headway measurements was to determine a sensitive index of traffic congestion. It was concluded that a lane of a two-lane highway is congested when 72 percent of the drivers have time headways of less than 9 sec.

Greenshields (5) presented a time headway distribution of vehicles traveling on a two-lane highway in Maryland. A total of 660 measured time headways was obtained when the volume rate was 550 vph. Moskowitz (6) measured time headways on a four-lane rural highway near Sacramento, Calif. More than 8,600 individual headways were recorded during a 6-hr period from 4:00 to 10:00 PM. The hourly volume rates (combining the four lanes) ranged from 650 to 2,400 vph, or average lane volume rate of 160 to 600 vph. Gerlough (7) measured 214 individual time headways in Lane 1 of the Arroyo Seco (Pasadena) Freeway in October 1960. A 30-min period was observed during which the volume rate was 440 vph.

Photographic techniques were employed on the Lodge Expressway to obtain approximately 64,000 individual time and distance headways (8). Results included modal time spacing vs three-lane directional hourly volume (1,000 to 6,000 vph), frequency of vehicles with time headways less than those considered safe (0.7 sec), and cumulative percentage of volume as a function of time spacing.

In connection with a ramp simulation study, approximately 20,000 time headways were measured at four locations in Chicago on the Congress and Edens Expressways (9). Curves for the probability of a time gap of a given length or longer for various hourly traffic volumes were developed from the collected data.

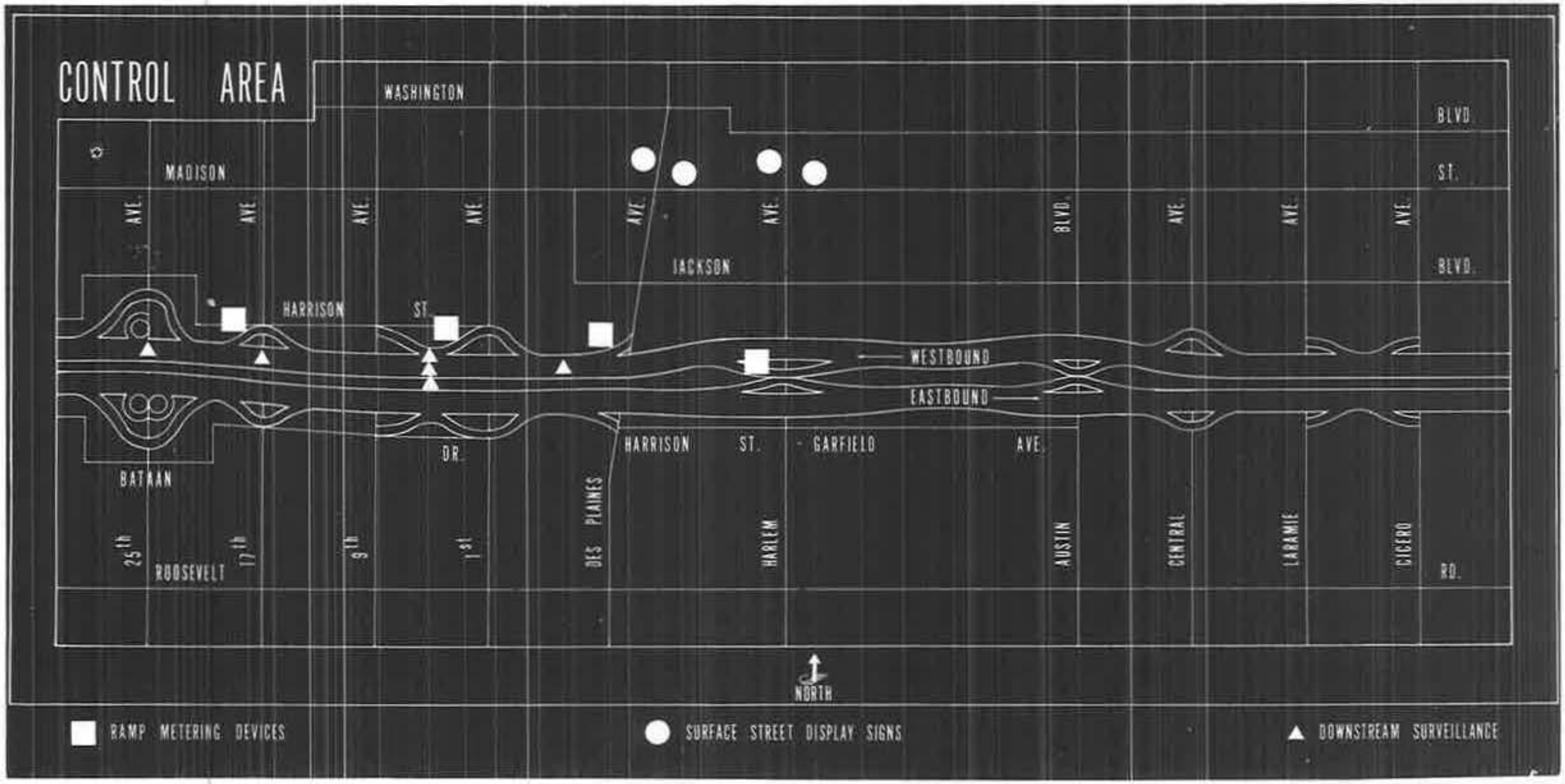

Figure 4. Locations for proposed extension of ramp metering and accompanying detector stations and surface street displays.

May (10) reported the measurement of individual time headways obtained for a 24-hr period on seven types of multilane facilities in Michigan. Data were analyzed on a per lane basis, and a discussion of central tendencies, frequency distributions, and percentiles related to various central tendencies is included. Lane volume rates varied from 100 to 2,400 vph/lane.

In connection with a study of the suitability of left-side on- and off-ramps, individual time headways were obtained at the Harlem Avenue left-hand entrance ramp and the First Avenue right-hand on-ramp on the Eisenhower Expressway in Chicago (11). The frequency of various sized time headways was calculated for four volume levels, and the percent of total time expanded in time headways of various sizes is included.

Basmaciyan (12) conducted studies of traffic flow on two-lane rural highways in North Carolina. Data were collected during times when lane density varied from 1 to 10 veh/mi to permit analysis of time headway distributions for various density levels. Regression equations for percentage of vehicles traveling closer than a specified time headway for various average lane densities were developed.

Fitting Known Mathematical Distributions

Much work has been undertaken in fitting known mathematical distributions to measured time headway distributions. Only a portion of the references are mentioned in this paper. For more extensive coverage, theory of traffic flow bibliographies (13, 14) are available, and Haight (15) gives a complete essay on mathematical distributions. For general reference in regard to mathematical distributions, a number of excellent mathematical and statistical books are available (16, 17).

The Poisson distribution has been widely used for time headway or gap data. Some of the earliest work in this direction was reported by Kinzer (18), Rader (19), Adams (20), and Garwood (21) before World War II. These papers proposed the use of the Poisson distribution; however, in most cases it was being applied to low-volume facilities. Adams surmised that the Poisson distribution might be applicable for traffic volumes as high as 1,000 vph; however, his data were for volumes up to 500 vph.

Immediately after World War II, considerable effort was made by a group of researchers at the Bureau of Highway Traffic at Yale University. Greenshields (22) compared the theoretical distribution resulting from Poisson to measured time headways obtained by Normann (23). The two cumulative distributions were compared in tabular form and visually appear to be similar except for 1-sec spacings. Comparisons were made for volumes between 100 and 1,800 vph at increments of 100 vph. Ricker (24) introduced a symposium on studies of weaving and merging traffic. In the preface, he reviews the earlier work and cautions as to the application of the Poisson distribution to time headway distributions when the single-lane volume exceeds 200 vph, the two-lane volume exceeds 400 vph, and the multilane exceeds 1,000 vph. Raff (25, 26) pursued the study of gap distributions for two intersecting traffic streams with the objective of developing volume warrants for urban stop signs. Raff introduced an interesting new concept of blocks and anti-blocks; blocks are defined as the intervals of time when a vehicle cannot cross an intersecting stream, and anti-blocks as the intervals of time when a vehicle can cross an intersecting stream.

Following the mathematical work of Garwood (21) and Tanner (27), Moskowitz (6) proceeded to test these mathematical statistical theories by measuring time headways on a four-lane freeway near Sacramento, Calif. The gap distribution was divided into five volume groups: 650, 850, 1,250, 1,600, and 2,400 vph (total for all four lanes). A series of graphs were prepared for computing the probability of waiting any length of time for any gaps in a stream of any average (i.e., hourly) rate of flow. It is shown that the California observations are in amazing agreement with the theories developed in England during the decade before. In concluding, Moskowitz said, "The unfortunate distance (geographically speaking) between English brains and California traffic is remarked once again."

The Eno Foundation published two papers in 1955 on Poisson and traffic (7). The first paper by Gerlough (28) describes the nature of the Poisson distribution and gives examples of its application to various traffic problems. Observed frequency of gaps in a single lane was obtained when the lane volume rate was 440 vph. A graphical comparison is made between the observed and theoretical frequencies, but no tests were made to determine the "goodness of fit." Multiple Poisson distributions are briefly discussed, particularly in regard to crossing streams of flow. Schuhl (29) proposed the use of a modified Poisson distribution for describing time headway distributions. Specifically, he suggested a multiple Poisson distribution, one for constrained vehicles and the other for free-flowing vehicles. Schuhl further suggested that the lower time interval should not be zero but some value representative of the minimum time headway. The formula proposed was

$$P(\theta) = N\tau (t_1 - \epsilon)\, e^{-\frac{\theta - \epsilon}{t_1 - \epsilon}} + N(I - \tau)\, t_2 e^{-\frac{\theta}{t_2}} \tag{1}$$

where

$P(\theta)$ = probability of no vehicles in time interval θ,
N = average number of vehicles in unit time,
τ = proportion of vehicles in constrained subset,
$I - \tau$ = proportion of vehicles in free-flowing subset,
t_2 = mean time spacing of free-flowing vehicles,
t_1 = mean time spacing of constrained vehicles,
ϵ = minimum time headway, and
e = Napierian base of logarithms.

The author proceeded to experiment with the selection of values for these variables to compare his measured time headways most favorably with the multiple Poisson distributions. The values for the variables finally selected were $\tau = 0.583$, $I - \tau = 0.417$, $t_1 = 1.98$, $t_2 = 13.16$, and $\epsilon = 0.81$.

The late 1950's ushered in the use of the digital computer for traffic flow simulation. One of the first problems that arises in computer simulation is the generation of traffic inputs. Empirical data, if available, can be used. However, in most computers the input time is many times slower than the computation time; therefore, mathematical distributions, if they are realistic, can be generated by the computer itself and usually save valuable time. Gerlough (30) devotes a complete paper to this subject, whereas others such as Levy, Kell, and Lewis discuss traffic inputs as one phase of simulation. In addition to discussing methods for computer generation of traffic inputs, Gerlough expands his early work (28) and the work of Schuhl (29) by pointing out the need for shifted composite Poisson distributions rather than regular Poisson distributions when vehicles are flowing in platoons or are constrained. Perchonock and Levy (9, 31), in their ramp simulation work, compared time headway distributions obtained at four expressway locations in Chicago with the Poisson distribution. When the χ^2 goodness of fit test was applied to the two distributions, there was an extremely close agreement, and the probability of gaps not being distributed exponentially was determined to be four chances in one million.

Kell (32, 33) pursued the shifted composite Poisson distribution as input format for his intersection simulation work. The equation he proposed is

$$p(h \geq t) = (1 - a)\, e^{-\frac{t - \lambda}{T_1 - \lambda}} + a\, e^{-\frac{t - \tau}{T_2 - \tau}} \tag{2}$$

where

$p(h \geq t)$ = probability of a headway (h) greater than or equal to the time (t),
a = proportion of the traffic stream in the restrained group,
$(1 - a)$ = proportion of the traffic stream in the free-moving group,

T_1 = average headway of the free-moving vehicles,
T_2 = average headway of the restrained vehicles,
λ = minimum headway of the free-moving vehicles,
τ = minimum headway of the restrained vehicles, and
e = natural or Napierian base of logarithms.

Kell then proceeded to modify his initial equation to facilitate solving for the five parameters, a, T_1, T_2, λ, and τ, which are functions of the traffic volume. Field data of individual time headways were obtained and numerous attempts were made to obtain values for these parameters which would result in the best χ^2 test.

Lewis (34, 35) proposed the use of a modified binomial distribution in which two different levels of probability are employed. The final equations developed were

$$P_a = 1 - (1 - B)^{\frac{1}{r - e + 1}} \tag{3a}$$

and

$$P_b = \frac{1 - B}{\bar{h} - e - \sum_{n=1}^{r-e} \left(1 - p_a\right)^n} \tag{3b}$$

where

P_a = enhanced probability of an arrival at a time increment when $e \leq h \leq r$,
P_b = diminished probability of an arrival at a time increment when $h > r$,

TABLE 4

SUMMARY OF APPLICATIONS OF MATHEMATICAL DISTRIBUTIONS TO TIME HEADWAY DISTRIBUTIONS

Year	Poisson Distribution: Regular	Poisson Distribution: Multiple (cross-stream)	Poisson Distribution: Shifted	Poisson Distribution: Multiple (single-stream)	Poisson Distribution: Generalized	Modified Bionomial Distrib.
1933	Kinzer (18)					
1934	Rader (19)					
1935						
1936	Adams (20)					
1937						
1938						
1939						
1940	Garwood (21)	Garwood (21)				
1941						
1942						
1943						
1944						
1945						
1946						
1947	Greenshields (22)	Greenshields (22)				
1948	Ricker (24)					
1949						
1950	Raff (25)	Raff (25)				
1951	Raff (26) Tanner (27)	Raff (26) Tanner (27)				
1952	Greenshields (5)					
1953						
1954	Moskowitz (6)					
1955	Gerlough (28) Schuhl (29)	Gerlough (28)	Schuhl (29)	Schuhl (29)		
1956						
1957						
1958						
1959			Gerlough (30)	Gerlough (30)	Haight (36)	
1960	Perchonock (9) Haight (37)		Kell (32) Haight (37)	Kell (32) Haight (37)		
1961	Levy (31) Pearson (39)				Haight (38)	
1962			Kell (33)	Kell (33) Oliver (41)		
1963						Lewis (34, 35)

B = bunching factor which is the fraction of all headways $\le$ r,
r = maximum headway for which the probability of an arrival is enhanced,
e = minimum headway permitted,
$\bar{h}$ = mean of all headways, and
n = any value of h.

