HIGHWAY RESEARCH BOARD

Bulletin 351

Traffic Characteristics and Intersection Capacities

I. Traffic Characteristics as Related to Highway Capacity

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Effect of Small and Compact Cars On Traffic Flow and Safety

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Time-headways between regular, small, and compact vehicles were used to study the effect of the latter on maximum traffic flow and therefore on highway capacity. Time-lapse photographic recording was the method used. From the records at each location, four complete 5-min samples were read to determine the normal complete distribution of time-headways. The film was then searched for small and compact cars and, for each of these, 4-car groups were recorded to give headways before and behind the smaller vehicle and a comparison headway between two regular vehicles ahead.

Records were made for peak and off-peak periods for six locations. Four of the locations were on Detroit expressways and two were on divided highways in Lansing, Mich.

Statistical analysis included comparison of the expected number of small vehicles with the actual number observed in the lower 15 and higher 15 percent of the time-headway distribution. Chi-square comparisons indicated no statistically significant differences from the expected number.

After eliminating stragglers, time-headway averages for the three different types of cars were compared by means of Fisher's t-function for lanes 1 and 2 at each location, combining lanes 1 and 2 and combining the three locations at which headways were similar. Time-headway differences between the different types of vehicles occurred in the different samples. However, these were not consistent in direction (headways between regular vehicles were not consistently larger or smaller than those including a small or compact vehicle). The differences were not statistically significant.

It was concluded that drivers of both the smaller vehicles and the surrounding standard automobiles operated in both peak and off-peak traffic in a way similar to operation of standard cars. Therefore, no consistent increase or decrease of traffic headways or of highway capacity is to be expected from inclusion of small and compact vehicles in the traffic stream.

There was some indication that time-headways may be of greater significance than distance-headways for certain purposes.

•IN THE LAST four or five years the increasing number of small low-powered European cars and the compact cars developed by U.S. automobile companies appearing on the roads in greater numbers have raised various problems. One of the claims for these cars is greater gasoline economy. Some of those interested in taxation problems have indicated concern about possible effects on gasoline tax income for highways. The questions have been raised, therefore, whether the smaller vehicles use a smaller, equal, or larger share of the highway because of their operating characteristics and the reactions of their own drivers and drivers of standard-size vehicles in the traffic stream.

As the proportion of smaller vehicles increases, it would be desirable to know enough about their effect on traffic flow to be able to predict whether they will increase, decrease, or leave unaffected the maximum capacity of highways, especially of the more expensive, free-flowing freeway design type of highways.

In addition to these questions, some have expressed opinions that the smaller vehicles introduce greater accident hazards on highways because of reduced visibility from within, and greater difficulty by other drivers in seeing the vehicle or seeing through it to other vehicles in the traffic stream. On the other hand, others have held that the small vehicles are as safe, or safer, because of greater maneuverability and because other drivers can see over them to other vehicles in the traffic stream. If either of these opinions represent an influence strong enough to affect the driving behavior of those operating the smaller vehicles or of the regular vehicles surrounding them, this should be reflected in vehicle spacings or headways while operating in traffic.

METHOD

If the small and compact vehicles are using more or less than their share of highway, a study of vehicle spacing and time-headways at different speeds should indicate this. The especially critical times for the problem would be those during the peak traffic flow when capacity of the highway is being taxed to the utmost. However, it is possible that in the off-peak periods any possible visibility hazard from the small cars felt by their own or other drivers might be indicated by headways during the lighter traffic hours.

Therefore, as a means of investigating the problem, a study of time and distance-headways was carried out using data from four expressway locations in Detroit and two divided highway locations in Lansing, Mich. Traffic flow was recorded for a minimum of eight hours on each of two days at the four Detroit locations from 7:00 to 11:00 AM and from 2:00 to 6:00 PM (with minor variations occasioned by different conditions). Thus, both morning and afternoon peak flow as well as off-peak flow were recorded.

The two Lansing locations were on divided highways near the downtown business section, using afternoon outbound traffic to get as heavy a flow as possible. It was found that only the peak hours from about 3:00 to 6:00 PM gave high enough volumes for the recorded headways to be of significance in reflecting driver judgments. Therefore, from three to four such periods were recorded. Some off-peak flow was also recorded but was so sparse that an analysis was not worthwhile. At both of these locations a traffic signal at a preceding intersection served to group or bunch up traffic so that it flowed through the experimental zone in close formation. (Distance from preceding signal to first marker was approximately 650 ft at Saginaw and 900 ft at S. Cedar locations.) These locations were used to see whether any markedly different relationships in the headways would be shown in such traffic.

Recording was accomplished by time-lapse photography from an overhead bridge, using a method reported previously (1). Three pairs of reference marks were painted on the gutter and curb of one side of the divided highway. A 16-mm camera with electronic timer was positioned on a bridge at the selected locations, each of which used straight and nearly level highway where possible. (Where a grade was present, corrections for it were included when the analysis grid was calculated.) Measurements for three pairs of reference marks at 100-ft intervals on the pavement edge and the distance and the elevation to the camera were surveyed in. Each reference mark was a white painted stripe about 2 ft wide and from 2 to 4 ft long. It did not extend into the roadway much beyond the gutter section. This is of importance to prevent drivers from feeling that something special is going on at this particular location and, therefore, altering their driving behavior. The short stripes on either side of the roadway were hardly noticeable from a car unless given special attention. In attention, the camera was always located on the far side of the bridge in the direction of flow so as to record the rear of the vehicles as they progressed away from the bridge. This, again, reduced driver attention to the camera equipment.

A few people coming in the opposite direction and using the cross-street stopped to ask what was going on. When told that "this was a traffic survey," they usually did not tarry.

EXPERIMENTAL LOCATIONS

Of the four freeway locations in Detroit, three were outbound (at Grand Boulevard, eastbound; Seward, northbound; and Addison, westbound) and one was inbound (at the Lonyo overpass which is near Edison Street). Thus, for the most part, traffic was flowing in such a direction as to disperse rather than back up because of congestion in downtown areas ahead. At one location on the Edsel Ford Freeway, there was both an inbound and an outbound location close together (Lonyo and Addison).

The Detroit Expressway records were made on two weekdays at each location during August and September 1960. Recording at the Lansing locations was during the winter of 1960 and spring of 1961. In each case, traffic flow was recorded only during clear, dry weather to eliminate possible effects from wet pavement and other weather conditions.

ANALYSIS OF RECORDS

Measurements from Film Record

The film records consisted of time-lapse photographs taken at 1-sec intervals for a total of 8 hr on each of two days at the Detroit locations. This totaled approximately five 100-ft rolls of 16-mm pictures each day for two days at each of four locations. Thus, a total of 4,000 ft of time-lapse photographic records resulted. At the Lansing locations, a total of five 100-ft rolls at each location, or 1,000 ft of photographic record, were obtained.

From these records, distances and speeds were obtained from the film by the method of Forbes and Fairman (1). In this method, a grid is calculated which eliminates the parallax error as the picture is projected to determine distances on a grid. By placing the grid on a darkroom floor, the photographic records were projected with considerable elongation so as to improve the accuracy of reading. With proper care in construction of the grid and lining up the projected pictures with the painted reference marks, vehicle positions at each 1-sec interval can be read to plus or minus 1-ft accuracy, as shown previously. Accuracy was checked for each location by means of a test vehicle using a fifth wheel and calibrated recorder.

Three positions of each vehicle were read, giving an average speed accurate to about plus or minus 1 mph. The time at which each vehicle passed an arbitrary zero point was calculated from the position and speed of each vehicle. The time-headway between each vehicle and the vehicle ahead was then obtained for the zero location. (see Fig. 1).

Four teams of students working in pairs recorded the vehicle positions. One man controlled the film advance using the remote control of a perceptoscope projector. The second student recorded the frame count and vehicle positions.

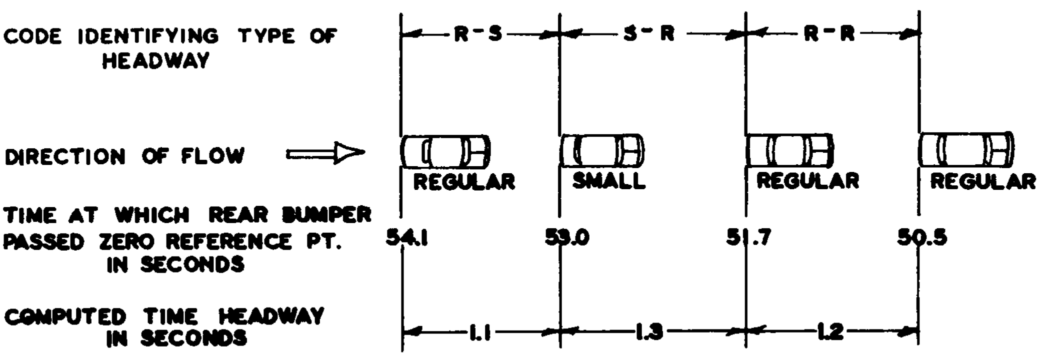


Figure 1. Time-headways within a four-car group.

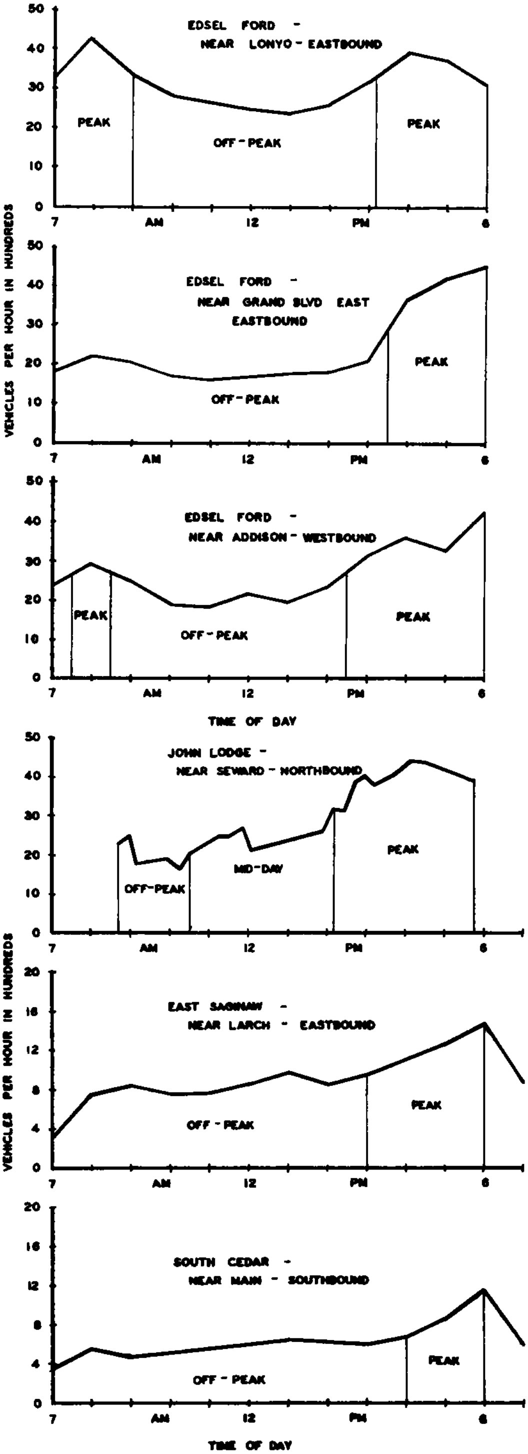


Figure 2. Traffic volumes by hours showing times of peak samples.

Continuous Samples

Vehicles in lane 1 (nearest the median) and lane 2 only were recorded, because the outermost lanes of the six- or eight-lane expressway at Detroit locations were more likely to involve weaving and other disturbances to traffic flow from outgoing or incoming vehicles.