Lewis proposed $1\frac{1}{2}$ sec for e, $4\frac{1}{2}$ sec for r, and $1 - e^{-0.00132V}$ for B. Comparisons are made between the modified binomial distribution and several sets of measured headways, and were used in his intersection simulation work.

Haight (36, 37, 38) proposed the use of a generalized Poisson distribution because "the customary assumption of random distribution of arrival times of cars has been found to be slightly unsatisfactory for two reasons: (a) very small gaps do not occur as frequently as that theory would require, and (b) there is some interference between vehicles, inducing an element of regularity into arrival times." The generalized Poisson distribution equation includes k and λ parameters which can be adjusted. When $k = 1$ the Poisson distribution results, whereas as $k \to \infty$ the result is an equally spaced (or regular) distribution. The intermediate distribution of vehicles (gap distributions which lie between random and equally spaced distributions) can be described by the generalized Poisson distribution equation corresponding to a Pearson Type III distribution. One interesting result when fitting measured time headway distributions to the generalized Poisson distribution is that the resulting k value is a quantitative measure of the nonrandomness of the traffic flow.

Other recent works involved with or closely related to the fitting of known mathematical distributions to measured time headway distributions are available (8, 39, 40, 41, 42, 43). A summary of references which include the application of mathematical distributions to time headway distributions is given in Table 4. This summary does not include all the available references, but an attempt has been made to be as complete as possible.

Gap Acceptance Studies

A rather thorough review of references has revealed only a limited number of gap acceptance studies which have been conducted in the United States. However, these types of studies are now being carried out at General Motors, Northwestern University and possibly at Carnegie Institute of Technology.

Gourlay (44) reported on a gap acceptance study of two locations in New York City. At one location, the ramp was on the right side and controlled by a stop sign. At the second location, the ramp was on the left side and was uncontrolled. Data were obtained by ground photography; 54 observations were made at the stop sign location, and 47 were made at the uncontrolled location. The major conclusion was that the majority of drivers will accept a 6-sec gap, if stopped, and a 3-sec gap if they are allowed to merge without stopping.

Strickland (45) reported on a study conducted at two locations at a rotary intersection in Hartford, Conn. In both cases, the lighter movement approached on the left and is more similar to left-side on-ramp conditions. There was no control at either location, although at times because of heavier mainline traffic, the traffic approaching on the left had to stop before entering to obtain an acceptable gap. The data were obtained using time-lapse photographic techniques; 205 observations were obtained at one location, and 139 were obtained at the second location. The results of this study were similar to Gourlay's results, except that there was some indication that a 2-sec headway was satisfactory when good merging conditions existed.

A gap availability study is reported in the Highway Capacity Manual (4), but unfortunately, the location is not identified. The results were obtained for one location which was a two-lane stop-controlled right-side on-ramp with no acceleration lane and operating with long queues. An interesting aspect of this study was that acceptable gap data for passenger cars and trucks were kept separately. A majority of the passenger vehicles accepted a 5-sec gap, whereas trucks required a 6-sec gap. In addition to determining the percent of vehicles accepting various sized gaps, the number of vehicles (0, 1, 2, etc.) sharing an individual gap was recorded, resulting in a tabulation of the average number of vehicles accepting various gap sizes.

TABLE 5
PERCENT OF VEHICLES ACCEPTING INDICATED GAP SIZE

Gap Size	Stop Sign Locations or Starting from Stopped Position						Uncontrolled Free Merge[a]		
	Gourlay	Strickland	Capacity Manual		Texas A and M	Pearson	Gourlay	Strickland	Texas A and M
			Pass. Car	Truck					
0-0.9	-	-	-	-	0	0	-	-	12
0-1.4	0	0	0	0	-	-	0	0	-
1.0-1.9	-	-	-	-	3	0	-	-	57
1.5-2.4	0	0	12	0	-	-	50	85	-
2.0-2.9	-	-	-	-	17	0	-	-	77
2.5-3.4	11	5	20	9	-	-	84	94	-
3.0-3.9	-	-	-	-	36	9	-	-	95
3.5-4.4	50	27	42	18	-	-	100	100	-
4.0-4.9	-	-	-	-	65	29	-	-	97
4.5-5.4	56	25	54	35	-	-	100	100	-
5.0-5.9	-	-	-	-	95	51	-	-	98
5.5-6.4	64	67	65	55	-	-	100	100	-
6.0-6.9	-	-	-	-	100	63	-	-	100
6.5-7.4	71	100	69	63	-	-	100	100	-
7.0-7.9	-	-	-	-	100	72	-	-	100
7.5-8.4	100	100	73	71	-	-	100	100	-
8.0-8.9	-	-	-	-	100	77	-	-	98
8.5-9.4	100	100	77	75	-	-	100	100	-
9.0-9.9	-	-	-	-	100	81	-	-	100
9.5-10.4	100	100	80	80	-	-	100	100	-

[a]No data available for Capacity Manual and Pearson studies.

In connection with a ramp simulation study (31), a gap acceptance study was conducted by Texas A&M, presumably on the Gulf Freeway in Houston, Texas. Gap acceptance data for passenger vehicles only were obtained and a total of 1,373 observations were made. Multiple vehicle entries were not considered and the data were subdivided into merging behavior of stopped vehicles and of moving vehicles. No details are given as to the geometrics of the ramp, but it is assumed to be a right-side on-ramp with an acceleration lane. A majority of the passenger vehicles which merged without stopping required 2 sec, whereas a majority of the passenger vehicles which merged after stopping required a 5-sec gap.

Pearson (39) conducted gap acceptance studies at four ramp locations on the Schuylkill Expressway in Philadelphia, Pa. All four ramps were right-side ramps controlled by stop signs and had acceleration lanes less than 150 ft in length. The primary purpose of the study was to estimate ramp capacities under present stop-controlled conditions and under conditions of no stop-control with adequate acceleration lane length.

The former was based on the gap acceptance studies, whereas the latter was based on observed maximum ramp volumes. Over 1,100 observations were made at the four ramp locations and the majority of the ramp vehicles accepted gaps of 6 sec. In addition to determining the percent of vehicles accepting various sized gaps, the number of vehicles (0, 1, 2, etc.) sharing an individual gap was recorded, resulting in a tabulation of the average number of vehicles accepting various gap sizes. For example, it was determined that for a 10-sec gap, 20 percent of the time no vehicles would accept it, 24 percent of the time one vehicle would accept it, 22 percent of the time two vehicles would accept it, and 34 percent of the time three vehicles would accept it. Summaries of the gap acceptance results from the five studies are given in Tables 5 and 6.

DATA COLLECTION AND DESCRIPTION OF STUDY SITES

The Expressway Surveillance Project's Pilot Detection System was employed to record on punch tape the arrival time of each vehicle (nearest 0.1 sec), and the running average occupancy level in the expressway lane adjacent to, and just upstream of, three of the four ramps to be metered in the near future. Data were collected at each of the three locations for the four afternoon peak periods (2:30 to 6:30 PM) on three weekdays

TABLE 6

AVERAGE NUMBER OF VEHICLES ACCEPTING INDICATED GAP SIZE[a]

Gap Size	Capacity Manual	Pearson
2	0.1	0.0
4	0.3	0.1
6	0.6	0.6
8	1.0	1.2
10	1.8	1.7
12	2.0	2.3
14	2.3	2.7
16	2.8	3.4
18	3.1	4.0
20	3.8	4.5

[a]No data available for Gourlay, Strickland or Texas A&M studies.

Figure 5. Harlem ramp—gap availability study site.

which were free of adverse weather and unusual traffic events. The punch tapes were processed on a digital computer and time headway frequencies were tabulated by minute volume groups (10 to 30 vpm) and by running average occupancy levels (10 to 30 percent occupancy).

The Eisenhower Expressway extends westward from the Chicago loop; at approximately 7 mi west of the loop the outbound roadway is reduced from four to three lanes. Continuing westward, the depressed expressway's first interchange (containing the first westbound ramp to be metered) is at Harlem Avenue and is an inside diamond interchange (off-ramp and on-ramp on left side). The gap availability study was conducted in the median lane at a point 650 ft in advance of the Harlem ramp which has a 1,000-ft acceleration lane (Fig. 5). Congestion normally occurs at this location for about 1 hr each afternoon; however, it is caused by congestion first occurring farther downstream.

The next interchange is approximately 0.6 mi further west and is at Des Plaines Avenue, the second ramp which will be metered. There is only a westbound right-side on-ramp at this point (no off-ramp), and there is a 1,000-ft 3 percent upgrade in advance of the on-ramp. The gap availability study was conducted in the shoulder lane at a point of 700 ft in advance of the Des Plaines ramp which has an 800-ft acceleration lane (Fig. 6). Congestion normally occurs for 1½ hr each weekday afternoon due to the combination effects of the upgrade and the Des Plaines on-ramp.

One mile further west is the First Avenue interchange, an outside diamond interchange. The right-side on-ramp at First Avenue has only a tapered 250-ft acceleration lane. The traffic at this ramp has been metered each weekday afternoon since September 1963 and will be the third ramp in the sequence for the extended metering program. The gap availability study was conducted in the shoulder lane at a point 750 ft in advance of the First Avenue on-ramp (Fig. 7). Congestion does not normally occur at this location, but the flow on the expressway and ramp is quite heavy (400 vph merging with 5,200 vph).

The fourth ramp in the group which will be metered is at the Seventeenth Avenue interchange. However, at the time of the gap availability study, detector equipment had not been installed at this location.

Figure 6. Des Plaines ramp—gap availability study site.

Figure 7. First Avenue ramp—gap availability study site.

STUDY RESULTS

Harlem Avenue

The results of the gap availability study at Harlem as related to various volume levels are given in Table 7. More than 21,000 individual time headways were recorded and grouped first into ½-sec time intervals from 0 to 10 sec, and then into average minute volumes of 10 to 30 vpm (hourly rates of 600 to 1,800 vehicles). An attempt is made here (Fig. 8) to present the individual and cumulative frequencies graphically. The vertical scale is time headway ranging from 0 to 10 sec, whereas the horizontal scale is minute volume ranging from 10 to 30 vpm. The heavy curved line on the graph indicates average time headway at the various minute volume levels. The individual frequency of time headways for the four volume groups is shown as the shaded areas. The horizontal measurements of the shaded areas indicate the percent of vehicles having those particular time headways.

The results of the gap availability study at Harlem Avenue as related to various occupancy levels are given in Table 8. The graphical presentation of individual and cumulative frequencies as related to various occupancy levels is shown in Figure 9.

TABLE 7
TIME HEADWAY DISTRIBUTIONS RELATED TO TRAFFIC VOLUME—HARLEM AVENUE

Gap Size	Average Minute Volume[a] (%)							
	10-14		15-19		20-24		25-30	
	F	C	F	C	F	C	F	C
0.0 - 0.4	6.1	6.1	1.2	1.2	1.5	1.5	1.7	1.7
0.5 - 0.9	6.6	12.7	6.4	7.6	7.7	9.2	10.6	12.3
1.0 - 1.4	6.8	19.5	11.4	19.0	17.2	26.4	23.6	35.9
1.5 - 1.9	7.7	27.3	15.9	34.9	19.4	45.8	22.3	58.2
2.0 - 2.4	6.6	33.8	15.7	50.6	16.4	62.2	15.8	74.0
2.5 - 2.9	7.7	41.5	13.0	63.6	11.2	73.4	9.5	83.5
3.0 - 3.4	6.3	47.8	8.8	72.4	8.1	81.5	5.9	89.4
3.5 - 3.9	6.1	53.9	6.5	78.9	5.5	86.8	3.5	92.9
4.0 - 4.4	4.9	58.8	4.3	83.2	3.7	90.5	2.3	95.2
4.5 - 4.9	2.7	61.5	3.3	86.5	2.5	93.0	1.6	96.8
5.0 - 5.4	3.0	64.5	2.2	88.7	1.4	94.6	1.0	97.8
5.5 - 5.9	3.1	67.6	1.9	90.6	1.2	95.8	0.5	98.3
6.0 - 6.4	2.5	70.1	1.4	92.0	0.9	96.7	0.5	98.8
6.5 - 6.9	2.4	72.5	1.0	93.0	0.7	97.4	0.5	99.3
7.0 - 7.4	2.0	74.5	1.2	94.2	0.6	98.0	0.2	99.5
7.5 - 7.9	1.5	76.0	0.8	95.0	0.4	98.4	0.2	99.7
8.0 - 8.4	1.7	77.7	0.5	95.5	0.3	98.7	0.0	99.7
8.5 - 8.9	1.8	79.5	0.7	96.2	0.3	99.0	0.1	99.8
9.0 - 9.4	1.3	80.8	0.5	96.7	0.2	99.2	0.1	99.9
> 9.5	9.2	100.0	3.3	100.0	0.8	100.0	0.1	100.0
Mode	1.8		1.8		1.8		1.2	
Median	3.6		2.4		2.0		1.7	
Mean	5.0		3.5		2.7		2.2	

[a] F = frequency, C = cumulative frequency.

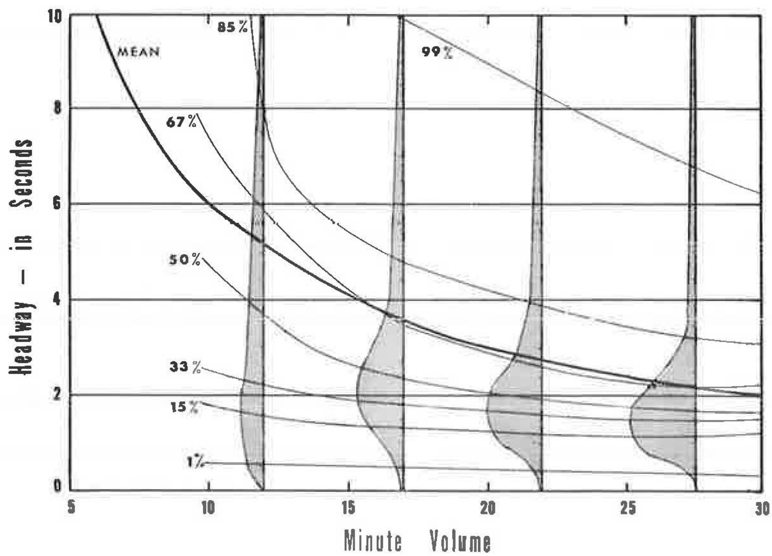

Figure 8. Time headway distributions related to traffic volume—Harlem Avenue.

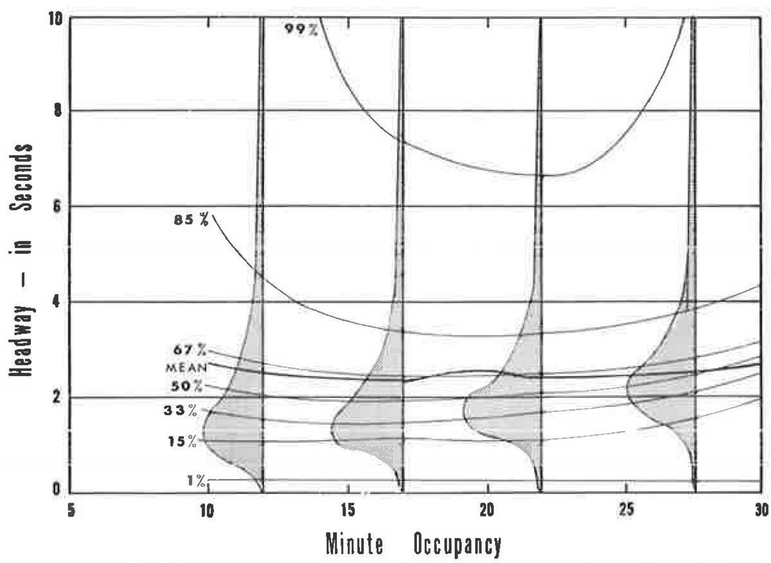

Figure 9. Time headway distributions related to occupancy levels—Harlem Avenue.

TABLE 8

TABULATION OF TIME HEADWAY DISTRIBUTIONS RELATED TO OCCUPANCY LEVELS—HARLEM AVENUE

Gap Size	Running Average Percent Occupancy[a]							
	10-14		15-19		20-24		25-30	
	F	C	F	C	F	C	F	C
0.0 - 0.4	2.0	2.0	2.0	2.0	1.5	1.5	1.8	1.8
0.5 - 0.9	11.0	13.0	7.4	9.4	3.1	4.6	1.9	3.7
1.0 - 1.4	20.6	33.6	24.3	33.7	16.9	21.5	7.2	10.9
1.5 - 1.9	17.0	50.6	21.3	55.0	27.1	48.6	20.0	30.9
2.0 - 2.4	12.5	63.1	15.0	70.0	19.9	68.5	23.1	54.0
2.5 - 2.9	8.2	71.3	9.9	79.9	12.8	81.3	16.0	70.0
3.0 - 3.4	6.0	77.3	6.4	86.3	6.7	88.0	10.5	80.5
3.5 - 3.9	4.7	82.0	4.4	90.7	3.9	91.9	6.3	86.8
4.0 - 4.4	3.7	85.7	2.7	93.4	2.6	94.5	3.4	90.2
4.5 - 4.9	2.7	88.4	1.8	95.2	1.7	96.2	2.2	92.4
5.0 - 5.4	1.8	90.2	1.4	96.6	1.3	97.5	1.4	93.8
5.5 - 5.9	1.6	91.8	0.5	97.1	0.8	98.3	1.0	94.8
6.0 - 6.4	1.4	93.2	0.8	97.9	0.1	98.4	1.4	96.2
6.5 - 6.9	1.0	94.2	0.5	98.4	0.4	98.8	0.5	96.7
7.0 - 7.4	1.0	95.2	0.3	98.7	0.1	98.9	0.5	97.2
7.5 - 7.9	0.6	95.8	0.3	99.0	0.3	99.2	0.2	97.4
8.0 - 8.4	0.6	96.4	0.2	99.2	0.1	99.3	0.2	97.6
8.5 - 8.9	0.5	96.9	0.1	99.3	0.1	99.4	0.4	98.0
9.0 - 9.4	0.4	97.3	0.2	99.5	0.1	99.5	0.2	98.2
9.5	2.7	100.0	0.5	100.0	0.5	100.0	1.8	100.0
Mode	1.2		1.2		1.8		2.2	
Median	2.0		1.9		2.0		2.4	
Mean	2.5		2.3		2.3		2.4	

[a]F = frequency, C = cumulative frequency.

TABLE 9

TIME HEADWAY DISTRIBUTIONS RELATED TO TRAFFIC VOLUME—DES PLAINES AVENUE

Gap Size	Average Minute Volumes[a] (%)							
	10-14		15-19		20-24		25-30	
	F	C	F	C	F	C	F	C
0.0 - 0.4	5.0	5.0	1.3	1.3	1.4	1.4	1.8	1.8
0.5 - 0.9	4.8	9.8	6.4	7.7	6.2	7.6	7.3	9.1
1.0 - 1.4	9.1	18.9	12.9	20.6	14.7	22.3	22.8	31.9
1.5 - 1.9	9.7	28.6	14.6	35.2	20.6	42.9	24.2	56.1
2.0 - 2.4	7.5	36.1	15.7	50.9	18.0	60.9	16.3	72.4
2.5 - 2.9	7.5	43.6	10.5	61.4	12.5	73.4	10.0	82.4
3.0 - 3.4	6.7	50.3	7.9	69.3	8.5	81.9	6.7	89.1
3.5 - 3.9	5.0	55.3	6.6	75.9	5.4	87.3	4.1	93.2
4.0 - 4.4	4.9	60.2	4.9	80.8	3.4	90.7	2.4	95.6
4.5 - 4.9	4.0	64.2	3.4	84.2	2.4	93.1	1.6	97.2
5.0 - 5.4	4.1	68.3	2.9	87.1	1.7	94.8	0.9	98.1
5.5 - 5.9	3.7	72.0	2.5	89.6	1.3	96.1	0.6	98.7
6.0 - 6.4	3.5	75.5	1.9	91.5	1.0	97.1	0.3	99.0
6.5 - 6.9	2.7	78.2	1.3	92.8	0.9	98.0	0.2	99.2
7.0 - 7.4	1.3	79.5	1.3	94.1	0.5	98.5	0.3	99.5
7.5 - 7.9	2.2	81.7	1.1	95.2	0.3	98.8	0.1	99.6
8.0 - 8.4	2.1	83.8	0.9	96.1	0.2	99.0	0.1	99.7
8.5 - 8.9	1.4	85.2	0.8	96.9	0.3	99.3	0.2	99.9
9.0 - 9.5	0.9	86.1	0.5	97.4	0.2	99.5	0.0	99.9
> 9.5	13.9	100.0	2.6	100.0	0.5	100.0	0.1	100.0
Mode	1.8		2.2		1.8		1.8	
Median	3.5		2.5		2.2		1.9	
Mean	5.0		3.5		2.7		2.2	

[a]F = frequency, C = cumulative frequency.

Des Plaines Avenue

The Des Plaines Avenue data are presented in the same manner as the Harlem Avenue data. There were over 14,500 measured individual time headways. The individual and cumulative frequencies as related to various volume levels are given in Table 9. These same frequencies are presented in graphical format in Figure 10. The individual and cumulative frequencies as related to various occupancy levels are given in Table 10. These same frequencies are presented in graphical format in Figure 11.

First Avenue

The First Avenue data are presented in the same manner as the Harlem Avenue and Des Plaines Avenue data. There were over 13,700 measured individual time head-

TABLE 10

TIME HEADWAY DISTRIBUTIONS RELATED TO OCCUPANCY LEVELS--DES PLAINES AVENUE

Gap Size	Running Average Percent Occupancy[a]							
	10-14		15-19		20-24		25-30	
	F	C	F	C	F	C	F	C
0.0 - 0.4	1.5	1.5	1.7	1.7	2.1	2.1	1.7	1.7
0.5 - 0.9	8.9	10.4	9.0	10.7	5.4	7.5	2.8	4.5
1.0 - 1.4	16.5	26.9	20.2	30.9	17.9	25.4	10.7	15.2
1.5 - 1.9	14.2	41.1	19.1	50.0	22.6	48.0	21.7	36.9
2.0 - 2.4	10.9	52.0	13.9	63.9	17.7	65.7	21.1	58.0
2.5 - 2.9	8.0	60.0	8.8	72.7	10.3	76.0	15.0	73.0
3.0 - 3.4	6.0	66.0	6.6	79.3	7.8	83.8	10.1	83.1
3.5 - 3.9	5.3	71.3	5.2	84.5	5.3	89.1	5.6	88.7
4.0 - 4.4	4.5	75.8	3.3	87.8	3.3	92.4	3.5	92.2
4.5 - 4.9	3.3	79.1	2.5	90.3	2.4	94.8	2.2	94.4
5.0 - 5.4	2.7	81.9	1.8	92.1	1.5	96.3	1.6	96.0
5.5 - 5.9	2.7	84.6	1.8	93.9	1.0	97.3	1.1	97.1
6.0 - 6.4	2.2	86.8	1.2	95.1	0.5	97.8	0.8	97.9
6.5 - 6.9	1.8	88.6	1.0	96.1	0.6	98.4	0.5	98.4
7.0 - 7.4	1.4	90.0	0.7	96.8	0.3	98.7	0.3	98.7
7.5 - 7.9	1.4	91.4	0.5	97.3	0.2	98.9	0.3	99.0
8.0 - 8.4	1.2	92.6	0.6	97.9	0.2	99.1	0.1	99.1
8.5 - 8.9	1.2	93.8	0.6	98.5	0.1	99.2	0.1	99.2
9.0 - 9.4	0.8	94.4	0.2	98.7	0.1	99.3	0.1	99.3
9.5	5.6	100.0	1.3	100.0	0.7	100.0	0.7	100.0
Mode	1.2		1.2		1.8		1.8	
Median	2.4		2.0		2.1		2.3	
Mean	3.3		2.5		2.4		2.4	

[a]F = frequency, C = cumulative frequency.

TABLE 11

TIME HEADWAY DISTRIBUTIONS RELATED TO TRAFFIC VOLUME—FIRST AVENUE

Gap Size	Average Minute Volumes[a] (%)							
	10-14		15-19		20-24		25-30	
	F	C	F	C	F	C	F	C
0.0 - 0.4	0.3	0.3	0.2	0.2	0.3	0.3	0.4	0.4
0.5 - 0.9	3.8	4.1	5.7	5.9	7.8	8.1	7.9	10.3
1.0 - 1.4	7.3	11.4	12.7	18.6	18.6	26.7	20.8	31.1
1.5 - 1.9	9.8	21.2	14.2	32.8	17.3	44.0	20.9	52.0
2.0 - 2.4	9.1	30.3	12.6	45.4	14.0	58.0	19.2	71.2
2.5 - 2.9	11.9	42.2	10.6	56.0	10.1	68.1	8.3	79.5
3.0 - 3.4	8.3	50.5	8.0	64.0	9.9	76.0	6.7	86.2
3.5 - 3.9	5.9	56.4	6.6	70.6	5.9	81.9	4.3	90.5
4.0 - 4.4	5.1	61.5	5.2	75.8	4.8	86.7	2.7	93.2
4.5 - 4.9	5.5	67.0	3.8	79.6	3.3	90.0	2.1	95.3
5.0 - 5.4	4.5	71.5	3.2	82.8	2.6	92.6	1.3	96.6
5.5 - 5.9	3.3	74.8	3.3	86.1	1.9	94.5	0.9	97.5
6.0 - 6.4	3.5	78.3	1.9	88.0	1.3	95.8	1.0	98.5
6.5 - 6.9	2.3	80.6	2.3	90.3	0.9	96.7	1.0	99.5
7.0 - 7.4	2.5	83.1	1.8	92.1	1.0	97.7	0.2	99.7
7.5 - 7.9	3.3	86.4	2.0	94.1	0.8	98.5	0.1	99.8
8.0 - 8.4	1.6	88.0	1.1	95.2	0.4	98.9	0.1	99.9
8.5 - 8.9	1.4	89.4	1.2	96.4	0.3	99.2	0.0	99.9
9.0 - 9.4	1.6	91.0	0.6	97.0	0.3	99.5	0.0	99.9
> 9.5	9.0	100.0	3.0	100.0	0.5	100.0	0.1	100.0
Mode	2.8		1.8		1.2		1.8	
Median	3.4		2.7		2.2		1.8	
Mean	5.0		3.5		2.7		2.2	

[a]F = frequency, C = cumulative occupancy.

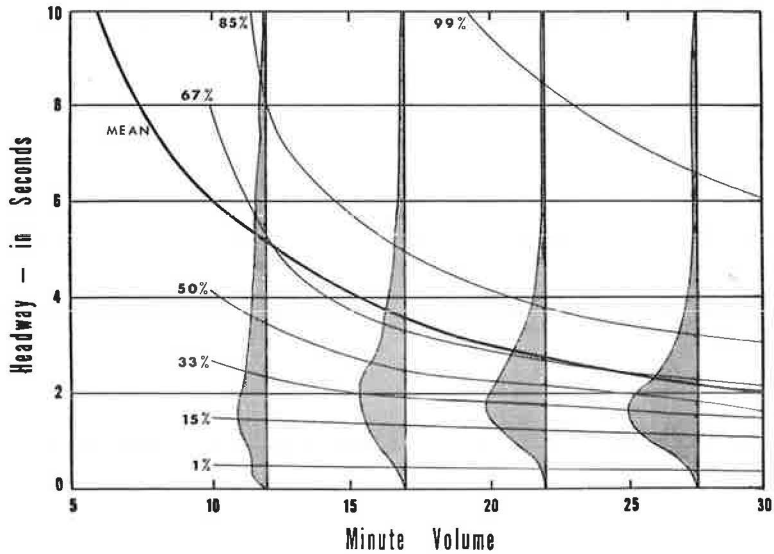

Figure 10. Time headway distributions related to traffic volume—Des Plaines Avenue.