At one location, one complete 5-min sample was read from each 15-min interval (the camera was ordinarily operated for 10 min of each 15, thus allowing a 5-min break for changing of film when necessary and making other checks of camera and timer operation). At other locations, two complete 5-min samples were recorded from peak hours and two from off-peak hours. For the two Lansing locations, four complete 5-min samples were recorded from peak-hour flow only. Figure 2 shows traffic volume curves and times of day from which peak and off-peak samples were selected.

The 5-min samples were used to establish a base distribution for later comparisons. Lanes 1 and 2 were read separately to avoid possible confusion in calling off position of cars in the two different lanes; i.e., after lane 1 was completed, lane 2 was read.

Four-Car Groups

In addition to the continuous samples, the complete film record for the Detroit locations and the complete record for the 3:00 to 6:00 period at the Lansing location were searched systematically for small and compact cars. For this purpose, the classification of the U.S. Department of Commerce Bureau of Public Roads for small and compact vehicles was used.

When a small or a compact vehicle appeared as the film was stepped along by means of the single-frame control, its position, the position of the cars immediately ahead and behind it and the position of a second standard vehicle ahead of it were recorded. This resulted in data from which speed, time-headway, and distance-headway were calculated between small and regular car ahead (S-R), regular following small (R-S), and between two standard-size cars (R-R) leading the small vehicle. A similar set of data resulted for each compact vehicle and the standard-size vehicles ahead and behind it.

A program was developed for a digital computer to carry out these routine calculations on the data recorded from the film. The correctness of the program was checked against some samples of data in which calculations were carried out by hand as a double-check.

ASSUMPTIONS AND HYPOTHESES

To obtain from the data answers to the questions posed in the problem statement, it was necessary to set up and test statistically certain hypotheses. Before this could be done, however, certain assumptions were necessary.

Assumptions

Necessary assumptions included the following:

1. Traffic flow and characteristics such as speed and headways between vehicles would be different under peak traffic conditions from that under off-peak lighter volume conditions.
2. Traffic flow characteristics would be generally similar within off-peak and within peak traffic periods (excluding any period during which actual stagnation and self-blocking of traffic was beginning to occur).
3. Driver reactions to vehicles ahead will affect speeds and time-headways when vehicles are operating close together under maximum volume conditions and/or in groups or "platoons."
4. Under heavy traffic volume conditions, at least, and to some extent in lighter volume conditions, the 85 percentile time-value determined from the distribution for a complete sample will furnish a division point beyond which vehicles may be considered as straggling between platoons. Such a method of determining platoons proved sufficiently valid for practical purposes in a previous study (2).

Hypotheses to Be Tested

The hypotheses required were the following:

1. A sample of four-car groups taken throughout the film wherever a small car or a compact car is found will give distributions generally similar to those from a continuous sample (but with equal numbers of each type of headway).
2. If small or compact vehicles use less or more than their share of the roadway, the average time-headway or distance-headway will be smaller or greater than that for a sample of standard-size vehicles taken at the same time and location as the small or compact car headways. Differences must be statistically significant (i.e., greater than chance expectancy) and also consistent in direction to be meaningful.
3. If visibility or other factors related to smaller vehicles present a hazard felt by drivers, then time and distance-headways used by those drivers should be longer as a result of their automatic and possibly unconscious reaction to such a feeling of hazard.

ANALYSIS OF DATA

The basic analysis was in terms of time-headways, because these represent an important index of driver response and involve both distance-headways and speed.

Continuous 5-Min Samples

Continuous 5-min samples were analyzed to show (a) range of time-headways; (b) expected proportion of vehicles at short and long (headway) ends of the distributions; and (c) whether small and compact vehicle-headways occurred at the two ends of the distribution consistently more or less frequently than expected.

For each continuous 5-min sample, time-headway distributions were tabulated by 0.2-sec intervals for regular cars (R-R) and each of the other combinations of small and compact cars (S-R, R-S, C-R, R-C). The time-headways between regular vehicles constituted the largest distribution because of the greater number of such cars in the stream. From each continuous distribution, the 85 percentile time-value was determined and also the 15 percentile time-value. Using these time-values, the expected number of small or compact cars in each end of the distribution was determined, using the headways between small and compact cars in these same distributions. A comparison was made by means of the χ^2 technique of the number of small car and compact car headways above the 85 percentile and below the 15 percentile time-value as compared to the expected number.

Four-Car Group Headways

The time-headways from the four-car group samples were analyzed to show (a) whether the samples so obtained differed from the continuous distributions significantly at either the short or long (headway) end of the distribution; and (b) whether, after elimination of stragglers, small and compact car average headways differed consistently from those between standard cars at the same time and place to a statistically significant extent.

From the four-car groups, time-headway distributions were tabulated by 0.2-sec intervals of R-R, S-R, and R-S headways. In each such distribution, the number falling below the 15 percentile and above the 85 percentile of the appropriate complete 5-min distribution of R-R headways was compared with the expected number based on continuous distribution for the same location and type of traffic flow. Differences were tested for greater than chance occurrence by the χ^2 technique.

Stragglers between platoons were eliminated by using as a cut-off point the 85 percentile time-headway of the appropriate continuous 5-min samples. Each of the four-car group samples was then scrutinized for all groups in which the S-R and R-S headways were both greater than the appropriate 85 percentile time-value. Where both were greater it was assumed that the small car was neither leading nor ending a platoon but was actually between platoons. Therefore, the two headways involving it and the accompanying R-R headway were all eliminated from the comparisons. Similarly, stragglers in the four-car groups involving compact cars were eliminated, because the drivers would not be judging position on the car ahead.

Next, each set of four-car headways for each location separately by peak and off-peak traffic periods was compared by Fisher's t-test to determine whether the average time-headways between S-R, R-S, and their corresponding R-R vehicle pairs showed statistically significant differences.

Time-Headways, Distance-Headways and Speed

For certain samples, time-headways, distance-headways and speeds were analyzed graphically to determine (a) relationships of each type of headway to speeds; and (b) whether time and distance-headways showed any markedly different relationships to each other or to speeds.

Time-headways and distance-headways for certain of the four-car groups were plotted against each other and the corresponding vehicle speed lines were drawn in on these plots. In this way, it was possible to see whether the two kinds of headway showed any noticeably different relationships and how both types of headways were related to vehicle speeds.

In addition, for several of the peak-hour, continuous 5-min samples, average values of speed, time-headway and distance-headway were computed for individual platoons. The time-headway and distance-headway values were then plotted by platoons, identifying those platoons in which small cars appeared. Thus, it was possible to see whether the inclusion of a small or compact car within a platoon resulted in any major difference in headway values or whether platoons containing such cars were in any way different from the others in the sample.

RESULTS

Continuous 5-Min Samples

The continuous 5-min samples gave a basis for evaluating the validity of other samples and of determining cut-off points for stragglers, as indicated under description of statistical analysis. Distributions were developed for three Detroit locations thrown together and for a fourth which differed from these. Other distributions were developed for the time-headways at the two locations in Lansing. Such distributions were developed both for peak-hour traffic and for off-peak traffic.

The most frequent time-headways under peak traffic conditions were consistently in the 1.0- to 1.35-sec range. In Lansing, where traffic was lighter, there was more straggling toward the long headways than in the heavier volume traffic in Detroit.

From these distributions, the percentage of small and compact cars in the samples were determined (Table 1). These proportions in Detroit ranged from 1.8 to 3.8 percent. At the Lansing locations during peak hours these proportions ranged from 0.6 to 5.0 percent. Off-peak traffic at these locations was relatively light, and this analysis was not carried out, but proportions were probably similar.

Small and Compact Cars in Ends of Distribution

From the continuous distributions, the expected number of small cars vs those actually occurring in the distribution was compared. The expected number below the 15 percentile and above the 85 percentile point was determined from the distribution of standard car headways. Table 2 shows that χ^2 analyses indicated no differences large enough to invalidate the hypothesis that the occurrence of the small and compact cars was similar to that of the standard car headways in the distribution.

TABLE 1

SUMMARY OF PROPORTION OF SMALL AND COMPACT CARS IN 5-MIN SAMPLES, LANES 1 AND 2 COMBINED

City	Street	All Cars	Peak Traffic: Compact No	%	Peak Traffic: Small No	%	All Cars	Off-Peak Traffic[a]: Compact No	%	Off-Peak Traffic[a]: Small No	%	All Cars	Total Traffic and Peak: Compact No	%	Total Traffic and Peak: Small No	%
Detroit	Seward	658	20	3.03	15	2.28	293	7	2.39	7	2.39	951	27	2.84	22	2.31
	Lonyo	425	10	2.4	16	3.8	329	9	2.7	7	2.1	754	19	2.5	23	3.1
	Grand Blvd	611	11	1.8	21	3.44	235	8	3.4	6	2.56	846	19	2.24	27	3.20
	Addison	525	13	2.5	11	2.1	216	7	3.2	4	1.9	741	20	2.7	15	2.0
	Total	2,219	54	2.43	63	2.84	1,073	31	2.89	24	2.24	3,292	85	2.58	87	2.64
Lansing	Cedar	498	11	2.2	3	0.6										
	Saginaw	564	16	2.8	28	5.0										
	Total	1,062	27	2.54	31	2.92										

[a]For Lansing, traffic was light, therefore, not analyzed

TABLE 2

SMALL AND COMPACT VEHICLES IN ENDS OF DISTRIBUTIONS OBSERVED VS EXPECTED IN 5-MIN SAMPLES

Street	Type of Distribution	Number	Obs	Exp	χ^2	Significant at $\alpha = 0.01$	Significant at $\alpha = 0.05$
Seward, Grand Blvd and Lonyo[1]	Below 15%.						
	C-R	41	8	6	0.78	No	No
	R-C	41	6	6	0	No	No
	S-R	48	11	7	2.67	No	No
	R-S	52	11	8	1.33	No	No
	Above 85%.						
	C-R	41	4	6	0.78	No	No
	R-C	41	10	6	3.12	No	No
	S-R	48	11	7	2.67	No	No
	R-S	52	11	8	1.33	No	No
Saginaw	Below 15%						
	C-R	16	1	2	0.57	No	No
	R-C	15	4	2	2.31	No	No
	S-R	28	3	4	0.29	No	No
	R-S	27	4	4	0	No	No
	Above 85%.						
	C-R	16	4	2	2.29	No	No
	R-C	15	1	2	0.58	No	No
	S-R	28	5	4	0.29	No	No
	R-S	27	7	4	2.64	No	No
Addison and S Cedar[2]							

[1]Individual location frequencies per cell too small for valid χ^2
[2]Frequencies per cell too small for valid χ^2

TABLE 3

SMALL AND COMPACT VEHICLES IN ENDS OF DISTRIBUTION—4-CAR GROUPS, NUMBER OF SMALLER VEHICLES OBSERVED IN 4-CAR SAMPLES VS NUMBER EXPECTED FROM 5-MIN SAMPLES[1]