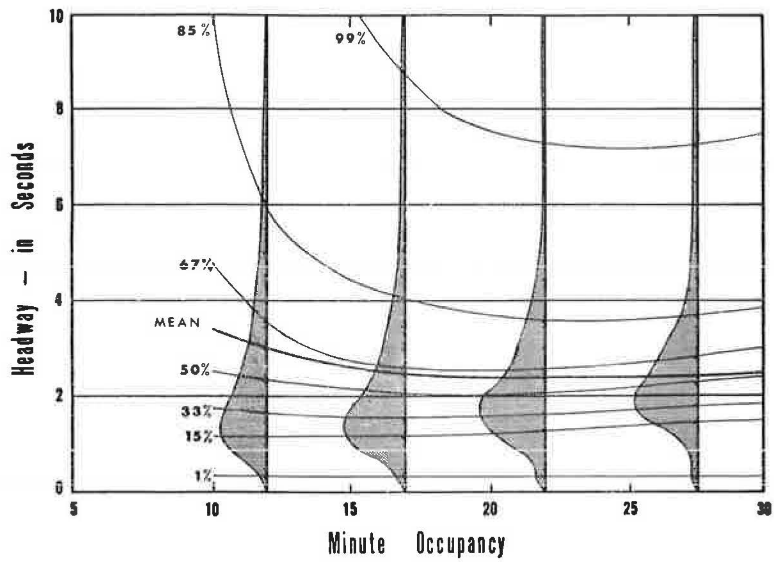

Figure 11. Time headway distributions related to occupancy levels—Des Plaines Avenue.

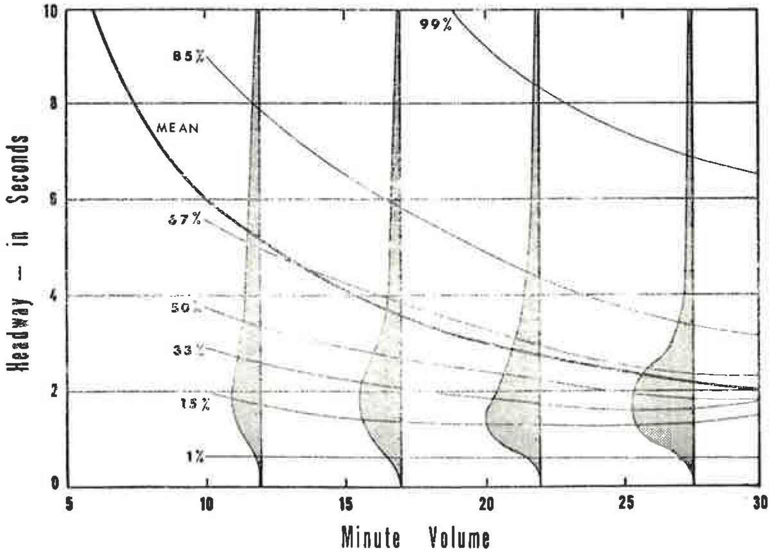

Figure 12. Time headway distributions related to traffic volume—First Avenue.

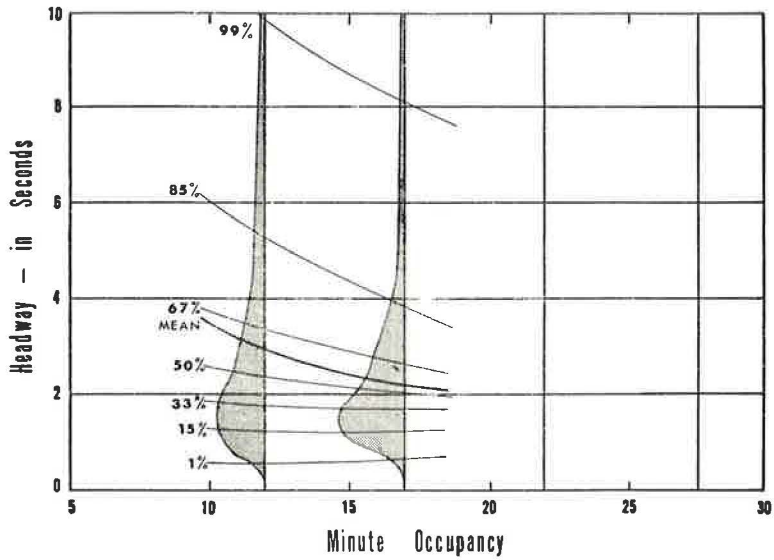

Figure 13. Time headway distributions related to occupancy levels—First Avenue.

ways. The individual and cumulative frequencies as related to various volume levels are given in Table 11 and as related to various occupancy levels in Table 12. These same frequencies are presented in graphical format in Figure 12 and 13, respectively.

ANALYSES OF DATA

Central Tendency Measurements

The mean, median, and mode of each measured time headway distribution were calculated as indications of central tendencies. In all distributions the mode was always

TABLE 12

TIME HEADWAY DISTRIBUTIONS RELATED TO OCCUPANCY LEVELS--FIRST AVENUE[a]

Gap Size	Running Average Percent Occupancy[b]			
	10-14		15-19	
	F	C	F	C
0.0 - 0.4	0.8	0.8	0.4	0.4
0.5 - 0.9	7.7	8.5	7.3	7.7
1.0 - 1.4	15.2	23.7	21.9	29.6
1.5 - 1.9	15.3	39.0	20.6	50.2
2.0 - 2.4	12.7	51.7	14.8	65.0
2.5 - 2.9	9.8	61.5	10.0	75.0
3.0 - 3.4	7.5	69.0	5.5	80.5
3.5 - 3.9	5.8	74.8	6.1	86.6
4.0 - 4.4	4.8	79.6	3.9	90.5
4.5 - 4.9	3.9	83.5	2.7	93.2
5.0 - 5.4	2.6	86.1	1.7	94.9
5.5 - 5.9	2.5	88.6	1.6	96.5
6.0 - 6.4	1.7	90.3	0.5	97.0
6.5 - 6.9	1.7	92.0	1.1	98.1
7.0 - 7.4	1.7	93.7	0.3	98.4
7.5 - 7.9	0.8	94.5	0.7	99.1
8.0 - 8.4	1.7	96.2	0.1	99.2
8.5 - 8.9	0.8	97.0	0.2	99.4
9.0 - 9.4	1.0	98.0	0.1	99.5
>9.5	2.0	100.0	0.5	100.0
Mode	1.2		1.2	
Median	2.5		2.0	
Mean	2.9		2.2	

[a]No data available for 20-24 and 25-30 groups.
[b]F = frequency, C = cumulative frequency.

less than the median, and the median was always less than the mean headway.

The modes and medians for the four volume groups and for the three locations are given in Table 13. Also included are the average modes and average medians for the four volume groups combining the three Chicago locations and similar average modes and average medians for the earlier Detroit study (10).

The average modes of the Chicago data are 0.4 to 0.7 sec greater than the average modes of the Detroit data. As would be expected, the modes decreased with increased volume rates. The average medians of the Chicago data are within + 0.2 sec of those of the Detroit data. There did not seem to be a consistent difference among the average medians of the three Chicago locations. Again, as would be expected, the medians decreased with increased volume rates.

The equation given in the Detroit study for median time headway in terms of the minute volume was

$$\text{median headway} = \frac{43.6}{\text{minute volume}} \tag{4a}$$

A similar equation for the Chicago study was developed:

$$\text{median headway} = \frac{36}{\text{minute volume}} + 0.5 \tag{4b}$$

The equation given in the Detroit study for mode time headway in terms of the minute volume was

$$\text{mode headway} = \frac{17}{\text{minute volume}} + 0.3 \tag{5a}$$

TABLE 13

CENTRAL TENDENCIES—VOLUME GROUPINGS[a]

Location	Modes (sec)				Medians (sec)			
	10-14 Vpm	15-19 Vpm	20-24 Vpm	25-30 Vpm	10-14 Vpm	15-19 Vpm	20-24 Vpm	25-30 Vpm
Harlem	1.8	1.8	1.8	1.2	3.6	2.4	2.0	1.7
Des Plaines	1.8	2.2	1.8	1.8	3.5	2.5	2.2	1.9
First Ave.	2.8	1.8	1.2	1.8	3.4	2.7	2.2	1.8
Chicago avg.	2.1	1.9	1.6	1.6	3.5	2.5	2.1	1.8
Detroit avg.	1.7	1.3	1.1	0.9	3.6	2.6	2.0	1.6

[a]H_t values are as follows: for 10-14 vpm, H_t = 5.0; for 15-19 vpm, H_t = 3.5; for 20-24 vpm, H_t = 2.7; and for 25-30 vpm, H_t = 2.2.

A similar equation for the Chicago study was developed:

$$\text{mode headway} = \frac{12}{\text{minute volume}} + 1.1 \tag{5b}$$

The projection of the median and mode from the developed Chicago equations indicate that at a volume rate of 35 to 50 vpm, the mean, median, and mode would be approximately equal.

Modes, medians, and means for the four occupancy groups and for the three locations are given in Table 14. Also included are the average modes, medians, and means for the three Chicago locations. The modes for the occupancy groups increased with increased occupancy levels. The medians and means reached their minimum value at 15 to 25 percent occupancy.

Cumulative Frequencies

The 15 and 85 percentile levels, and the 15 to 85 percentile range were calculated for each of the four volume groups and for the three locations and are summarized in Table 15. The 15 percentile level at First Avenue under lower volume conditions occurs at a slightly higher time headway than for other Chicago locations. The results of other Chicago data are quite similar. The 15 percentile level of the Chicago data occurs at a slightly higher time headway than the Detroit data. As would be expected the time headway at the 15 percentile level decreased with increased volume rates.

TABLE 14

CENTRAL TENDENCIES—OCCUPANCY GROUPINGS

Location	Modes (sec)				Medians (sec)				Means (sec)			
	10-14%	15-19%	20-24%	25-30%	10-14%	15-19%	20-24%	25-30%	10-14%	15-19%	20-24%	25-30%
Harlem	1.2	1.2	1.8	2.2	2.0	1.9	2.0	2.4	2.5	2.3	2.3	2.4
Des Plaines	1.2	1.2	1.8	1.8	2.4	2.0	2.1	2.3	3.3	2.5	2.4	2.4
First Ave.	1.2	1.2	-	-	2.5	2.0	-	-	2.9	2.2	-	-
Chicago avg.	1.2	1.2	1.8	2.0	2.3	2.0	2.0	2.3	2.9	2.3	2.3	2.4

TABLE 15

CUMULATIVE FREQUENCY

Location	15% Level (sec)				85% Level (sec)				15-85% Range (sec)			
	10-14 Vpm	15-19 Vpm	20-24 Vpm	25-30 Vpm	10-14 Vpm	15-19 Vpm	20-24 Vpm	25-30 Vpm	10-14 Vpm	15-19 Vpm	20-24 Vpm	25-30 Vpm
Harlem	1.2	1.3	1.2	1.1	11.5	4.8	3.8	3.1	10.3	3.5	2.6	2.0
Des Plaines	1.3	1.2	1.2	1.1	9.0	5.0	3.8	3.2	7.7	3.8	2.6	2.1
First Ave.	1.7	1.4	1.2	1.1	7.8	5.8	4.3	3.4	6.1	4.4	3.1	2.3
Chicago avg.	1.4	1.3	1.2	1.1	9.4	5.2	4.0	3.2	8.0	3.9	2.8	2.1
Detroit avg.	1.3	1.0	0.9	0.8	8.2	6.1	4.6	3.5	6.9	5.1	3.7	2.7

TABLE 16

NUMBER OF INDIVIDUAL TIME HEADWAY OBSERVATIONS

Location	Average Minute Volume Groups				Average Minute Percent Occupancy Groups			
	10-14 Vpm	15-19 Vpm	20-24 Vpm	25-30 Vpm	10-14%	15-19%	20-24%	25-30%
Harlem	1,642	3,034	9,351	7,292	11,010	4,134	1,663	4,493
Des Plains	1,320	3,432	6,327	3,491	4,018	3,420	2,462	4,631
First Ave.	1,667	4,242	6,017	1,927	11,838	1,815	-	-

The 85 percentile level at First Avenue occurs at a slightly higher time headway at the higher volume levels than for other Chicago locations. The 85 percentile level of the Chicago data occurs at a slightly higher time headway at the lowest volume group and at a slightly lower time headway at the higher volume group, when compared with the Detroit data. As would be expected, the time headway at the 85 percentile level decreased with increased volume rates.

The 15 to 85 percentile range, expressed in seconds, is an indication of the uniformity of vehicle time headways. In comparing the Chicago locations, First Avenue has a smaller 15 to 85 percentile range at low volume rates, and a higher 15 to 85 percentile range at the three higher volume rates. It is of interest to note that 70 percent of the vehicles travel within a 2- to $2^1/_2$-sec time headway range when the volume rate is 25 to 30 vpm. The Detroit 15 to 85 percentile range is smaller (more uniform) at the lowest volume group and larger (less uniform) at the three larger volume groups than the Chicago values. As would be expected, the 15 to 85 percentile range decreased as traffic volume increased.

Comparison of Measured Time Headway Distributions Between Levels

Although the individual time headways were initially grouped into 1-vpm and 1 percent occupancy intervals, the intervals were increased for analysis, resulting in fewer groups but more observations in each group (Table 16). It then seemed appropriate to undertake statistical analyses of significant differences between the distributions of adjacent levels to determine whether the groups should be further combined. The Kolmogorov-Smirnov method of analysis was employed to test whether two samples are from populations having the same distribution. The difference in observed frequencies for each $^1/_2$-sec interval was accumulated algebraically and the maximum accumulative value compared with the maximum allowable difference at the 0.05 confidence level. References (46, 47) are available for more detailed description of the Kolmogorov-Smirnov test.

The results of the comparison of time headway distributions between volume levels for the three study locations are given in Table 17. For example, in comparing the Harlem time headway distribution of the 10- to 14-vpm group with the 15- to 19-vpm group, the calculated maximum accumulative difference was 25.8 and the maximum allowable difference at the 0.05 confidence level was 4.2. Therefore, the hypothesis that there is no significant difference between the two time headway distributions was rejected. All hypotheses tested were rejected, indicating significant differences of time headway distributions for the various volume levels at the three study locations.

The results of the comparison of time headway distributions between occupancy levels for the three study locations are given in Table 18. Again, as in the case of the volume levels, all hypotheses tested were rejected, indicating significant differences of time headway distributions for the various occupancy levels at the three study locations.

Comparison of Measured Time Headway Distributions Between Locations

Kolmogorov-Smirnov tests were also utilized in determining whether there were significant differences between time headway distributions obtained at the three study

TABLE 17

COMPARISON OF TIME HEADWAY DISTRIBUTIONS AMONG VOLUME LEVELS

Location	Vpm	10-14 Vpm	15-19 Vpm	20-24 Vpm	25-30 Vpm
Harlem	10-14	-	25.8/4.2	-33.7/3.6	-43.7/3.7
	15-19	-	-	-11.6/2.8	-25.1/2.9
	20-24	-	-	-	-14.1/2.1
Des Plaines	10-14	-	-20.6/4.4	-32.0/4.1	-38.8/4.4
	15-19	-	-	-12.6/2.9	-21.5/3.3
	20-24	-	-	-	-13.2/2.9
First Ave.	10-14	-	-15.1/3.9	-27.7/3.8	-40.9/4.5
	15-19	-	-	-14.0/2.7	-25.8/3.7
	20-24	-	-	-	-13.2/3.6

TABLE 18

COMPARISON OF TIME HEADWAY DISTRIBUTIONS AMONG OCCUPANCY LEVELS

Location	Vpm	10-14 Vpm	15-19 Vpm	20-24 Vpm	25-30 Vpm
Harlem	10-14	-	-10.8/2.5	-11.7/3.6	+20.9/2.4
	15-19	-	-	+12.2/3.9	+24.1/2.9
	20-24	-	-	-	+17.7/3.9
Des Plaines	10-14	-	-13.3/3.2	-17.8/3.5	-17.4/2.9
	15-19	-	-	+ 5.5/3.6	+15.7/3.1
	20-24	-	-	-	+11.1/3.4
First Ave.	10-14	-	-13.5/4.5	-	-
	15-19	-	-	-	-
	20-24	-	-	-	-

TABLE 19

COMPARISON OF TIME HEADWAY DISTRIBUTIONS FOR THREE STUDY LOCATIONS AT SIMILAR VOLUME OR OCCUPANCY LEVELS

Vpm	Des Plaines to Harlem	First to Harlem	First to Des Plaines
(a) Volume Levels			
10-14	-6.9/5.0	-11.2/4.7	+7.5/5.0
15-19	+3.1/3.4	+ 8.4/3.2	+5.5/3.1
20-24	+4.1/2.2	+ 5.3/2.2	+5.3/2.4
25-30	+5.7/2.8	+ 7.9/3.5	+4.1/3.9
(b) Occupancy Levels			
10-14	+9.5/2.5	+ 9.8/1.9	-4.4/2.5
15-19	+7.2/3.1	+ 5.8/3.8	+3.0/3.9
20-24	+5.3/4.3	-	-
25-30	-6.0/2.8	-	-

locations but at the same volume or occupancy level. These results are given in Table 19. In each cell the first number is the calculated maximum accumulative difference, and the second number is the maximum allowable difference at the 0.05 confidence level. All hypotheses tested were rejected except for the 15- to 19-vpm volume group at Des Plaines and Harlem, and the 15 to 19 percent occupancy group at First Ave., and Des Plaines. Therefore, in most cases there was a significant difference between time headway distributions obtained at the three study locations.

Comparison of Measured Time Headway Distributions with Poisson Distribution

A summary of applications of mathematical distributions to time headway distributions was given in Table 4 and indicates that a rather large number of investigators have proposed using the negative exponential distributions derived from the Poisson distribution as representative of time headway distributions, particularly at the lower volume levels. Therefore, χ^2 tests were undertaken for the various locations and volume/occupancy levels to test the hypothesis that there is no significant difference between the observed time headway distributions and the Poisson distribution (negative exponential). The results of the χ^2 tests are given in Table 20, and the hypothesis that there was no significant difference between the observed time headway distributions and the Poisson distribution (negative exponential) was rejected in every case. The degree of freedom was equal to the number of time headway intervals (20) minus 2, or 18 and, therefore, selecting a 1 percent level, the $\chi^2_{n=18,\ \alpha=0.01}$ was equal to 34.8. The only two cells with calculated χ^2 values less than 100 were Cell 1 - 1 (Harlem with 10 to 14 vpm) and Cell 1 - 3 (Des Plaines with 10 to 14 vpm).

Further investigations were undertaken to ascertain what portions of the Poisson distribution provided the best fit for the time headway distributions. Table 21 gives the comparison between the two distributions for each ½-sec time interval and for the 22 cells. A plus sign is shown when the Poisson frequency exceeded the observed frequency, and a minus sign is shown when the observed frequency exceeded the Poisson frequency. To indicate the closeness of fit, the plus and minus symbols were modified. If the χ^2 contribution for the individual cell was less than $\chi^2_{n=18,\ \alpha=0.05} = 28.9$, the plus and minus symbols are not modified. If the χ^2 contribution for the individual cell was greater than $\chi^2_{n=18,\ \alpha=0.05} = 28.9$ but less than ten times $\chi^2_{n=18,\ \alpha=0.05} = 289$, then the plus and minus symbols are placed in parentheses. If the χ^2 contribution for the individual cell was greater than ten times $\chi^2_{n=18,\ \alpha=0.05} = 289$, then the plus and minus symbols are circled.

Table 21 indicates that the portions of the Poisson distributions which provided the best fit for the observed time headway distributions were for time intervals 8 thru 19 (3.5 to 9.5 sec, inclusive). Time interval 8 (3.5 to 4.0 sec) was most frequently minus, whereas, time intervals 10 thru 19 (4.5 to 9.5 sec) were most frequently plus. The portions of the Poisson distribution which provided the worst fit for the observed time headway distributions were for time intervals 1, 4, and 5 (0 to 0.5 sec and 1.5 to 2.5 sec). The time interval 1 (0.0 to 0.5 sec) was always plus, whereas the time intervals 4 and 5 (1.5 to 2.5 sec) were always minus. In time interval 20 (> 9.5 sec), the sign was almost always plus, and in over one-half of the cases it was substantially plus. A generalized graphical comparison is made between the two distributions in Figure 14.

TABLE 20

COMPARISON OF MEASURED TIME HEADWAY DISTRIBUTIONS WITH POISSON DISTRIBUTION

Location	Cell No.	Parameter	Level	Frequencies	Computed χ^2
Harlem	1-1	Volume[a]	10-14	1,642	85
	2-1		15-19	3,034	1,274
	3-1		20-24	9,351	4,446
	4-1		25-30	7,292	4,332
	1-2	Occupancy[b]	10-14	11,010	3,636
	2-2		15-19	4,134	2,278
	3-2		20-24	1,663	1,419
	4-2		25-30	4,493	3,950
Des Plaines	1-3	Volume[a]	10-14	1,320	99
	2-3		15-19	3,432	1,190
	3-3		20-24	6,327	3,561
	4-3		25-30	3,491	2,263
	1-4	Occupancy[b]	10-14	4,018	861
	2-4		15-19	3,420	1,282
	3-4		20-24	2,462	1,434
	4-4		25-30	4,631	3,665
First Ave.	1-5	Volume[a]	10-14	1,667	492
	2-5		15-19	4,242	1,292
	3-5		20-24	6,017	2,558
	4-5		25-30	1,927	1,112
	1-6	Occupancy[b]	10-14	11,838	3,409
	2-6		15-19	1,815	933
	3-6		20-24	-	-
	4-6		25-30	-	-

[a]Vehicles per minute. [b]Percent.

TABLE 21

COMPARISON OF POISSON DISTRIBUTION AND OBSERVED TIME HEADWAY DISTRIBUTIONS[a]

Cell No.	Time Interval[b]																			
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1-1	+	+	+	-	-	-	-	-	+	+	+	+	+	+	+	+	+	-	+	-
2-1	⊕	(+)	-	(-)	(-)	(-)	(-)	-	-	+	+	+	+	+	+	+	+	+	+	(+)
3-1	⊕	(+)	(-)	⊖	⊖	⊖	(-)	-	+	+	+	+	+	+	+	+	+	+	+	(+)
4-1	⊕	(+)	⊖	⊖	⊖	(-)	+	+	+	+	+	+	+	+	+	+	+	+	+	(+)
1-2	⊕	(+)	⊖	⊖	(-)	-	-	-	-	+	+	+	+	+	+	+	+	+	+	-
2-2	⊕	(+)	⊖	⊖	(-)	(-)	-	-	+	+	+	+	+	+	+	+	+	+	+	(+)
3-2	⊕	(+)	-	⊖	⊖	(-)	-	+	+	+	+	+	+	+	+	+	+	+	+	+
4-2	⊕	⊕	(+)	⊖	⊖	⊖	(-)	-	+	+	+	+	+	+	+	+	+	+	+	+
1-3	+	+	-	-	-	-	-	-	-	-	-	-	-	-	+	-	-	+	+	+
2-3	⊕	(+)	-	(-)	⊖	(-)	(-)	-	-	+	+	+	+	+	+	+	+	+	+	(+)
3-3	⊕	⊕	(-)	⊖	⊖	⊖	(-)	-	+	+	+	+	+	+	+	+	+	+	+	(+)
4-3	⊕	(+)	(-)	⊖	(-)	(-)	-	+	+	+	+	+	+	+	+	+	+	+	+	(+)
1-4	⊕	(+)	(-)	(-)	(-)	-	-	-	-	+	+	-	+	+	+	+	+	-	+	+
2-4	⊕	(+)	(-)	(-)	(-)	-	-	-	+	+	+	+	+	+	+	+	+	+	+	+
3-4	⊕	(+)	(-)	⊖	⊖	(-)	-	-	+	+	+	+	+	+	+	+	+	+	+	+
4-4	⊕	⊕	+	⊖	⊖	⊖	(-)	-	+	+	+	+	+	+	+	+	+	+	+	(+)
1-5	(+)	(+)	+	-	-	(-)	(-)	-	-	-	-	-	-	+	-	-	+	+	-	(+)
2-5	⊕	(+)	(-)	(-)	(-)	(-)	(-)	-	-	-	-	-	+	-	-	-	+	-	+	(+)
3-5	⊕	(+)	(-)	⊖	⊖	(-)	(-)	-	-	-	+	+	+	+	+	+	+	+	+	(+)
4-5	⊕	(+)	( )	(-)	⊖	-	-	-	+	+	+	+	+	+	+	+	+	+	+	+
1-6	⊕	(+)	(-)	⊖	⊖	(-)	(-)	-	-	-	+	+	+	-	-	+	-	+	-	(+)
2-6	⊕	(+)	(-)	(-)	(-)	(-)	-	-	-	-	+	+	+	-	+	-	+	+	+	+

[a]Legend:

	Slight	Considerable	Large
Poisson value > measured value	+	(+)	⊕
Poisson value < measured value	-	(-)	⊖

[b]For ½-sec intervals between 0 and > 9.5 sec.

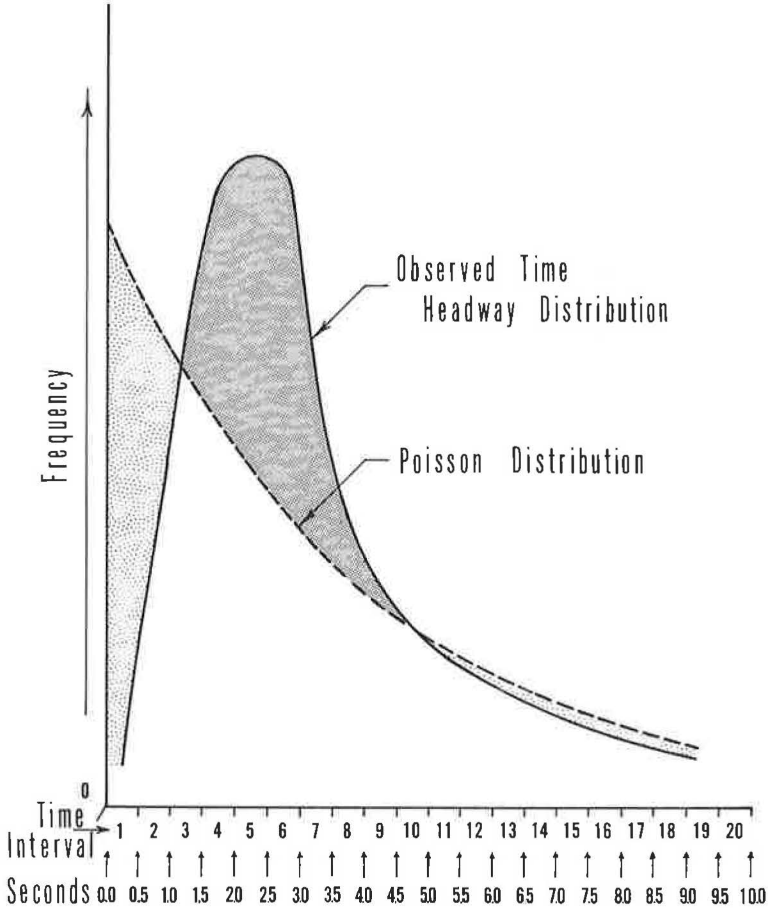

Figure 14. Generalized comparison between Poisson distribution and observed time headway distribution.

Attempts were made to improve the fit between measured time headway distributions and the Poisson distribution by modifying the distributions. The three attempts were to use (a) larger time intervals, (b) cumulative frequencies, and (c) shifted Poisson distribution.

It was thought that if time headway intervals greater than $1/2$ sec were employed, such as 1 or 2 sec, the result might be an acceptance of the hypothesis that there is no significant difference between the observed and Poisson distribution. Table 22 summarizes the resulting calculated χ^2 values for each cell and for the three time intervals: $1/2$ and 2 sec. The χ^2 values given in this table exceeded the $\chi^2_{n=18,\ \alpha=0.01} = 34.8$ when $1/2$-sec intervals were used, exceeded the $\chi^2_{n=8,\ \alpha=0.01} = 20.1$ when 1-sec intervals were used, and exceeded the $\chi^2_{n=3,\ \alpha=.01} = 11.3$ when 2-sec intervals were used. Cells 1 - 1 and 1 - 3 exhibited the best comparison between distributions at all three time intervals selected. When 1-sec intervals were used, the computed χ^2 values were slightly less than when $1/2$-sec intervals were used. But the χ^2 value at the 0.01 level was considerably reduced, and the net result was a larger indication of differences between the measured 1-sec interval distribution and the Poisson distribution. The use of 2-sec intervals was slightly better than when $1/2$-sec intervals were used.