Street	Period	Lane	Headway Type	15 Percentile N	E	0	χ^2	Signif Level	85 Percentile E	0	χ^2	Signif Level
Seward	Peak	1	R-R (all)	312	47	43	0.40		265	245	10.02	0.01
			R-R (small)	120	18	16	0 26		102	94	4 19	
			S-R	120	18	19	0 07		102	99	0 59	
			R-S	120	18	16	0.26		102	103	0.07	
			R-R(compact)	192	29	27	0 16		163	151	5 85	0 05
			C-R	192	29	33	0 65		163	152	4 91	
			R-C	192	29	28	0 04		163	151	5 85	0 05
	Peak	2	R-R (all)	313	47	23	14 42	0.01	266	266	0	
			R-R (small)	121	18	8	6 55	0 05	103	104	0 07	
			S-R	121	18	9	5 29	0.05	103	100	0 59	
			R-S	121	18	4	12 76	0.01	103	97	2 35	
			R-R (compact)	192	29	14	9 14	0 01	163	163	0	
			C-R	192	29	12	11 73	0 01	163	160	0 37	
			R-C	192	29	10	14.66	0 01	163	175	5 84	0 05
	Off-peak	1	R-R (all)	73	11	10	0 11		62	60	0 42	
			R-R (small)	26	4	3	0 30		22	22	0	
			S-R	26	4	4	0		22	22	0	
			R-S	26	4	4	0		22	23	0 30	
			R-R (compact)	47	7	6	0 17		40	38	0 67	
			C-R	47	7	7	0		40	43	1.51	
			R-C	47	7	8	0 17		40	42	0 67	
	Off-peak	2	R-R (all)	83	12	15	0 98		71	70	0 10	
			R-R (small)	33	5	6	0.24		28	26	0 94	
			S-R	33	5	4	0 24		28	29	0 24	
			R-S	33	5	7	0 94		28	28	0	
			R-R (compact)	50	8	9	0 15		42	44	0 60	
			C-R	50	8	8	0		42	44	0 60	
			R-C	50	8	6	0 60		42	46	2.38	
Lonyo	Peak	1	R-R (all)	308	46	41	1 49		262	284	12 35	0 01
			R-R (small)	153	23	17	1 84		130	145	11 53	0 01
			S-R	153	23	27	0 82		130	144	10 04	0.01
			R-S	153	23	18	1 38		130	137	2 51	
			R-R (compact)	155	23	24	0 05		132	139	2 50	
			C-R	155	23	20	0 46		132	140	3 27	
			R-C	155	23	20	0.46		132	135	0 46	
	Peak	2	R-R (all)	338	51	53	0 09		287	291	0 37	
			R-R (small)	176	26	28	0 180		150	153	0 406	
			S-R	176	26	33	2 21		150	153	0.41	
			R-S	176	26	25	0.05		150	154	0 72	
			R-R (compact)	162	24	25	0.048		138	138	0	
			C-R	162	24	31	2 40		138	133	1 22	
			R-C	162	24	26	0 20		138	136	0 20	
	Off-peak	1	R-R (all)	105	16	13	0 66		89	98	5 96	0 05
			R-R (small)	45	7	4	1.52		38	40	0 68	
			S-R	45	7	7	0		38	39	0 17	
			R-S	45	7	2	4.24		38	37	0.17	
			R-R (compact)	60	9	2	6.40	0 05	51	53	0.52	
			C-R	60	9	6	1 18		51	55	2 09	
			R-C	60	9	9	0		51	56	3 27	
	Off-peak	2	R-R (all)	169	25	38	7 93	0 01	144	154	4 69	
			R-R (small)	80	12	17	2 45		68	71	0 88	
			S-R	80	12	18	3 53		68	71	0 88	
			R-S	80	12	11	0.09		68	71	0 88	
			R-R (compact)	89	13	21	5 77	0 05	76	83	4 42	
			C-R	89	13	23	9 02	0.01	76	85	7 29	0.05
			R-C	89	13	12	0.09		76	79	0 81	
Grand Blvd	Peak	1	R-R (all)	216	32	25	1 80		184	207	19 41	0.01
			R-R (small)	121	18	15	0 59		103	115	9 40	0 01
			S-R	121	18	21	0 59		103	117	12 79	0 01

			R-S	121	18	22	1 05		103	115	9 40	0.01
			R-R (compact)	95	14	10	1 34		81	92	10.13	0 01
			C-R	95	14	11	0 75		81	92	10 13	0 01
			R-C	95	14	16	0 34		81	92	10 13	0 01
	Peak	2	R-R (all)	185	28	50	20 36	0 01	157	162	1 05	
			R-R (small)	91	14	23	6 84	0 05	77	82	2 11	
			S-R	91	14	24	8 44	0 01	77	74	0 76	
			R-S	91	14	27	14 26	0 01	77	79	0 34	
			R-R (compact)	94	14	27	14 88	0 01	80	80	0 00	
			C-R	94	14	22	5.37	0.05	80	81	0 03	
			R-C	94	14	20	3 02		80	75	2 10	
	Off-peak	1	R-R (all)	219	33	36	0 32		186	201	8 03	0 01
			R-R (small)	120	18	19	0.07		102	109	3 20	
			S-R	120	18	31	11.05	0 01	102	112	6 54	0 05
			R-S	120	18	21	0 59		102	108	2.35	
			R-R (compact)	99	15	17	0 32		84	92	3 32	
			C-R	99	15	22	3.85		84	92	5 03	0.05
			R-C	99	15	12	0.71		84	89	1 97	
	Off-peak	2	R-R (all)	221	33	22	0 04		188	208	14.25	0.01
			R-R (small)	132	20	20	0 00		112	129	17 03	0.01
			S-R	132	20	19	0 06		112	115	0 53	
			R-S	132	20	23	0.53		112	121	4 77	
			R-R (compact)	89	13	12	0 09		76	79	0 80	
			C-R	89	13	20	4 41		76	76	0.00	
			R-C	89	13	20	4 41		76	82	3 24	
Addison	Peak	1	R-R (all)	308	46	53	1.25		262	251	3 09	
			R-R (small)	157	24	26	0 20		133	127	1 77	
			S-R	157	24	23	0 05		133	131	0.20	
			R-S	157	24	26	0.20		133	126	2 41	
			R-R (compact)	151	23	27	0 82		128	124	0.82	
			C-R	151	23	33	5 13	05	128	120	3 28	
			R-C	151	23	16	2 15		128	126	0.20	
	Peak	2	R-R (all)	176	26	22	0 72		150	147	0.42	
			R-R (small)	98	15	14	0 08		83	82	0 08	
			S-R	98	15	9	2 83		83	75	5 04	0 05
			R-S	98	15	15	0		83	82	0 08	
			R-R (compact)	78	12	8	1 57		66	65	0 10	
			C-R	78	12	6	3 54		66	63	0 89	
			R-C	78	12	6	3 54		66	62	1 57	
	Off-peak	1	R-R (all)	86	13	9	1 45		73	74	0 09	
			R-R (small)	32	5	5	0.00		27	28	0 24	
			S-R	32	5	4	0.24		27	29	0 95	
			R-S	32	5	2	2.13		27	31	3 79	
			R-R (compact)	54	8	4	2 35		46	46	0 00	
			C-R	54	8	5	1 32		46	50	2.35	
			R-C	54	8	8	0		46	53	7 18	0 05
	Off-peak	2	R-R (all)	71	11	8	0 97		60	65	2 69	
			R-R (small)	41	6	6	0 00		35	36	0 20	
			S-R	41	6	2	3.13		35	39	3 13	
			R-S	41	6	3	1 76		35	37	0 77	
			R-R (compact)	30	4	2	1 15		26	29	2 60	
			C-R	30	4	4	0.00		26	26	0.00	
			R-C	30	4	2	1 15		26	25	0.29	
Cedar	Peak	1	R-R (all)	63	9	11	0.519	0 05	54	62	8.28	0 01
			R-R (small)	23	3	2	0.383		20	22	1 83	
			S-R	23	3	2	0.383		20	21	0 383	
			R-S	23	3	1	1 53		20	18	1.53	
			R-R (compact)	40	6	9	1 315		34	40	7 06	0 05
			C-R	40	6	7	0.22		34	36	0 86	
			R-C	40	6	7	0 22		34	34	0	
	Peak	2	R-R (all)	75	11	11	0		64	71	5.22	0.05
			R-R (small)	32	5	3	0.95		27	30	2.10	
			S-R	32	5	5	0		27	28	0.24	
			R-S	32	5	3	0.95		27	26	0.24	
			R-R (compact)	43	6	8	0.77		37	41	3.10	
			C-R	43	6	5	0.19		37	36	0.19	
			R-C	43	6	4	0.77		37	30	9 50	0.01

[1] No off-peak pictures analyzed.

Small and Compact Vehicles in End of Distribution in 4-Car Groups

To test roughly whether the 4-car groups produced a total sample similar to the continuous samples, the number of time-headways falling above and below the 15 percentile time value determined from the continuous samples was submitted to χ^2 comparison. Table 3 shows that when these comparisons were made by lane for each of the locations a few significant differences were found in nine sets of comparisons, but these were not consistent within each lane, type of headway, and location nor between locations. The remainder were not statistically significant.

This inconsistency and lack of statistical significance leads to the conclusion that there were some variations between the distributions derived from the 4-car samples and the continuous distributions but that they were not consistent nor great enough to invalidate use of the samples derived from the 4-car groups. Therefore, it seemed that no great violence would be done by using these samples and by using a cut-off point for stragglers determined by means of the 85 percentile time-value from the continuous distributions.

Comparison of Averages for Headway Types

After eliminating stragglers, as described earlier, comparison of averages for time-headways at each location by lane for each type of vehicle combination was carried out by means of Fisher's t-function. A summary of the results is given in Table 4. These results indicate that although some of the differences in mean time-headways were on the order of 10 to 15 percent, others were negligible, and very few were significant at the 0.01 probability level. The values for lanes 1 and 2 were then combined in the summary table which shows that in the combined comparisons one was significant at the 0.01 level and three between this and the 0.05 level. Furthermore, the direction of the differences, as shown by plus and minus signs, was not consistent in the different samples. Therefore, time-headway differences between different types of vehicles were not great enough nor consistent enough in direction to indicate a true difference (greater than might be obtained by chance).

Over-All Time-Headways

To summarize concisely and show how similar the peak-hour time-headway values were, over-all averages were computed as shown in Table 5 for peak-hour, 4-car group samples (stragglers eliminated). The number of headways from which each average was determined is shown in the R$\rightarrow$R columns. Averages for small car headways and headways between standard cars taken immediately following the small car headways are shown on the left side part of the table. A similar set of time-headway averages is shown for compact cars and the comparison standard car headways for each of the compact cars on the right side of the table. Here again, the direction of differences varies in the different samples but the average value ranges from 1.67 to 2.22 sec in the peak-hour traffic in Detroit. The peak-hour traffic at the Lansing locations showed greater variability, which would be expected from the more dispersed characteristic and the lighter volume at these locations.

Off-Peak Time-Headways

Average time-headways for the off-peak traffic have not been given in the over-all average table nor in the preceding comparison of averages. These average time-headways were more variable and differences between types showed less consistency and statistical significance than the peak-hour headways. Therefore, no statistically significant differences between headways involving standard and smaller vehicles were shown in off-peak traffic.

Time, Distance and Speed by Platoons in Continuous Samples

To investigate relationships between time-headway and distance-headway in relation to platoon speeds, average values were calculated and plotted.