It was thought that if cumulative frequencies were employed, the result might be an acceptance of the hypothesis that there is no significant difference between the observed

TABLE 22

COMPUTED χ^2 VALUES

Cell No.	Straight Frequency			Cumulative Frequency			Shift ½-Second Interval	
	½-Sec	1-Sec	2-Sec	½-Sec	1-Sec	2-Sec	½-Sec	1-Sec
1-1	85	65	49	128	56	20	103	611
2-1	1,274	1,050	654	1,260	588	383	696	269
3-1	4,446	4,908	1,528	4,465	2,030	351	1,794	179
4-1	4,332	4,690	750	3,208	1,404	100	1,433	71
1-2	3,636	1,391	277	3,333	1,391	48	1,454	1,208
2-2	2,278	2,196	397	1,643	888	52	783	177
3-2	1,419	1,231	367	874	523	51	769	135
4-2	3,950	3,384	2,402	3,810	1,778	546	2,591	998
1-3	99	88	26	122	62	9	16	89
2-3	1,190	1,090	516	1,263	590	121	521	117
3-3	3,561	3,195	1,411	3,218	1,512	151	1,723	219
4-3	2,263	2,116	464	1,746	833	58	900	82
1-4	861	717	66	958	401	17	166	319
2-4	1,282	1,175	190	1,237	598	29	356	142
3-4	1,434	1,296	411	1,249	596	60	611	196
4-4	3,665	3,171	1,898	3,426	1,599	371	2,100	658
1-5	492	385	255	637	286	85	164	72
2-5	1,292	1,191	499	1,705	769	138	464	39
3-5	2,558	2,184	674	2,163	1,174	123	844	67
4-5	1,112	947	256	954	414	34	416	60
1-6	3,409	3,010	864	4,607	2,024	276	1,010	8,863
2-6	933	866	171	955	433	32	349	32
$\chi^2_{\alpha=0.01}$	34.8 $n = 18$	20.1 $n = 8$	11.3 $n = 3$	34.8 $n = 18$	20.1 $n = 8$	11.3 $n = 3$	33.4 $n = 17$	32.0 $n = 16$

and Poisson distributions. The χ^2 values given in Table 22 for cumulative frequencies exceeded the $\chi^2_{n=18,\ \alpha=0.01} = 34.8$ when ½-sec intervals were used, exceeded the $\chi^2_{n=8,\ \alpha=0.01} = 20.1$ when 1-sec intervals were used, and exceeded the $\chi^2_{n=3,\ \alpha=0.01} = 11.3$ when 2-sec intervals were used, except for Cell 1 - 3. When the cumulative frequency with a 2-sec interval was employed for the measured headway distribution of Cell 1 - 3 (Des Plaines, 10 to 14 vpm), the hypothesis that there is no significant difference between the observed and Poisson distributions could not be rejected at the 0.01 level (however, it would be rejected at the 0.05 level). Again Cells 1 - 1 and 1 - 3 exhibited the best comparison between distributions at all three time intervals selected. There was a slight improvement when 1-sec intervals were employed, and a considerable improvement when 2-sec intervals were used.

The last modification was to shift the Poisson distribution ½ and 1 sec and compare these shifted Poisson distributions with the straight (½-sec) frequency of the measured time headway distributions. The resulting χ^2 values (Table 22) exceeded the $\chi^2_{n=16,\ \alpha=0.01} = 32.0$ for both the ½- and 1-sec shift for all cells, except for Cell 1 - 3 when a ½-sec shift was used and for Cell 2 - 6 when a 1-sec shift was employed. In comparing the three columns of computed χ^2 values (no shift, ½- and 1-sec shift), the computed values of χ^2 decreased with an increase shift for 18 of the 22 cells tested. Cell 1 - 1 exhibited more favorable comparisons with no shift, whereas Cells 1 - 3, 1 - 4, and 1 - 6 exhibited more favorable comparisons with a ½-sec shift.

In summarizing the various analyses undertaken to test the hypothesis that there is no significant difference between measured time headway distributions and the Poisson distribution, it has become evident that except in a few isolated instances, the Poisson distribution cannot be used to represent the time headway distributions obtained at the three locations under the volume and occupancy conditions studied. It is apparent that the best fits were obtained at the lowest volume levels studied (10 to 14 vpm or 600 to

940 vph). This indicates that at lower volumes the Poisson distribution is more representative of the measured time headway distributions. This confirms results of earlier studies. However, for most applications in freeway operations, the time headway distributions for the higher volume conditions are more frequently used, and consequently, there does seem to be a need for mathematical distributions in addition to the Poisson distribution. Modification of the measured time headway distributions by using larger time intervals and cumulative frequencies and of the Poisson distribution by a shift of 1.0 sec, result in improved "fits" between the measured and Poisson distributions. Even with these modifications, the Poisson distribution does not appear to be satisfactory for locations and traffic conditions studied.

Investigation of Other Mathematical Distributions

Preliminary investigations were made of a composite normal-Poisson distribution and the log-normal distribution as applied to the measured time headway distributions for the various volume groups discussed in previous sections of this paper. The development of the composite normal-Poisson distribution was patterned after the earlier work of Schuhl (29) and Kell (32, 33). It is assumed that the vehicles in the traffic stream are either restrained (in a platoon) or free flowing (out of a platoon). It is also assumed that the time headways of vehicles in platoons are normally distributed, whereas the time headways of vehicles out of platoons follow the Poisson distribution.

In regard to vehicles within platoons, a mean time headway of 1.5 sec was taken, assuming that maximum flow occurred when essentially all vehicles were in platoon, and the flow rate was 2,400 vph/lane. The normal distribution would be symmetrical about the mean time headway and would include individual time headways from 0.0 to 3.0 sec (1.5 sec either side of the mean time headway). The mean time headway ± 3 std. dev. includes 99.7 percent of individual observations associated with a normal distribution, and, therefore, a standard deviation of 0.5 sec resulted. The next step was to determine the proportion of the vehicles in the traffic stream which would be considered to be in a platoon. This was accomplished on the basis that the proportion of vehicles having headways from 0 to 1.5 sec represented 50 percent of the vehicles in platoons. This proportion was obtained from the time headways measured at the

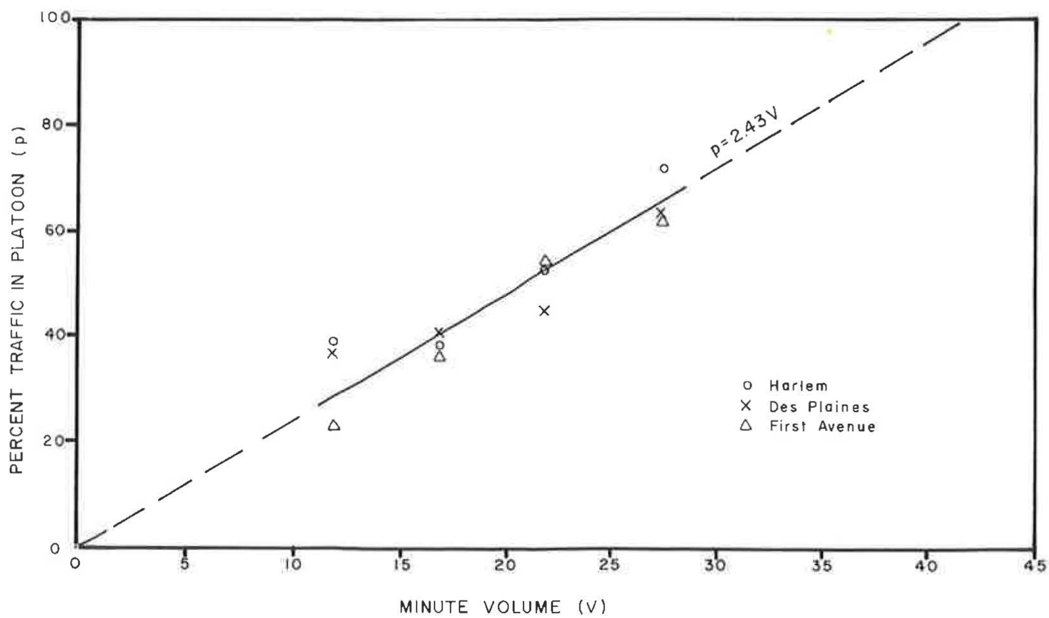

Figure 15. Estimating percent traffic in platoon from minute volumes.

three locations and at the four volume levels. Figure 15 is a plot of the proportion of the vehicles in platoons vs the minute volumes. The equation of the line which best represents the data points and goes through the origin is p = 2.43 V, where p is the proportion of vehicles in platoons and V is the minute volume. All vehicles are in platoons (p = 100 percent), when the volume is 41.2 vpm or 2,470 vph. This supports the earlier assumption that the mean time headway of vehicles in platoons is 1.5 sec; when all vehicles are in platoons, this results in a flow rate of 2,400 vph.

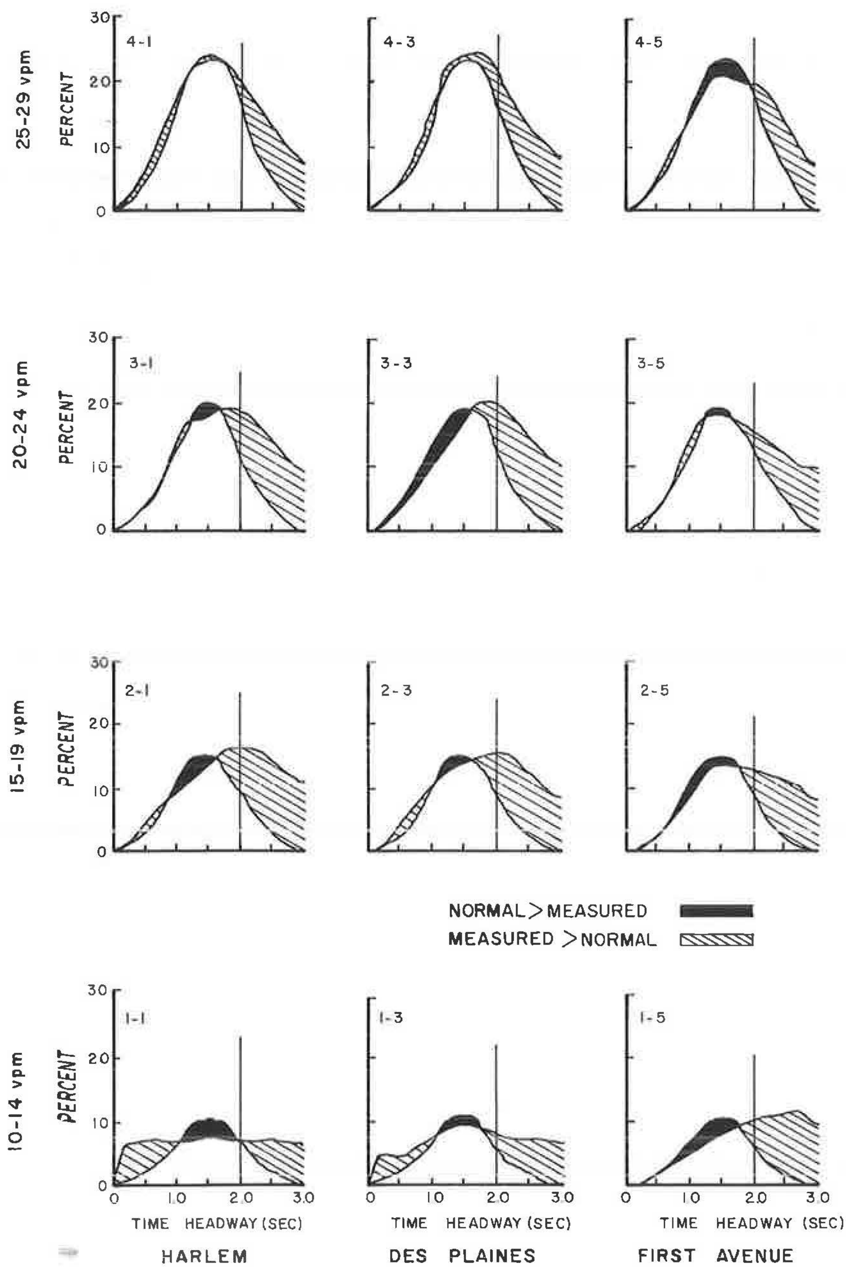

Figure 16. Headway distributions of vehicles in platoons.

A comparison of the measured time headway distributions (for the three locations and four volume groups) with the assumed normal distribution of time headways of vehicles within platoons is shown in Figure 16. The darkened areas indicate when the normal distribution gives higher frequencies than the measured distribution, whereas the hatched areas indicate when the measured distribution gives higher frequencies than the normal distribution. At time headways over 2 sec, the measured distribution always gave higher frequencies than the normal distribution, which indicates that vehicles outside of platoons influence the measured distribution at headways greater than 2.0 sec. The largest differences between the normal and measured distributions occurred at the lowest volume levels (10 to 14 vpm).

In developing the composite normal-Poisson distribution, the Poisson distribution characteristics associated with the vehicles out of platoons was next determined. The proportion of vehicles not in platoons and the mean time headway of these vehicles were computed for each volume group (Table 23). By setting the minimum time headway for vehicles outside of platoons as 2.0 sec and assuming a Poisson distribution (negative exponential), the distribution of time headways was determined. The composite normal-Poisson distribution obtained from the vehicles within and outside of platoons was compared with the measured time headway distributions, and the χ^2 test was applied to test the hypothesis that there was no significant difference between the measured time headway distributions and the selected composite normal-Poisson distribution. The results of the χ^2 tests are given in Table 24. The computed χ^2 values for the normal-Poisson distribution were considerably less than those from the Poisson distribution (compare Tables 22 and 24). However, the computed χ^2 values exceeded the corresponding table χ^2 values ($\alpha = 0.01$) in every case, except for Cells 4-3 and 4-5

TABLE 23

CHARACTERISTICS OF VEHICLES WITHIN AND OUTSIDE PLATOONS

Volume Range	Total Vehicles		Vehicles in Platoons		Vehicles Out of Platoons	
	Avg. Vol	Mean Time Headway (sec)	Percent	Mean Time Headway (sec)	Percent	Mean Time Headway (sec)
10-14	12	5.00	29.2	1.50	70.8	6.45
15-19	17	3.53	41.3	1.50	58.7	4.96
20-24	22	2.73	53.5	1.50	46.5	4.15
25-30	27.5	2.18	66.8	1.50	33.2	3.56

TABLE 24

COMPARISON OF MEASURED TIME HEADWAY DISTRIBUTIONS WITH COMPOSITE NORMAL-POISSON DISTRIBUTION

Location	Cell No.	Vol. (vpm) Level	½-Sec Interval	1-Sec Interval	2-Sec Interval	½-Sec Interval	1-Sec Interval	2-Sec Interval
Harlem	1-1	10-14	900	323	38	1,051	625	209
	2-1	15-19	356	157	151	197	189	170
	3-1	20-24	334	275	219	180	171	150
	4-1	25-30	135	90	50	53	51	40
Des Plaines	1-3	10-14	411	99	15	479	250	95
	2-3	15-19	238	99	88	128	125	112
	3-3	20-24	370	259	243	185	163	146
	4-3	25-30	111	74	36	31	14	8
First Ave.	1-5	10-14	118	52	47	73	63	54
	2-5	15-19	128	65	35	71	41	37
	3-5	20-24	318	126	58	80	46	30
	4-5	25-30	67	48	23	31	17	16
$\chi^2_{\alpha = 0.01}$			32.0 n = 16	16.8 n = 6	6.6 n = 1	32.0 n = 16	16.8 n = 6	6.6 n = 1

when a cumulative frequency with a 1-sec time interval was used. The composite normal-Poisson distribution is particularly better than the Poisson distribution at the higher volume levels. The portions of the measured time headway distributions which showed the greatest difference from the normal-Poisson distribution (and consequently contributed to the computed χ^2 value) were the time intervals 2.5 to 2.9 and 3.0 to 3.4 sec. The frequency of the measured distribution exceeded the normal-Poisson distribution frequency for the two time intervals in all cases, with average differences of 1.9 and 1.7 percent, respectively. It is of interest to note how a relatively small difference can cause the rejection of hypothesis that there is no significant difference between a measured and a mathematical distribution. This is graphically illustrated in Figure 17, where the measured and normal-Poisson distributions are compared for the four volume groups at Harlem Avenue.

Daou (48) and Greenberg (49) proposed the use of the log-normal distribution for describing time headway distributions. The works of Hald (50) and Aitchison and Brown (51) are general references describing the log-normal distribution. In a discussion of the paper by Dauo (48), Foote includes quotations from Aitchison and Brown (51) and

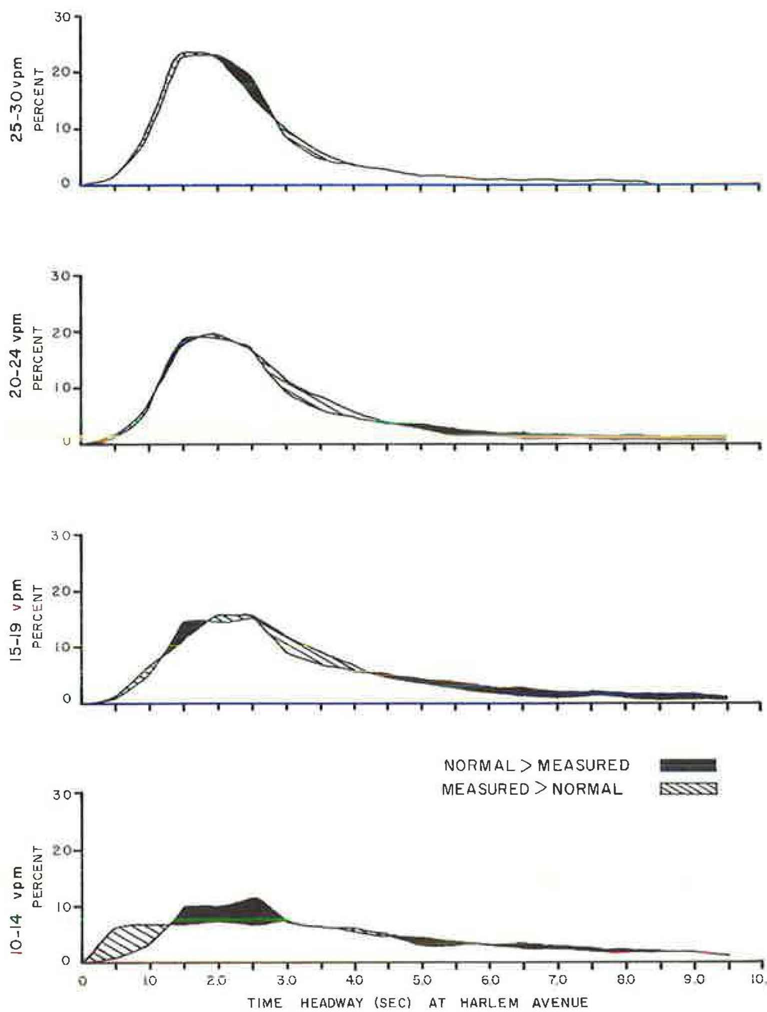

Figure 17. Comparison of measured time headway distributions with composite normal-Poisson distribution.

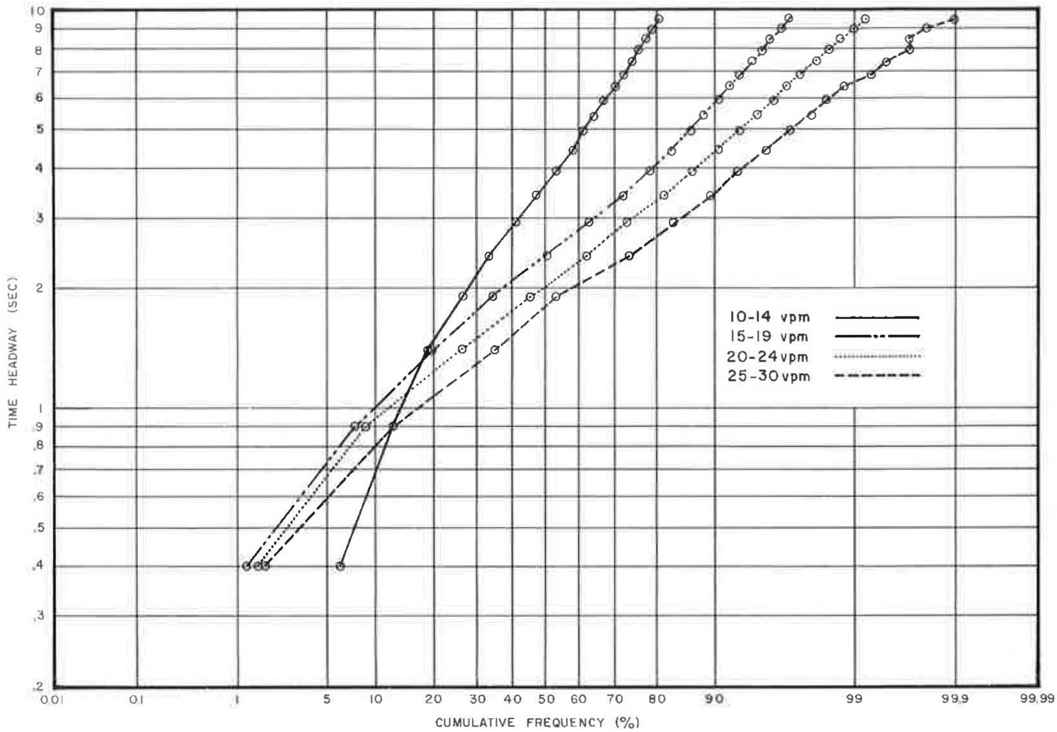

Figure 18. Log-normal plot of time headway distributions—Harlem Avenue.

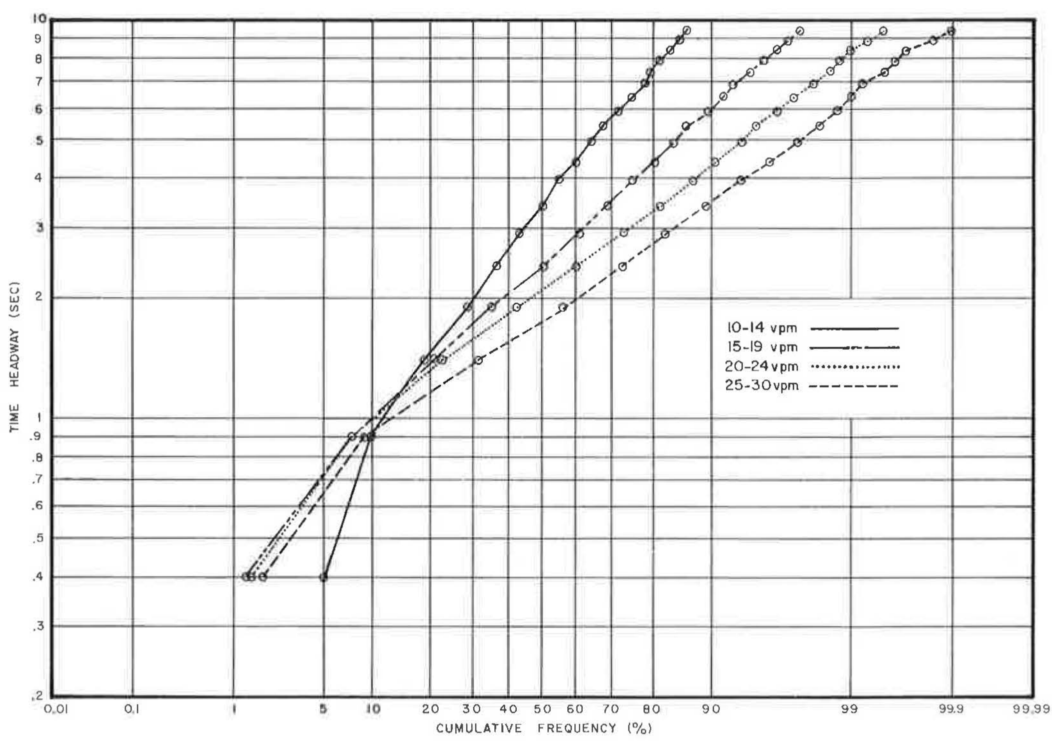

Figure 19. Log-normal plot of time headway distributions—Des Plaines Avenue.

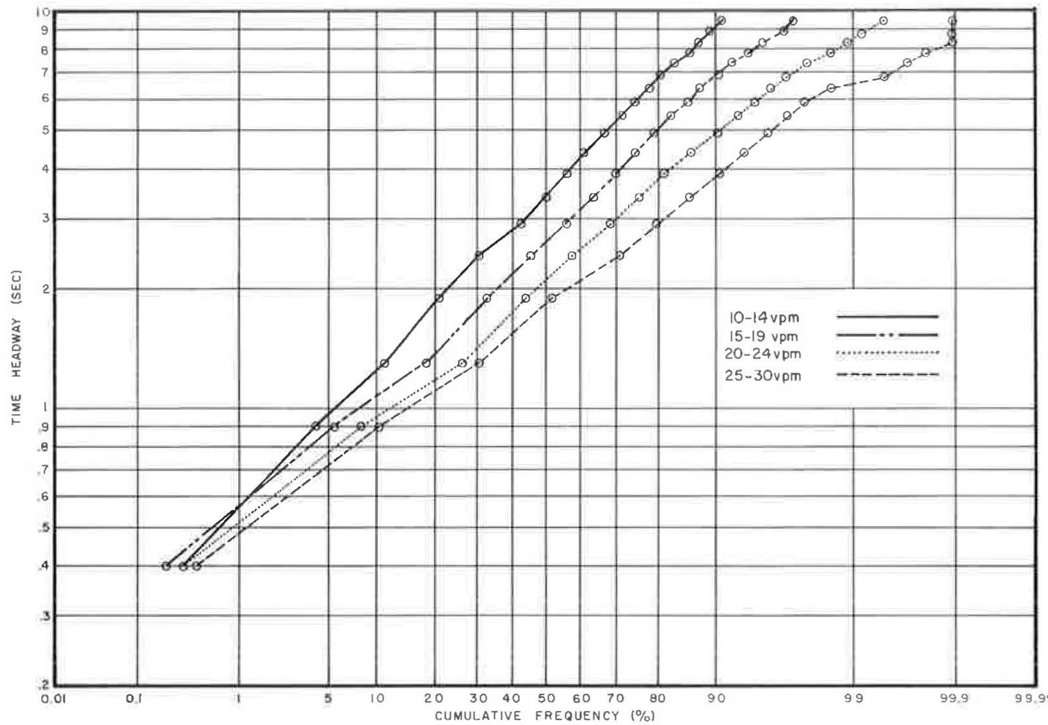

Figure 20. Log-normal plot of time headway distributions—First Avenue.

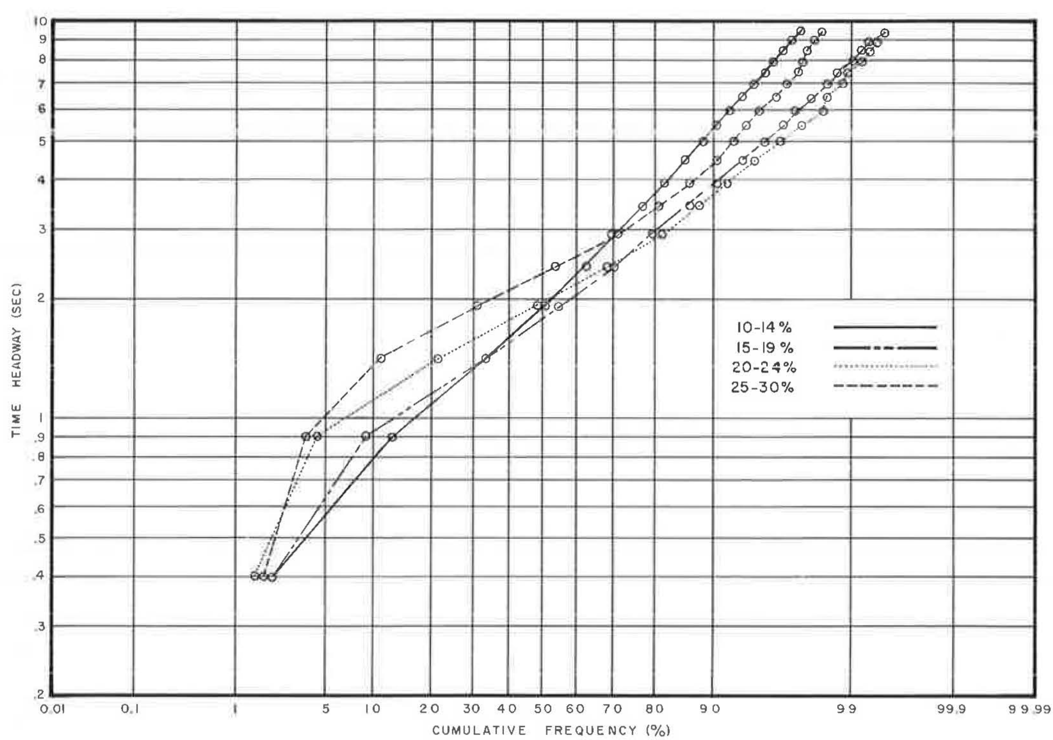

Figure 21. Log-normal plot of time headway distributions—Harlem Avenue.