TABLE 4

COMPARISON OF AVERAGES FOR HEADWAY TYPES t - VALUES, FOUR-CAR GROUPS, PEAK TRAFFIC, LANES 1 AND 2 COMBINED

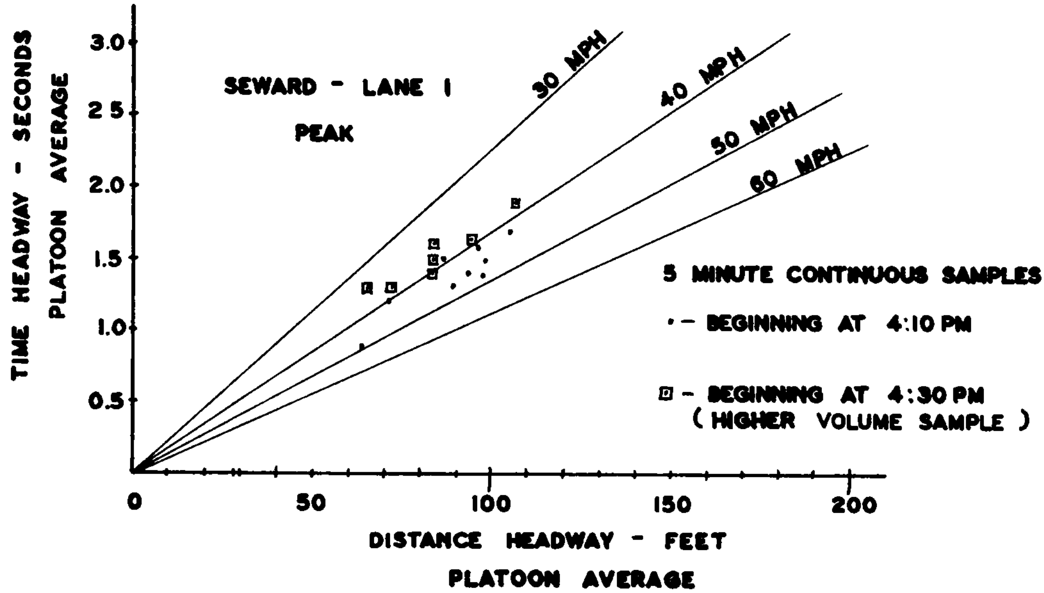
Street	Headway Type	$\bar{X}_1$	$\bar{N}_2$	X_2	N_2	$\bar{X}_1-\bar{X}_2$	t	Signif. Level
Seward	(S-R) vs (R-S)	1.774	232	1.794	232	-0.020	-0.186	
	(S-R) vs (R-R)	1.774	232	1.894	232	-0.120	-1.063	
	(R-S) vs (R-R)	1.794	232	1.894	232	-0.100	-0.906	
	(C-R) vs (R-C)	1.850	372	1.690	372	+0.160	+1.824	0.05
	(C-R) vs (R-R)	1.850	372	1.964	372	-0.114	-1.074	
	(R-C) vs (R-R)	1.690	372	1.964	372	-0.274	-2.881	0.01
Gr. Blvd.	(S-R) vs (R-S)	1.791	206	1.676	206	+0.115	+1.105	
	(S-R) vs (R-R)	1.791	206	1.727	206	+0.064	+0.629	
	(R-S) vs (R-R)	1.676	206	1.727	206	-0.051	-0.578	
	(C-R) vs (R-C)	1.909	187	1.752	187	+0.157	+1.375	
	(C-R) vs (R-R)	1.909	187	1.672	187	+0.237	+1.841	0.05
	(R-C) vs (R-R)	1.752	187	1.672	187	+0.080	+0.672	
Lonyo	(S-R) vs (R-S)	1.972	325	2.168	325	-0.196	-1.530	
	(S-R) vs (R-R)	1.972	325	2.050	325	-0.078	-0.629	
	(R-S) vs (R-R)	2.168	325	2.050	325	+0.118	+0.866	
	(C-R) vs (R-C)	2.092	310	2.158	310	-0.066	-0.523	
	(C-R) vs (R-R)	2.092	310	2.220	310	-0.128	-0.853	
	(R-C) vs (R-R)	2.158	310	2.220	310	-0.062	-0.407	
Seward, Gr. Blvd. and Lonyo	(S-R) vs (R-S)	1.863	763	1.921	763	-0.058	-0.826	
	(S-R) vs (R-R)	1.863	763	1.915	763	-0.052	-0.753	
	(R-S) vs (R-R)	1.921	763	1.915	763	+0.006	+0.084	
	(C-R) vs (R-C)	1.949	869	1.870	869	+0.079	+1.229	
	(C-R) vs (R-R)	1.949	869	1.993	869	-0.044	-0.577	
	(R-C) vs (R-R)	1.870	869	1.993	869	-0.123	-1.667	0.05
Addison	(S-R) vs (R-S)	2.226	237	2.107	237	+0.119	+0.712	
	(S-R) vs (R-R)	2.226	237	2.311	237	-0.085	-0.376	
	(R-S) vs (R-R)	2.107	237	2.311	237	-0.204	-0.928	
	(C-R) vs (R-C)	2.169	215	2.346	215	-0.177	-0.535	
	(C-R) vs (R-R)	2.169	215	2.213	215	-0.044	-0.168	
	(R-C) vs (R-R)	2.346	215	2.213	215	+0.133	+0.440	
S. Cedar	(S-R) vs (R-S)	5.109	54	5.380	54	-0.271	-0.228	
	(S-R) vs (R-R)	5.109	54	3.728	54	+1.381	+1.190	
	(R-S) vs (R-R)	5.380	54	3.728	54	+1.652	+1.524	
	(C-R) vs (R-C)	3.672	79	5.867	79	-2.195	-1.985	0.025
	(C-R) vs (R-R)	3.672	79	4.333	79	-0.661	-0.846	
	(R-C) vs (R-R)	5.867	79	4.333	79	+1.534	+1.329	

TABLE 5

SUMMARY OF OVER-ALL AVERAGE TIME-HEADWAYS, SMALL, COMPACT AND REGULAR CAR HEADWAYS, FOUR-CAR GROUPS, PEAK TRAFFIC, LANES 1 AND 2, TOTAL OBSERVATIONS

City	Street	Time-Headway (sec)			No. of Observ.	Time-Headway (sec)			No. of Observ.
		R→R	R→S	S→R		R→R	R→C	C→R	
Detroit	Seward	1.894	1.794	1.774	232	1.964	1.690	1.850	372
	Grand Blvd. E	1.727	1.676	1.791	206	1.672	1.752	1.909	187
	Lonyo	2.050	2.168	1.972	325	2.220	2.158	2.092	310
	Above 3 locations	1.915	1.921	1.863	763	1.993	1.870	1.949	869
	Addison	2.311	2.107	2.226	237	2.213	2.346	2.169	215
Lansing	S. Cedar	3.728	5.380	5.109	54	4.333	5.867	3.672	79
	Saginaw[a]								

[a]No 4-car group sample.



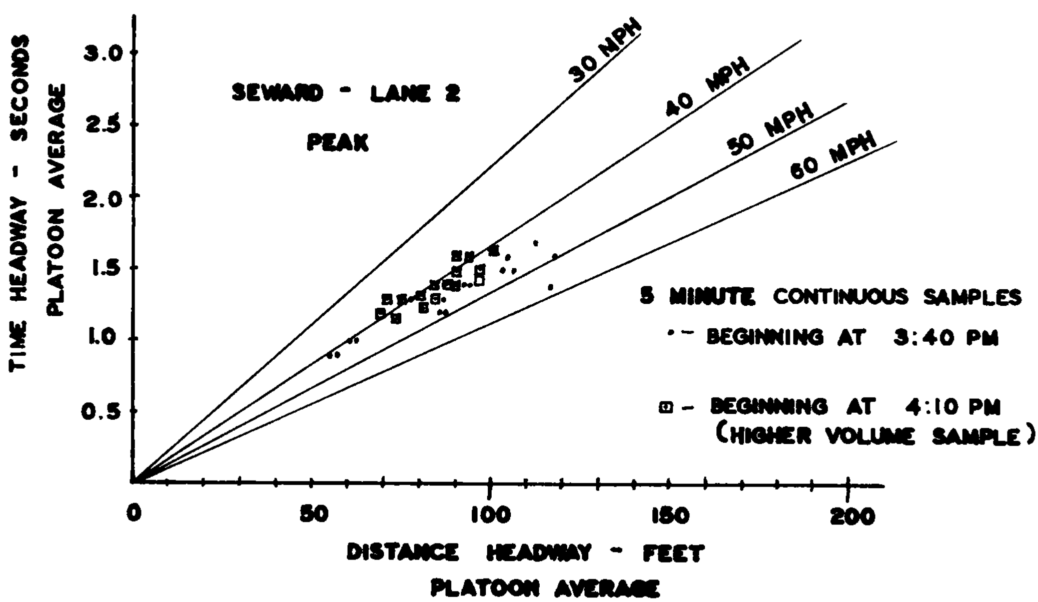
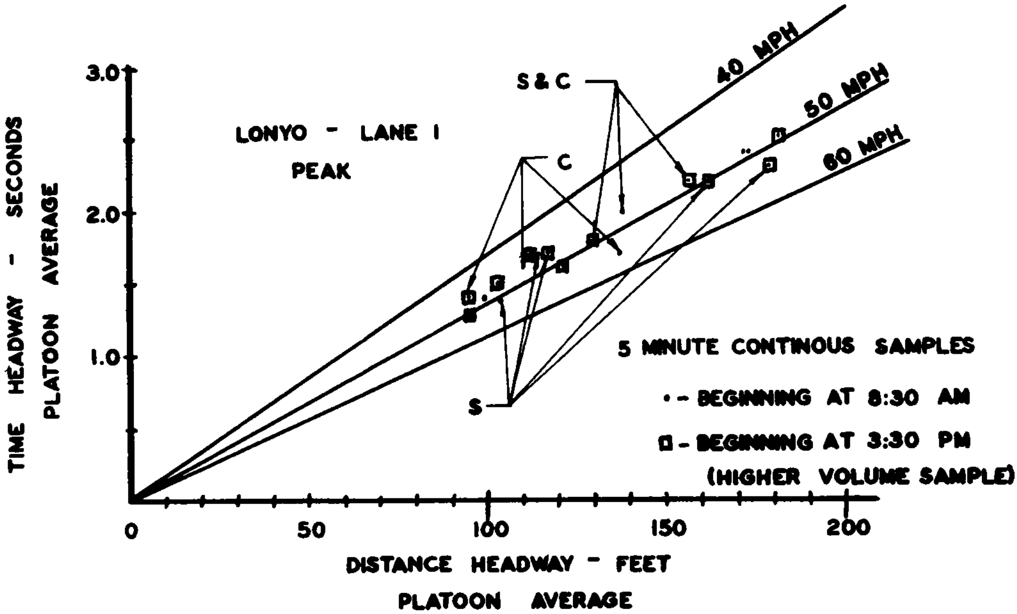


Figure 3. Average headways and speeds by platoons—John Lodge Expressway at Seward (outbound).



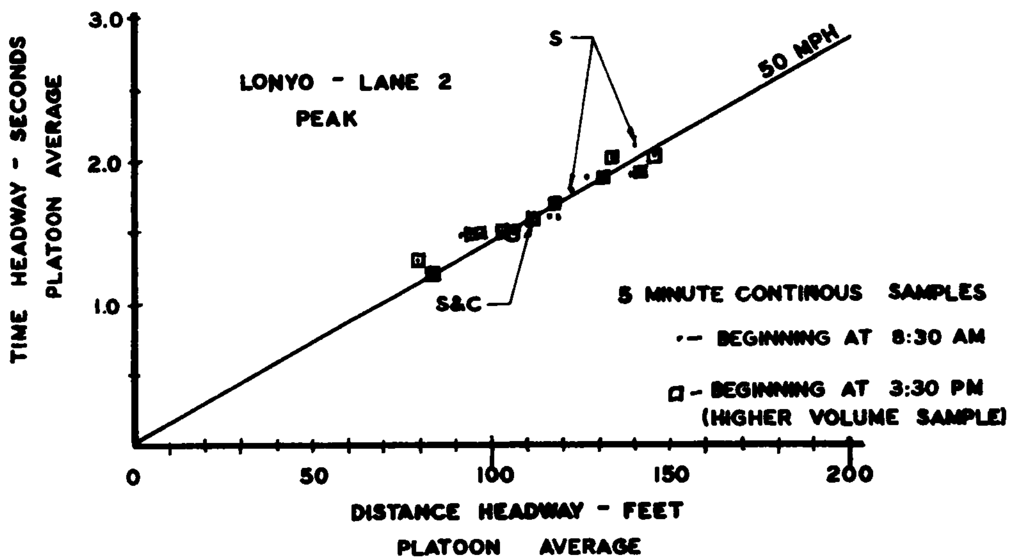
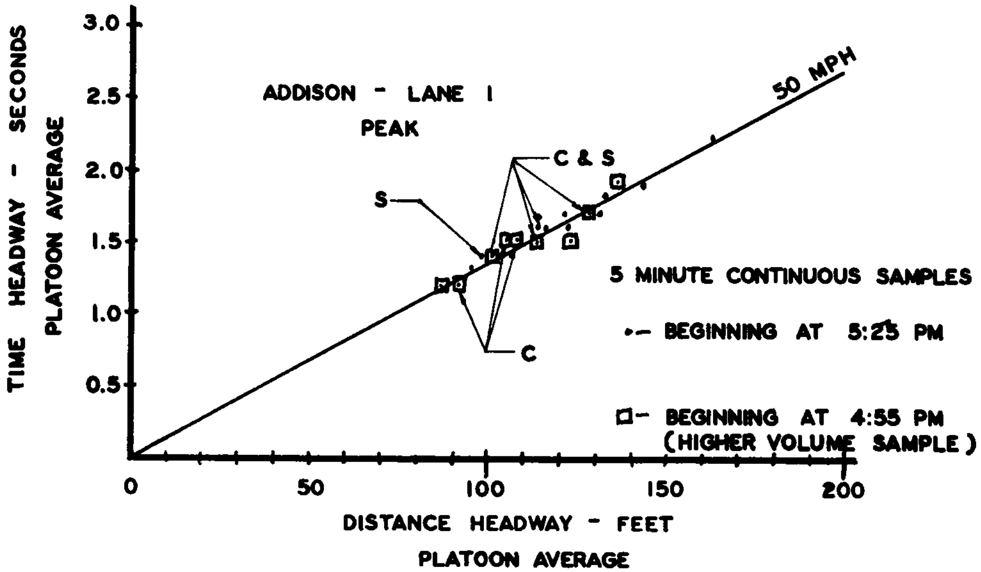


Figure 4. Average headways and speed by platoons—Edsel Ford Expressway at Lonyo (inbound), 5-min continuous samples.



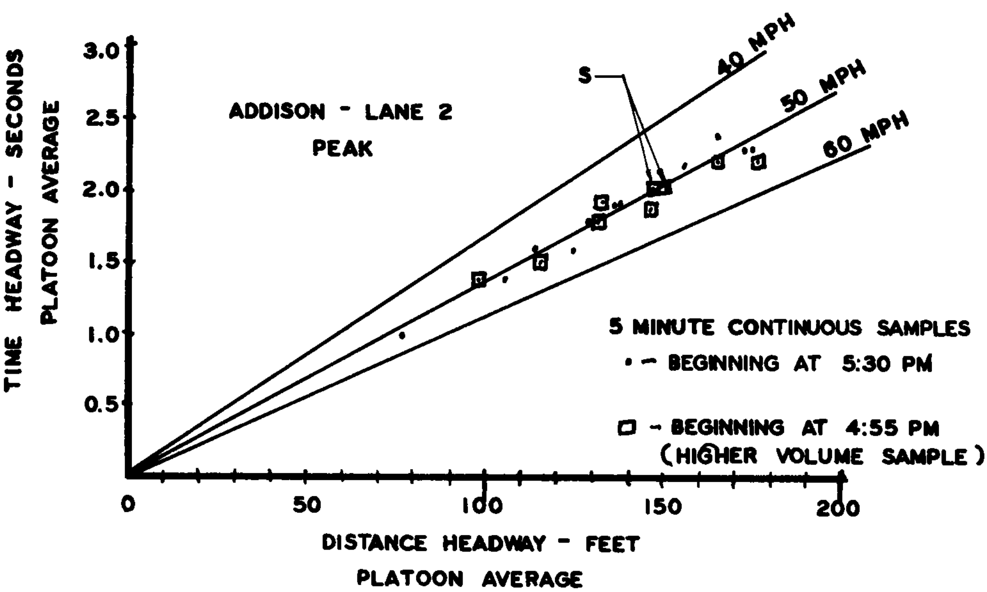


Figure 5. Average headways and speed by platoons—Edsel Ford Expressway at Addison (outbound), 5-min continuous samples.

Figures 3, 4, and 5 show plots of time-headways vs distance-headways by platoons in two 5-min samples at three Detroit locations. The lines radiating from the origin in this plot represent 40-, 50-, and 60-mph speeds and platoons including small and compact cars have been identified.

A linear relationship between average distance-headway and average time-headway for the platoons is shown at two of the locations. The plotted points cluster closely around the 50-mph speed line. At the third location (Seward) more slowing occurred, as shown by the tendency for plotted points to range between the 50- and the 30-mph line. However, there was very little indication of a curvilinear relationship, except possibly in the plot for the Seward Avenue bridge from which northbound large expressway traffic was recorded.

Time, Distance and Speed Analysis for Cars in 4-Car Groups

To determine briefly in graphic fashion the relationship between individual time-headways, distance-headways, and speed of the vehicles, two types of headways for the three combinations of vehicles (R-R, R-S, and S-R) were tabulated. Figure 6 shows the plot of average ordinates (time-headways) for given distance-headway values. The lines radiating from the ordinate again show speeds of 40, 50, and 60 mph, and the headways between the different types of vehicles are indicated by the three different symbols. It is clear that, for this location at least, there was a wide range of headways with only slight tendency toward curvilinearity representing a shorter time-headway relative to distance-headway as speeds increased; and the relationship was very similar for each type of headway.

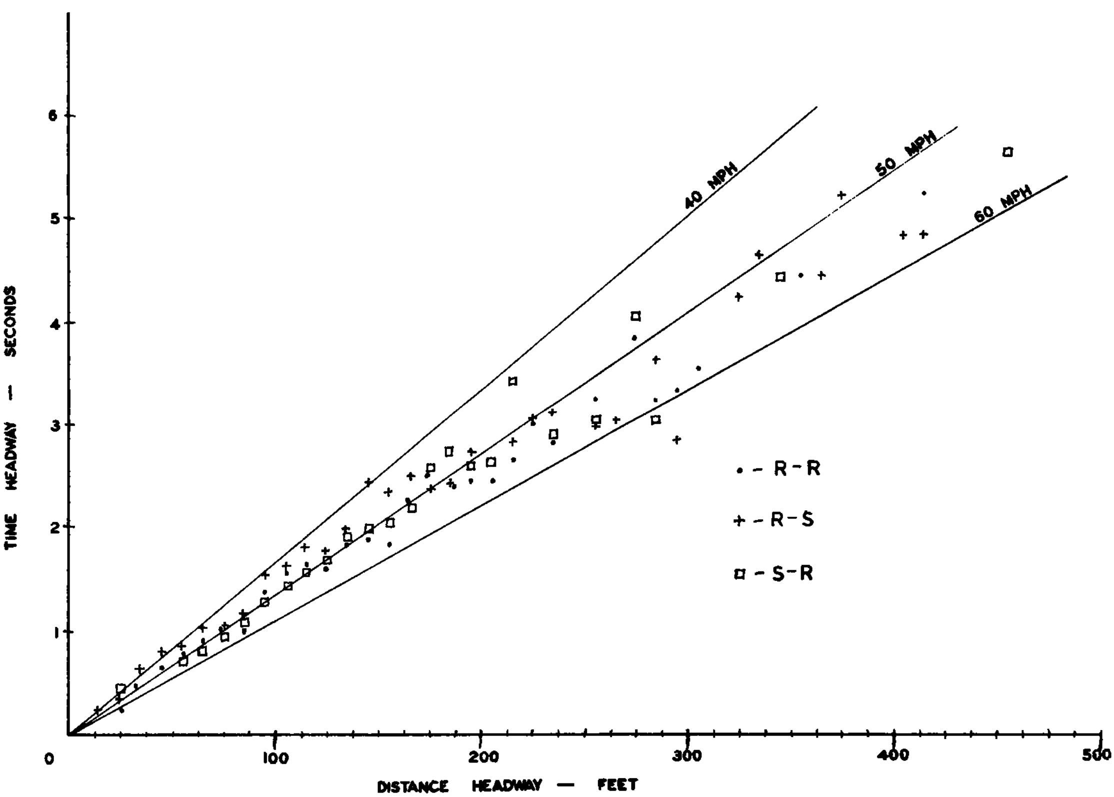


Figure 6. Time-headways, Distance-headways and speed in 4-car groups; average time-headways for 10-ft units of distance-headways, peak traffic, Ford at Addison.

Comment

The lack of significant differences or of consistent direction of differences indicates that small and standard car drivers operate their vehicles pretty much alike in heavy traffic. The greater variability found in lighter traffic and in the more dispersed traffic in Lansing is to be expected.

The very slight tendency toward a curvilinear relationship between time-headways and distance-headways of platoons at lower speeds may be of significance. However, it was not definite in the samples plotted. Facilities did not allow further analysis of samples which included marked slowing.

The definite linearity between time and distance headways for platoons and for individual cases in 4-car groups is of special interest when it is recalled that the most frequent time-headways cluster between 1.0 to 1.35 sec. It is suggested that time-headways may be more fundamental than distance-headways.

CONCLUSIONS

The following conclusions may be drawn from these samples of small and compact cars in relation to continuous samples in Detroit and Lansing.

1. The proportion of small and compact cars ranged from 1.8 to 3.2 percent in 5-min samples during peak hours in Detroit and 0.6 to 5.0 percent at one of the Lansing locations.
2. Distributions of time-headways obtained from 4-car groups did not differ markedly or consistently from continuous distributions taken at the same locations. Samples from 4-car groups could therefore be used for investigating the problem.
3. When stragglers between platoons were eliminated (drivers who would not be making judgments on other cars), the differences in average time-headways involving standard cars only and involving small and compact cars were not consistent in direction nor statistically significant.
4. Some of the differences were as large as 10 or 15 percent. Because average time-headways are the inverse of volume, this might mean a considerable effect on traffic capacity. However, these differences were not greater than could be expected by chance. These comparisons, therefore, indicate that small and compact cars added to standard cars in traffic flow when freeways were approaching capacity did not affect the capacity of the highway.
5. Platoon average time-headways and distance-headways and speeds indicated no consistent effect where platoons include small or compact cars. Also, relationships between the two types of headway were linear and followed closely around the 50-mph line in two samples analyzed. There was some possible curvilinearity at low and at highest speeds but the data analyzed were insufficient to determine a trend.
6. The individual car headways in the sample analyzed showed essentially the same relationships for R-R, R-S, and S-R headways.
7. Because average time-headways are the inverse of volume in vehicles per hour, and because other studies have reported a curvilinear relationship between volume and density, it appears that the present type of analysis (Figs. 3 to 6) may have certain advantages in throwing light on the characteristics of traffic flow.

   Although the relationships shown are essentially linear, it may be that more curvilinearity would be shown if samples had been analyzed just preceding the occurrence of traffic stoppages. It was not possible to do that in this particular project. Platoon averages, however, showed internal densities on the order of 80 to 100 cars per mile, ranging downward to 50 to 60 cars per mile. The latter figure was found for 1-min averages in a previous study (3).
8. Time-headways may be more fundamental than distance-headways. Time-headways and distance-headways can be measured for individual cars whereas traffic volumes and densities must involve a large number of vehicles or a projected estimate from a number of vehicles. Time-headways involve the second (following) driver's combination judgment of distance and speed under conditions of the traffic stream. It may well be of significance that average time-headways in Figures 3 to 5 were more

similar than distance-headways and that a linear relationship resulted in the close spacing end of Figure 6.

ACKNOWLEDGMENTS

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Small-Car Speeds and Spacings On Urban Expressways

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•THE Traffic Operations Research Division of the Bureau of Public Roads undertook studies in April and May 1960 to provide general data on small-car speeds and their effect on highway capacity. The studies were limited in scope and were intended as a pilot study to aid in determining the need for, and the areas of, further investigations of small-car behavior.

The objectives parallel those of the Michigan study, however, the methods of obtaining and analyzing the data differ. Four locations were selected in the Washington metropolitan area on four-lane divided facilities of expressway-type design. Three locations were on near-level tangent sections, the fourth location was on a grade. The character and composition of traffic is different for each of the locations. Data for these locations were obtained using the Bureau's traffic analyzer (1) to obtain spot speeds and time spacings. The traffic analyzer automatically records on adding machine tape the speed and precise time of day each vehicle passes the point of observation. Vehicle speed is recorded as travel time in hundredths of a second over the known distance between speed detectors and the time of day is continuously measured in increments of 0.0001 hour. Vehicle classification is observed manually and the vehicle code entered on the adding machine to be printed with the automatic recording.

Table 1 gives the volume range and composition of traffic at the study locations. The total sample obtained for each of three locations was 7,000, 6,500, and 3,200 vehicles, respectively. The combined percentage of foreign and compact cars in the traffic stream was appreciably higher than for the Michigan studies—12 percent as compared to 5 percent.