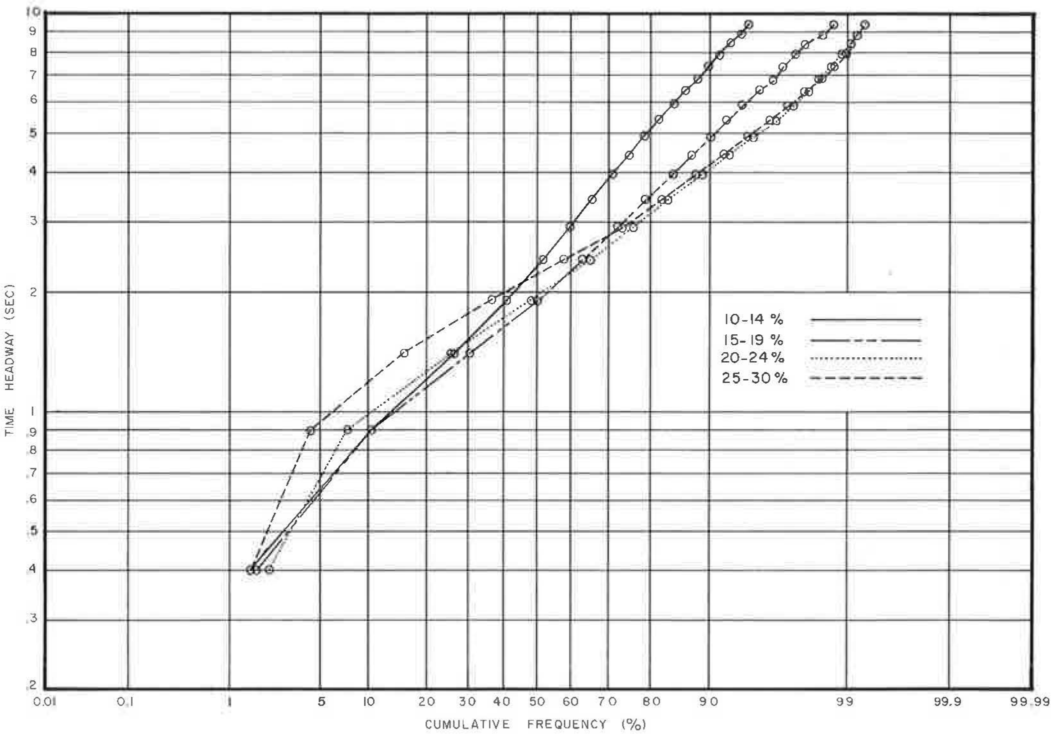

Figure 22. Log-normal plot of time headway distributions—Des Plaines Avenue.

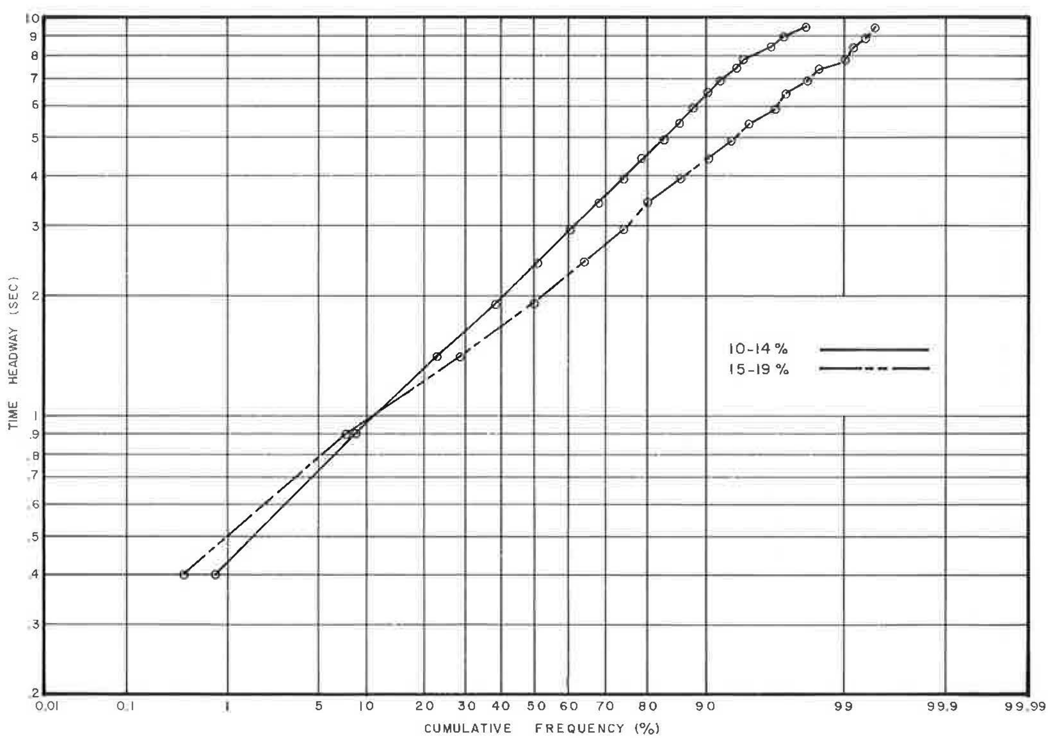

Figure 23. Log-normal plot of time headway distributions—First Avenue.

concludes that "the use of the log-normal distribution to describe headways selected by populations of motorists traveling at nearly equivalent speeds may have a base in theory as well as experiment." This encouraged the author to investigate the use of the log-normal distribution to describe time headway distributions for the volume and occupancy levels obtained in this study. Log-normal plots of the measured time headway distributions for the three locations based on the volume level groupings are shown in Figures 18, 19, and 20, whereas those based on the occupancy level groupings are shown in Figures 21, 22, and 23. All log-normal plots based on volume level groupings are quite linear, particularly at time headways greater than 1 sec. As the volume level increases, the cumulative frequency shifts to the right and has a flatter slope, indicating more uniformity in time headways. A horizontal line, for example, would indicate all vehicles traveling at equal headways and the standard deviation would be zero.

χ^2 tests were not undertaken to test the hypothesis that there is no significant difference between the measured time headway distributions and the log-normal distribution. Inspection of the log plots are encouraging because of their linearity and their relative positions. However, some preliminary calculations comparing some characteristics of the measured and log-normal distributions were disconcerting. For example, when straight lines were superimposed over each plot, there were occasional differences of 2 percent between the straight line and the individual plotted points. As indicated in the discussion of the normal-Poisson distribution, differences of 2 percent could result in rather large computed χ^2 values. Hald (50) gives equations for estimating the standard deviation of a log-normal distribution and for relating means, medians, and modes. An estimate of the standard deviation is obtained by subtracting the log value of the headway associated with the 15.9 percent cumulation frequency from the log value of the headway associated with the 50 percent cumulative frequency. The equations as given by Hald (50) for central tendencies of log-normal distributions are as follows:

median = log x = log of the headway corresponding to the 50 percent cumulative frequency (6)

$$\text{mean} = \log x + 1.1513\,\sigma^2 \qquad (7)$$

$$\text{mode} = \log x - 2.3026\,\sigma^2 \qquad (8)$$

Utilizing these equations, an estimate of the standard deviation was determined, as well as estimates for the means and modes of the log-normal distributions (Table 25). The standard deviation of the measured distributions had not been computed, so comparisons with the log-normal estimated standard deviations were not possible. The modes shown for the measured distributions are for ½-sec intervals and not for individual headways; therefore, they cannot be compared directly with the computed modes of the log-normal distributions. The most realistic comparison was between the means of the measured and the log-normal distributions. The means appear to be considerably different, and one set of means not consistently greater than the other.

More analyses will have to be undertaken in reference to the log-normal distribution, particularly to perform the χ^2 tests. The results thus far are encouraging and warrant this further work.

The measured time headway distributions based on occupancy level groupings (Figs. 21, 22, and 23) were not as encouraging as those based on volume level groupings. At low occupancy levels the log plots appeared to be linear, whereas at the higher occupancy levels the log plots did not appear to be linear. Further work was not undertaken with the measured time headway distributions based on occupancy level groupings.

Estimation of Metering Rates

One of the important areas investigated in connection with this paper was the estimation of metering rates which might be employed in the Chicago Expressway Sur-

TABLE 25

COMPARISON OF MEASURED HEADWAY AND LOG-NORMAL DISTRIBUTION CHARACTERISTICS

Location	Volume Range	Median	Std. Dev. Est.	Means		Modes	
				Measured	Log-Normal	Measured	Log-Normal
Harlem	10-14	3.60	3.19	5.00	7.04	1.8	0.94
	15-19	2.40	2.00	3.53	3.05	1.8	1.48
	20-24	2.00	1.85	2.72	2.42	1.8	1.37
	25-30	1.72	1.72	2.18	1.99	1.2	1.28
Des Plaines	10-14	3.23	2.60	5.00	5.11	1.8	1.29
	15-19	2.40	2.00	3.53	3.05	2.2	1.48
	20-24	2.10	1.75	2.72	2.46	1.8	1.54
	25-30	1.80	1.71	2.18	2.08	1.8	1.35
First Ave.	10-14	3.35	2.12	5.00	4.44	2.8	1.90
	15-19	2.56	2.13	3.53	3.41	1.8	1.44
	20-24	2.09	1.83	2.72	2.51	1.2	1.45
	25-30	1.80	1.76	2.18	2.11	1.8	1.30

veillance Project's ramp metering program. The approach employed was combining of the gap availability (time headway) distributions and the gap acceptability distributions for various volume and occupancy levels to estimate the number of available acceptable gaps for different volume and occupancy levels. The measured time headway distributions were available as gap availability data, and the results of previous gap acceptability studies were reviewed in an earlier portion of this paper. However, of the five gap acceptability studies (4, 29, 31, 44, 45), only the study conducted by Texas A&M for a Midwest Research Institute ramp simulation project (31) was conducted on a freeway where ramp vehicles merged without starting from a stopped position. It should be mentioned that the earlier studies were static studies; that is, the time headway measured at the ramp nose of vehicles in the shoulder lane was assumed to remain constant. Other complexities should also be considered, such as multiple vehicle entries and reclassifying acceptable gaps as those which have been accepted and have not adversely affected vehicles in the mainstream.

The gap acceptance data (31) were combined with the measured time headway distribution data, and the resulting number of acceptable available gaps for the four volume groups and the four occupancy groups are summarized in Table 26. This analysis results in an increased number of acceptable available gaps as the shoulder lane volume and occupancy level increases. This is contrary to what would be expected and to the existing ramp metering scheme. One would anticipate that as the shoulder lane volume and occupancy increase, the allowable number of ramp vehicles that could merge would decrease. Some comments were made previously as to some limitations of the existing gap acceptance studies. It is proposed that before additional work can be effectively undertaken in this area, a rather comprehensive gap acceptance study be carried out. It is suggested that the following thoughts be considered if such a study is undertaken:

1. Approach the study on a microscopic basis to the extent that the distance-time plots of individual vehicles in the shoulder lane and on the ramp can be traced through the merge area and studied in great detail.
2. Consider a dynamic means rather than a static means of measuring gap sizes.
3. Classify gap acceptance data by volume, speed, and occupancy (or density) levels.

TABLE 26

ACCEPTABLE AVAILABLE GAPS[a]

Location	Volume Levels				Occupancy Levels			
	10-14 Vpm	15-19 Vpm	20-24 Vpm	25-30 Vpm	10-14%	15-19%	20-24%	25-30%
Harlem	9.4	12.6	15.2	17.5	8.0	11.2	15.1	20.6
Des Plaines	9.4	12.6	12.9	17.9	8.7	11.5	15.1	20.0
First Ave.	8.9	13.0	15.6	18.1	8.8	11.6	-	-

[a]Values given as no./min.

4. Differentiate between merges involving ramp vehicles which stopped before entering and those involving ramp vehicles which were free flowing.
5. Differentiate between types of vehicles both on the ramp and in the shoulder lane.
6. Measure multiple vehicle entries.
7. Classify gap acceptance data by level of ramp demand (pressure on lead vehicle to force a merge).
8. Measure the effect of a gap being accepted on the traffic in the shoulder lane and relate it to gap acceptance data.

SUMMARY

1. The central tendency measurements indicated that the mode was always less than the median, and the median was always less than the mean headway. Equations were developed between the central tendency measurements and indicated that at volume rates of 35 to 50 vpm, the mode, median, and mean would be approximately equal.
2. The analyses of the cumulative frequencies indicated that as the volume level increases, the time headways become more uniform. At 10 to 14 vpm, 70 percent of the vehicles had headways between 1.4 and 9.4 sec. At 25 to 30 vpm, 70 percent of the vehicles had headways between 1.1 and 3.2 sec.
3. There was a significant difference between the measured time headway distributions for the volume levels (10 to 14, 15 to 19, 20 to 24, and 25 to 30 vpm) and occupancy levels (10 to 14, 15 to 19, 20 to 24, and 25 to 30 percent) selected at the three study sites.
4. The comparison of measured time headway distributions between locations indicated significant differences, except in two cases under low-volume or low-occupancy conditions.
5. There was a significant difference between each measured time headway distribution and the Poisson distribution (negative exponential).
6. The use of larger time intervals and cumulative frequencies, and shift of the Poisson distribution, provide closer agreement between the measured and Poisson distribution. Even with these modifications, the Poisson distribution does not appear to be satisfactory for locations and traffic conditions studied.
7. The composite normal-Poisson distribution and the log-normal distribution show promise as being representative of measured time headway distributions. It is recommended that additional work be undertaken in this area.
8. Combining the measured gap availability distributions with the results of existing gap acceptance data to estimate the desired metering rates for various levels of volume and occupancy gave disconcerting results. Essentially the results indicated that as the volume and occupancy level increased, more ramp vehicles could merge into the expressway.
9. It is recommended that a comprehensive gap acceptance study be undertaken, and suggestions to be considered in designing such a study are included in the body of this report.

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