Study location 1 was on the Washington-Baltimore Parkway at the District of Columbia line. This is a primary inter-urban route which also serves commuter traffic during the peak periods. In this area the posted speed limit is 45 mph and because this is a National parkway, truck traffic is prohibited. The parkway was completed about 1955.

Study location 2 was on the Shirley Highway between Ridge and Glebe Roads in Arlington County, Va. This is a primary through route serving commuter traffic during the peak periods. In the study area the posted speed limit is 50 mph for passenger cars and 45 mph for trucks. This facility was built during World War II and is now designated as an Interstate route.

Study location 3 was on the Mt. Vernon Memorial Highway between the entrances to Washington National Airport. The parkway is a major arterial link between Washington, D.C., and Alexandria, Va., and carries commuter traffic during the peak periods. The posted speed limit at the study location is 40 mph, and because this is a National parkway, truck traffic is prohibited. The parkway was constructed prior to 1940.

A fourth location was selected on the Suitland Parkway, east of Alabama Avenue, to determine the effect of grade. The study location was on a 5 percent grade approximately 1,500 ft in length. Stations were set up at the bottom of the grade, at the beginning of the vertical curve near the crest of the grade, and at a point midway between these two. Stations were operated in pairs for 12 min of a 15-min period and alternated to provide study of all stations equally during the study period. It was necessary to alternate between pairs of stations, due to the fact that the traffic analyzer only has capacity to handle a maximum of four lanes at a time. This parkway serves as an arterial between Washington and government installations in Prince Georges County and also serves

TABLE 1

VOLUME RANGE AND COMPOSITION TRAFFIC[1]

Location	Traffic Flow	Date of Test (1960)	Lane	Vol Range[2]	Traffic Total (no)	Standard (%)	Compact (%)	Foreign (%)	Small Cars Combined (%)
Wash -Balt Pkwy	North-bound	Apr 25	1	600-1,550	3,831	90 3	4.2	4 6	8 8
			2	340-1,510	3,206	88 5	4 8	5 0	9 8
			1 + 2	940-2,990	7,037	89 5	4 4	4 8	9 2
Shirley Hwg	South-bound	Apr 26	1	620-1,700	3,038	75 6	5 3	8 1	13.4
			2	460-2,020	3,458	82 3	4 9	8.7	13 6
			1 + 2	1,090-3,560	6,496	79 2	5.1	8 4	13 5
Mt Vernon Mem. Pkwy	South-bound	May 4	1	270-820	1,688	81 5	5 1	10 2	15.3
			2	120-1,030	1,544	82 8	6 2	9.3	15.5
			1 + 2	440-1,780	3,232	82 1	5 6	9 7	15 3

[1]Percentages shown do not total 100 percent Difference, not shown, consist of other vehicle types such as trucks and motorcycles
[2]Based on 6-min counts

TABLE 2

AVERAGE SPEED BY TYPE OF VEHICLE

Location	Traffic Flow	Time Period	Volume per Hour Lane 1	Volume per Hour Lane 2	Average Speed (mph) Lane 1 Std	Lane 1 Compact	Lane 1 Foreign	Lane 2 Std	Lane 2 Compact	Lane 2 Foreign
Wash -Balt Pkwy	Northbound	2 42 - 4 06	731	474	43 6	44 9	44 8	51 0	50 6	52 3
		4 06 - 5 54	1,310	1,188	42 6	42 2	42 3	49 2	49 0	49 5
		5 54 - 6 30	968	872	43 5	45 3	44 6	50 1	51.7	50 0
		2 42 - 6 30	815	682	43 0	43 4	43 4	49 7	49 4	50 0
		2 42 - 6 30[a]	1,497		46 1	46 3	46 5			
Shirley Hwy	Southbound	2 30 - 3 30	725	605	44 1	42 7	45 3	52 4	52 4	51 1
		3 30 - 4 30	1,049	1,314	41 0	41 6	42.2	47 9	48.1	48 7
		4 30 - 5 00	1,594	1,908	35 1	35 2	34 6	37 7	36 2	36 2
		5 00 - 5 18	1,557	1,950	28 6	31 5	31 4	29 8	32 2	32 1
		2 30 - 5 18	1,085	1,235	38 3	38 8	38 4	43 4	41 6	41 2
		2 30 - 5 18[a]	2,320		41 0	40 2	40 0			
Mt Vernon Mem Pkwy	Southbound	2 30 - 3 30	352	203	40 3	40 0	42.0	45.4	47 3	47 1
		3 30 - 4 30	493	433	40 4	39.9	40 3	44 7	46 8	43.6
		5 00 - 6 12	703	757	40 7	39 5	41 1	45.6	45 3	45 0
		2 30 - 6 12	528	483	40 5	39.6	41 0	45 3	46 0	44 9
		2 30 - 6 12[a]	1,010		42 8	43.0	42 8			

[a]Combined average for both lanes, for the entire study period

TABLE 3

AVERAGE SPEED BY TYPE OF VEHICLE AND STATION, SUITLAND PARKWAY, WESTBOUND (5 PERCENT GRADE)

Lane	Time Period	Average Speed (mph) Station 1 Std.	Station 1 Compact	Station 1 Foreign	Station 2 Std	Station 2 Compact	Station 2 Foreign	Station 3 Std.	Station 3 Compact	Station 3 Foreign
1	7 45 - 9 00	44.9	45.8	44.4	42.4	43.6	38.6	41.2	41.8	40.0
	9·00 - 10:00	43.6	43.4	46.0	42.8	39.0	40.4	42.0	44.1	40.9
	10 00 - 11·00	44.4	45.4	40.6	42.2	41.4	43.6	41.0	42.1	35.6
	7 45 - 11:00	44.4	44.8	43.7	42.4	41.8	40.3	41.4	42.4	39.6
2	7 45 - 9.00	48.8	49 2	46 4	47 2	47 6	45 2	47.6	46.1	48.2
	9 00 - 10·00	47 6	42 6	44.8	45 7	47.1	-	45.9	44.3	46 4
	10 00 - 11 00	43.6	43 8	44.8	45 8	44.8	46.8	45.3	46.1	49.5
	7 45 - 11·00	48.0	46.2	45.8	46 7	47 2	45.8	46.7	45.3	48 2
1 + 2	7:45 - 9·00	46.4	47.5	44.9	44 2	45.1	41.2	43.6	42 5	41.4
	9.00 - 10·00	44.5	43.0	45 8	43.5	42.0	40.4	43.0	44.2	42.3
	10 00 - 11·00	44.3	45.1	41.3	43.6	41 6	45.0	41.7	42.6	42.6
	7 45 - 11 00	45.4	45.3	44 1	43.6	43.4	42 4	42.9	43 0	41.8

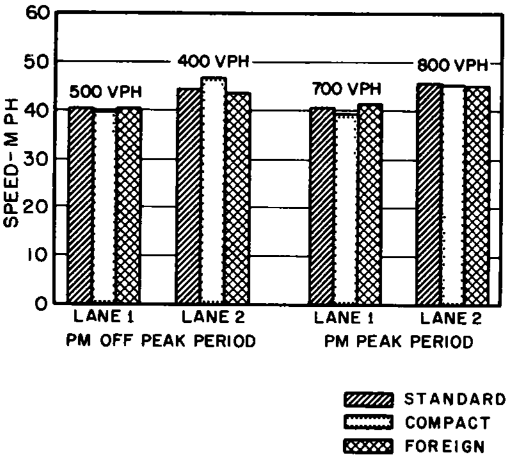

Figure 1. Average speed by type of vehicle and traffic condition, site 1, Mount Vernon Memorial Parkway southbound.

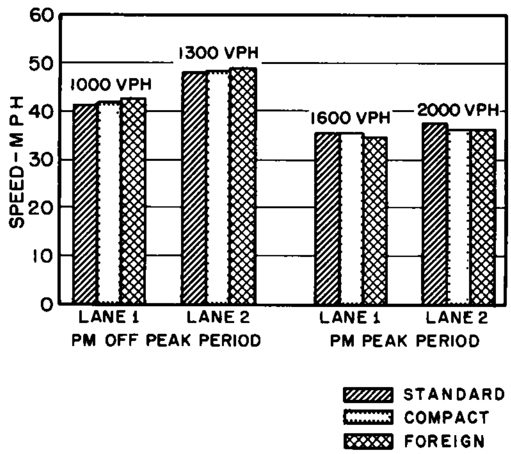

Figure 2. Average speed by type of vehicle and traffic condition, site 2, Shirley Highway southbound.

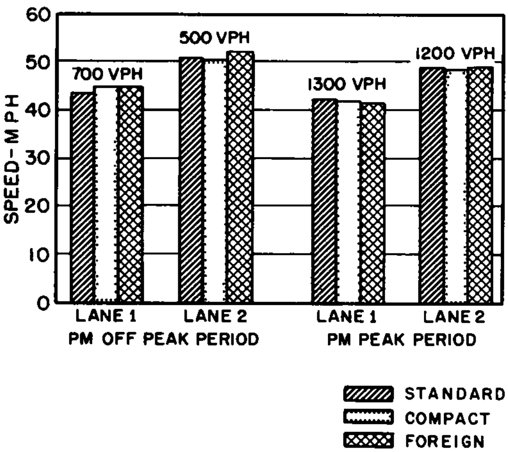

Figure 3. Average speed by type of vehicle and traffic condition, site 3, Washington-Baltimore Parkway northbound.

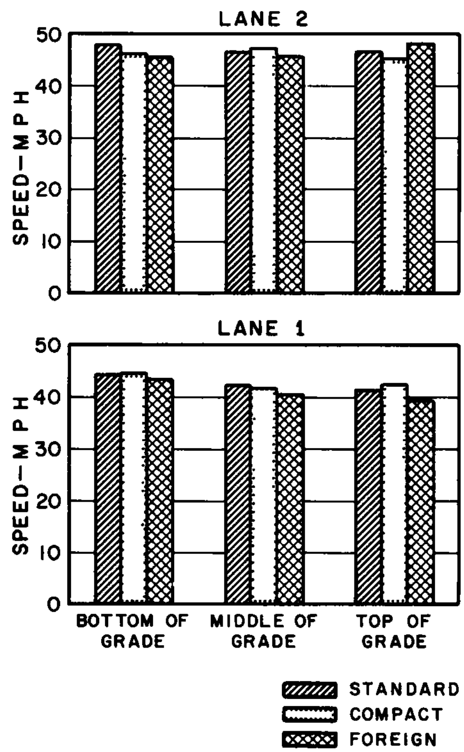

Figure 4. Average speed by type of vehicle and study location, site 4, Suitland Parkway westbound (5 percent grade).

commuter traffic during peak periods. The posted speed limit is 45 mph and truck traffic is prohibited.

Table 2 gives the average speeds obtained at the tangent locations by vehicle type, time period, volume rate, and lane. Figures 1, 2, and 3 show a comparison of the average speeds by vehicle type for the three locations with insignificant grades. These are based on a sample of approximately 40 percent of the standard cars and all compact and foreign cars. Lane 1 is the outside or curb lane, lane 2 is the median lane.

TABLE 4

TIME-SPACING VALUES DETERMINED BY THREE METHODS FOR STANDARD AMERICAN, COMPACT AND FOREIGN PASSENGER CARS

Lane	Location	Date of Test (1960)	Time Period (PM)	Volume (vph)	Avg Speed (mph)	Spacing (sec) 1st Method (BPR) Std	Cmpct	Forgn	2nd Method Std	Cmpct	Forgn	3rd Method Std	Cmpct	Forgn	Uniform Spacing This Volume
1	Wash -Balt Pkwy	Apr 25	2 42 - 4 06	731	44 0	4 7	5 7	4 0	6 4	6.4	5 4	8 2	8 8	7 5	5 0
			4.06 - 5 54	1,310	42 5	2 7	2 8	2.5	4 9	5 2	4 4	5 1	5 7	4 9	2 7
			5 54 - 6 30	968	43 8	3.7	3 1	3 7	5 8	6 0	5 6	7 3	7 3	7 3	3 7
	Shirley Hwy	Apr 26	2 30 - 3 30	725	43 8	4 0	5 4	5 5	7 1	5 8	7 2	9 6	9 0	9.2	5 0
			3 30 - 4 30	1,049	41 2	2 6	4 5	4 6	5 4	5 8	6 5	6 0	6 7	7 7	3 4
			4 30 - 5 00	1,594	35 1	2 0	2 3	2 4	4 4	4 1	4 8	4 5	4 5	4 9	2 3
	Mt. Vernon Mem. Pkwy	May 4	2 30 - 3 30	352	40 5	10 0	-	11 8	8 1	7 0	9 3	12.7	8.9	13 0	10 2
			3 30 - 4 30	493	40 4	7 3	4 8	6 4	7 4	7 2	7 8	11 4	11.2	10 3	7 3
			5 00 - 6 12	703	40 7	4 2	5 8	5 6	6 7	6 7	6 7	9 9	10 0	9 2	5 1
2	Wash -Balt Pkwy	Apr 25	2 42 - 4 06	474	49 2	7 7	3 9	6 0	5 6	6 0	5 3	8 4	9 8	7 5	7 6
			4 06 - 5 54	1,188	49 2	3.0	2 7	2 4	4 5	4 5	4 2	5 4	5 9	4 9	3 0
			5 54 - 6 30	872	50 2	4 0	4 8	4 2	4 7	5 0	5 6	6 4	8 6	6 7	4 1
	Shirley Hwy	Apr 26	2 30 - 3 30	605	51 9	4 2	5 6	5 8	7 1	5 8	7 2	9 5	9 5	9.5	6.0
			3 30 - 4.30	1,314	48 1	2 6	1 9	2 8	5 4	5 8	6 5	4 9	4 6	5 2	2 7
			4 30 - 5 00	1,908	37 2	1 8	1 6	1 8	4 4	4 6	4 5	3 8	3 5	4 0	1.9
	Mt. Vernon Mem Pkwy	May 4	2 30 - 3 30	203	45 9	18 1	10 1	14 8	8 1	7 0	9 3	11 8	13 5	9 9	17 8
			3 30 - 4 30	433	44 8	8 2	7 4	6 6	7 4	7 2	7 8	10 0	9 5	10 8	8.3
			5 00 - 6 12	757	45 4	4 0	6 8	5 7	6 8	6 7	6 7	7 5	8 6	8 8	4 8

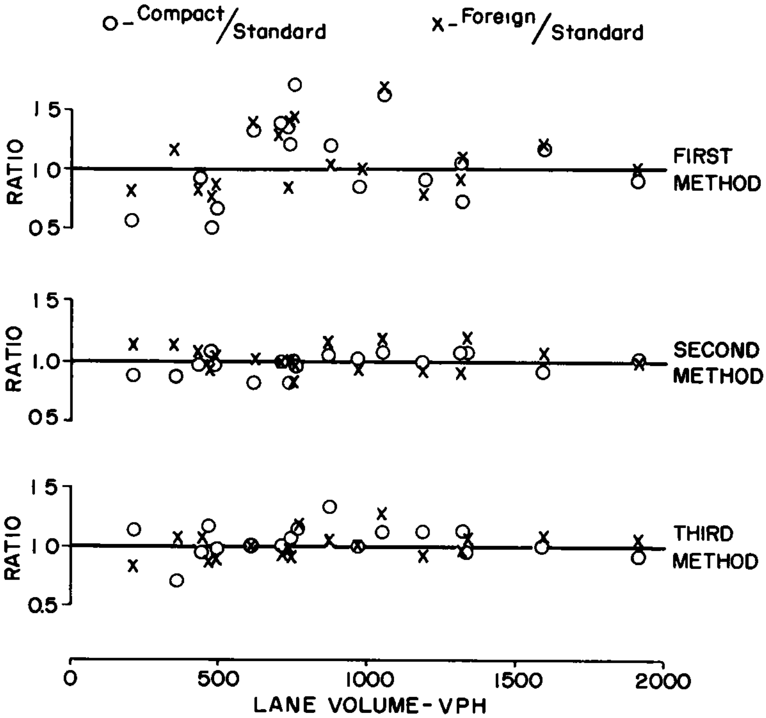

Figure 5. Ratio of time spacings, compact vehicles to standard passenger cars, and foreign vehicles to standard passenger cars, by volume.

Table 3 gives the average speed obtained at the grade location by lane, station, time period, and type of vehicle, and Figure 4 shows a comparison of the speeds by vehicle type.

TABLE 5

HEADWAYS OBTAINED FROM USING QUEUES, SHIRLEY HIGHWAY, SOUTHBOUND, APRIL 26, 1960

Time (PM)	Lane	Traffic Vol. (vph)	Uniform Spacing (sec)	Vehicle		Headway[1] (sec)			
				Type	No.	1	2	3	4
2:30 - 3:30	1	725	5.0	Compact	4	4.1	2.2	3.8	4.1
				Foreign	14	5.5	4.6	6.1	4.8
	2	605	6.0	Compact	12	5.7	5.3	3.7	6.3
				Foreign	19	7.3	3.8	6.2	4.6
3:30 - 4:30	1	1,049	3.4	Compact	22	2.6	3.3	2.5	2.6
				Foreign	29	2.8	3.4	4.1	3.3
	2	1,314	2.7	Compact	32	3.1	2.7	2.0	2.4
				Foreign	44	2.6	2.1	2.8	3.0

[1]Headway 1 Standard-Standard (end of queue); Headway 2 Standard-Small, ahead; Headway 3 Small-Standard, ahead; Headway 4 Standard-Standard (beginning of queue).

There appears to be no great difference in speed or consistent trend toward a higher or lower speed for compact or foreign cars as compared to standard American passenger cars at any of the locations. The tables do show the normal difference in speed associated with varying volume. There is also an expected difference in speed between lanes 1 and 2, the difference being about the same for each vehicle size.

Spacing values were calculated for the several study locatins. Three different procedures were used to determine the space occupied by the small cars in the traffic stream. The first calculation was based on the formula used by the Bureau of Public Roads for determining the passenger car equivalents of commercial vehicles. This method utilizes the complete range of time spacings, introducing a possible bias due to the large spacings at lighter volumes. The second method was used to remedy this bias. Here spaces exceeding 20 time units, approximately 6 sec, were eliminated and only those spaces involving passenger cars were used. Finally, a third analysis was made which did not attempt to determine an actual time-space value but attempted to determine the average gap used by each type. The time space ahead and behind a vehicle was tabulated and the average (a + b) value calculated for each type. The vehicle ahead and behind had to be a standard passenger car and all values exceeding 20 sec were eliminated.

Table 4 gives the time-space values, in seconds, obtained by the three methods. The volume rates and average speed for the condition under which these values were obtained is also shown. Figure 5 shows the ratio of time space for compact and foreign cars as related to the standard passenger car, obtained by the three methods at varying volume rates. A comparison of the time-spacing and time-space ratios does not show any consistent difference in spacing for the small cars. The ratios show compact and foreign car spacings to be scattered and both higher and lower than standard car spacings. There is the normal decrease in the average spacing for all vehicle types with increase in volume. However, the ratio of the space occupied by standard cars and small and compact cars remains quite uniform.

After review of the paper by Forbes and Wagner, a limited analysis based on two hours of the study on Shirley Highway was done on a comparable basis to the Michigan study. This involved determination of headways, where the small car was in the middle of a five-car queue. Table 5 gives the values obtained by using a method similar to that used by Forbes and Wagner. Again there is no apparent trend in the data to signify a difference for small cars.

The data presented here represents traffic having a higher percentage of small cars than shown in the Michigan study and the use of a larger number of vehicles in this preliminary analysis. The data in both studies indicate that there is no significant difference between the operation of standard cars and the small cars as measured by their speed and spacing.

These studies were limited to highways that would be classed as urban expressways. It is possible that further research, where consideration is given to the effect of the small cars on city streets and on open rural highways where the average speed more nearly approaches the speed capabilities of the small cars, could yield valuable information for the determination of the over-all influence of small cars on driver behavior.

REFERENCE

1. Taragin, A., and Hopkins, R. E., "A Traffic Analyzer: Its Development and Application." Public Roads, 31:5, pp. 120-124.

Congress Street Expressway Traffic Characteristics

LEO G. WILKIE, Traffic Engineer, Cook County Highway Department, Chicago

• EXPRESSWAY FACILITIES reach volumes far beyond design requirements soon after they are built. Their level of service for most hours of the day is exceptionally high, however, during peak hours when the public critically expects a high level of service, volumes build up and intolerable congestion prevails. These periods of congestion bring strong public criticisms against costly expressways. This study was designed to isolate those elements of expressway performance which affect traffic flow.

CONGRESS STREET EXPRESSWAY

This facility is approximately parallel to and five blocks south of the east-west street number base line of Chicago. It is generally a depressed section 16 miles long and utilizes four lanes in each direction between the Main Post Office, the Eastern Terminus, and Austin Avenue (6000 W) a distance of 7.5 miles and three lanes in each direction from Austin to the Tri-State Tollway—a distance of 8.2 miles.

The first section of Congress Street Expressway, 1st Avenue to Mannheim Road was opened to traffic in December 1954 and five subsequent sections. The last one (Central to Desplaines) which was put into operation in October 1960 completed this central portion of the comprehensive expressway system.

There are 18 on-ramps and 24 off-ramps westbound and 24 on-ramps and 19 off-ramps eastbound, a total of 85 ramps included in this study, an average spacing of $\frac{1}{2}$ to $\frac{3}{4}$ miles (see Fig. 1).

The maximum recently observed 24 hr totals were approximately 105,000 in the six-lane section and 165,000 in the eight-lane section.

Further, the maximum observed lane usage was 2,240 vehicles at Kedzie Avenue eastbound inside or curb lane AM peak hour.

Truck traffic in the AM peak eastbound averaged 7.2 percent for all on-ramps, the range extending to 32 percent. Westbound ramp truck traffic averaged 7.1 percent for all on-ramps, ranging up to 18 percent.

The prevailing ramp design is commonly described as diamond type except that in the west section both 25th Avenue and Mannheim Road are a cloverleaf type.

The ramp width is 16 ft and the ramp terminal length is 400 ft long, and where an additional weaving lane is provided it is 600 ft long and 12 ft wide.

The angles of intersection of the ramp centerlines with the freeway lanes is an average of 5^0 ranging between 4^0 and $14\frac{1}{2}^0$. Ramp grades vary from 2.25 to 4 percent.

With the exception of the Austin Avenue and Harlem Avenue interchanges the ramp exists and entrances are located on the right-hand side, except for two on the left-hand side.

A significant feature of the Congress Street Expressway is the presence of mass transit facilities in the median. This service was opened in June 1958. This mass transit service includes 14 stops which serve 22 cross-street loading points. In 1959, 35,000 passengers per day were transported, and in 1960, contrary to general trends 40,000 passengers were transported. These are 24-hr volumes. This expressway assuming a 1.5-person per vehicle car occupancy and including mass transit usage produces 350,000 person-trips per day.

SURVEY STRUCTURE

The field portion of this study was held on March 15 and 16, 1961, westbound outbound from the CBD on the 15th, and eastbound towards the CBD on the 16th. This study

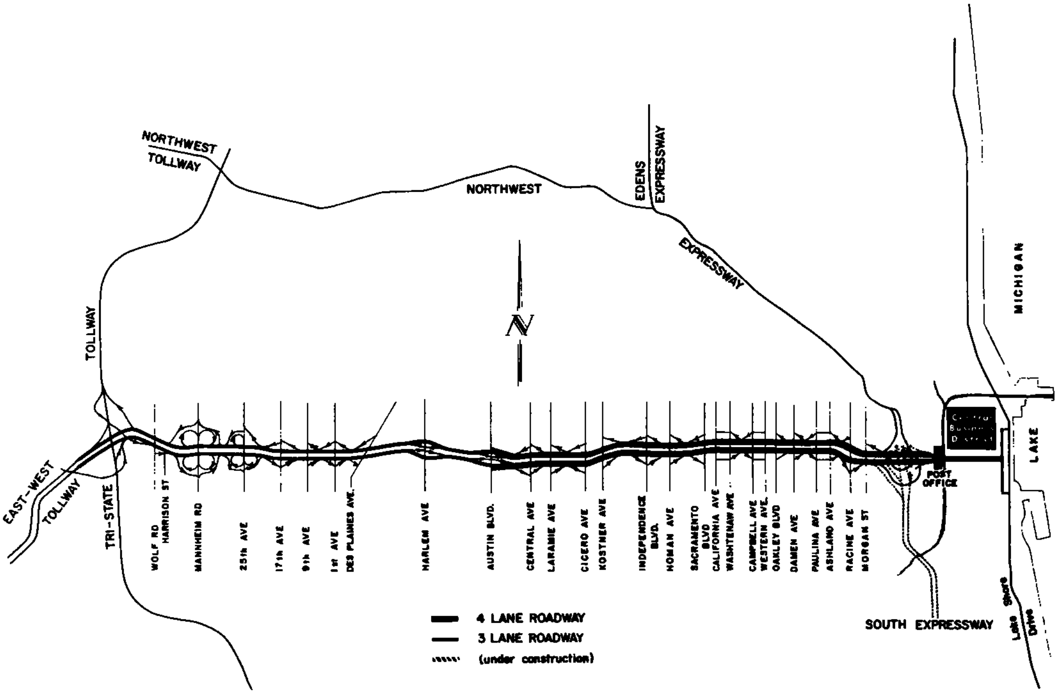

Figure 1. Congress Street Expressway, study area diagram.

was designed around the distribution and collection of pre-punched IBM cards distributed and collected at all 85 on- and off-ramps of the 16-mi expressway.

Card accumulation (normal card color for passenger cars and red for trucks) was in 15-min increments. Westbound and eastbound cards were distributed and collected between 7 AM and 1 PM. Adverse light conditions prevented a PM peak study. In the two 6-hr sections of trip study 95,000 cards were collected. This accumulation of data resulted in 70 separate analysis which will be contained in the final report. To conduct this study 112 men were required on each of the two days.

RAMP VOLUMES

Pouring into the expressway via 18 on-ramps during the first day of the survey Wednesday, March 15, between 7 AM and 1 PM were 44,000 vehicles westbound, 85 percent of which were autos. The actual average hourly volume was 7,300 vehicles per hour with the peak rate (based on the highest 15-min period) of 11,100 vehicles per hour. The lanes east of the post office, the east end of the study section supplied 30 percent of the 6-hr volume and 24 percent of the peak volume.

Entering trucks accounted for 7.1 percent of the westbound AM peak hour. The westbound volume of 44,000 vehicles produced 202,000 vehicle-miles in the 7 AM to 1 PM period.

During the same 6-hr period on the following day, an eastbound volume of 50,000 vehicles (88 percent automobiles) produced 291,000 vehicle-miles. For this direction of travel, 8,400 vehicles was the average hourly rate with a peak hourly rate again based on the maximum 15-min flow of 11,900 vehicles per hour. Twenty-four entrance ramps produced these flows with the west terminal (Roosevelt Road) contributing 12 percent of the 6-hr total and 14 percent of the peak hour volume. The eastbound truckflow during the AM peak was 7.2 percent of the total peak hour. It was found that the eastbound traffic exceeded that of the westbound during the 6 hours by 12 percent and during the peak hour by 7 percent. A significant fact, however, is that the eastbound vehicle-miles of travel exceeded the westbound by 45 percent.

Expressway volumes were determined by the successive net accumulation of vehicles just downstream of each ramp. It was found that for the eastbound portion of the study about ½ the total expressway was operating above an assumed capacity of 1,500 vehicles per hour per lane. In spite of this, reasonable average speeds were experienced.

RAMP USAGE CHARACTERISTICS

AM Peak Hour, Westbound

Reading from right to left the direction of travel, Figure 2 shows entrance and exit ramp volumes. A relatively small percentage of the total trips are destined to the western terminus of the expressway (Roosevelt Road). The bulk of the trips leave this facility (Congress Street) within the limits of Chicago (Austin Avenue).

AM Peak Hour, Eastbound

By far the greatest bulk of trips during the Eastbound AM peak entered the expressway in the west portion (from Roosevelt to Mannheim Road). The preponderance of exiting traffic eastbound AM peak occurred between Racine Avenue and the post office. This can be seen by the bars at the extreme right-hand side in Figure 3.

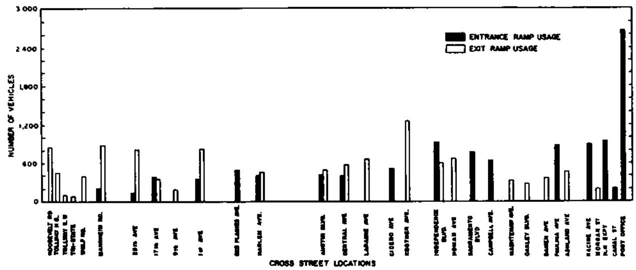

Figure 2. Ramp usage characteristics, westbound, AM peak hour.

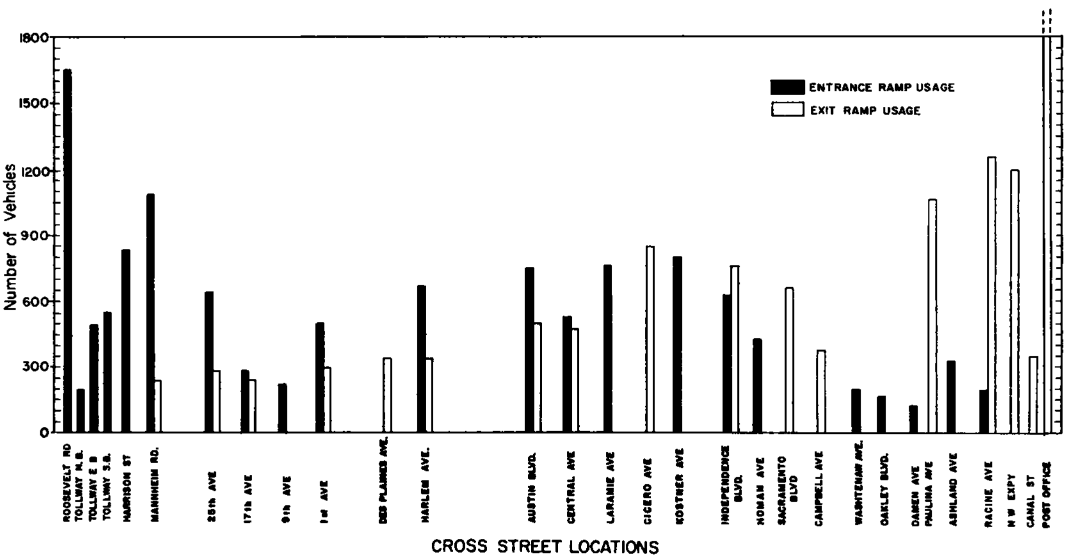

Figure 3. Ramp usage characteristics, eastbound, AM peak hour.

EXPRESSWAY VOLUMES

Normally highway volumes are stated as 24-hr two-way totals crossing a line. Traffic movements before or after such a counting line are obviously not included. In this study all ramp volumes were totaled. Based on this approach, the "expressway usage" volume would be 240,000 vehicles in 24 hr for Congress Street Expressway.

This "expressway usage" total reflects all on and off movements occurring over the entire expressway rather than across one line.

Figure 4 shows the eastbound volumes (in black) in large part exceed 1,500 vehicles per hour per lane. The density chart (Fig. 10) shows that this high volume alone, does not necessarily create congestion. Many of the high-volume sections were found to be operating at reasonably high speeds.

TRIP LENGTHS

Figure 5 shows the time required to make trips of various lengths at speeds of 25, 45, and 60 mph which correspond to arterial highway speeds, expressway speeds at peak hour, and expressway speeds at noon peak hours where posted speeds can be realized. The impact from this figure does not come from the relative time differentials (which are constant) but from the absolute time differentials; that is, a two-mile trip at 60 mph which requires 2 min, requires only slightly over 4 min at 25 mph.

On the other hand, a saving of 18 min can be realized on a trip of 16 mi (the length of this expressway) using the same speed values for the trip of 25 and 60 mph—25 mph being the criterion on arterial systems and 60 mph on expressways at non-peak hours. It is evident from this that the real value of a costly highway facility is to provide for maximum trip lengths of 10 miles or greater.

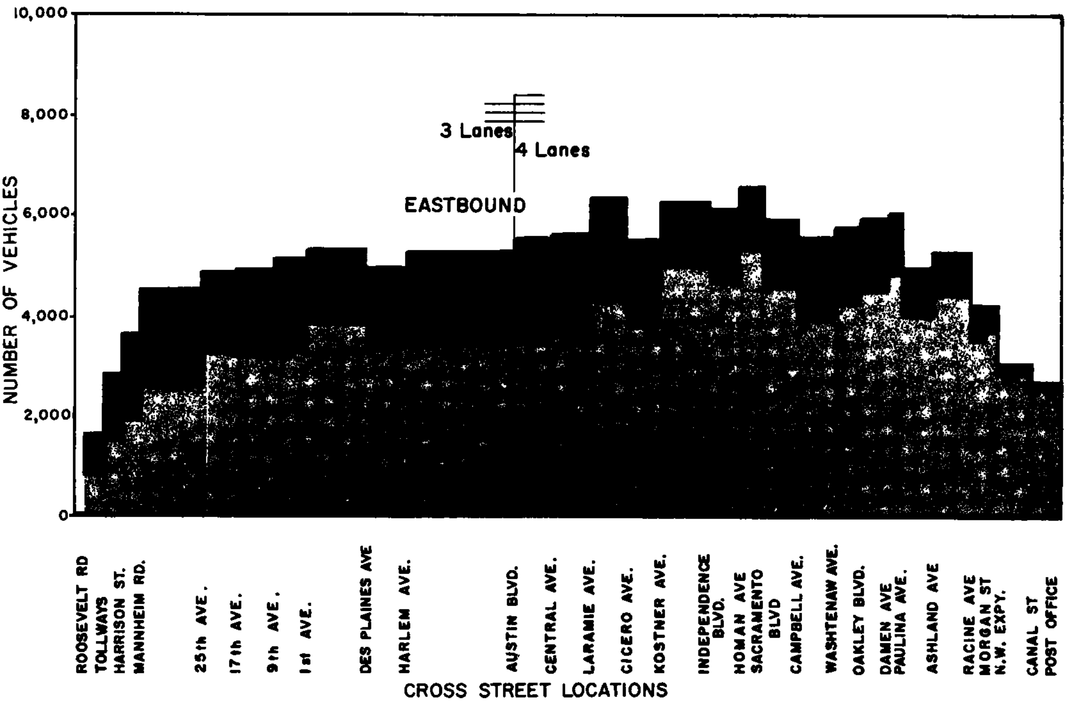

Figure 4. Vehicular accumulation, AM peak hour.

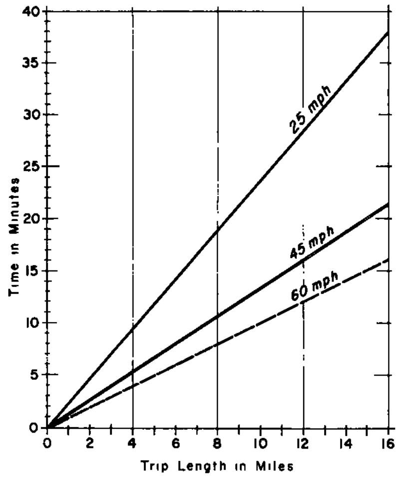

Figure 5. Expressway and arterial travel times related to trip length.

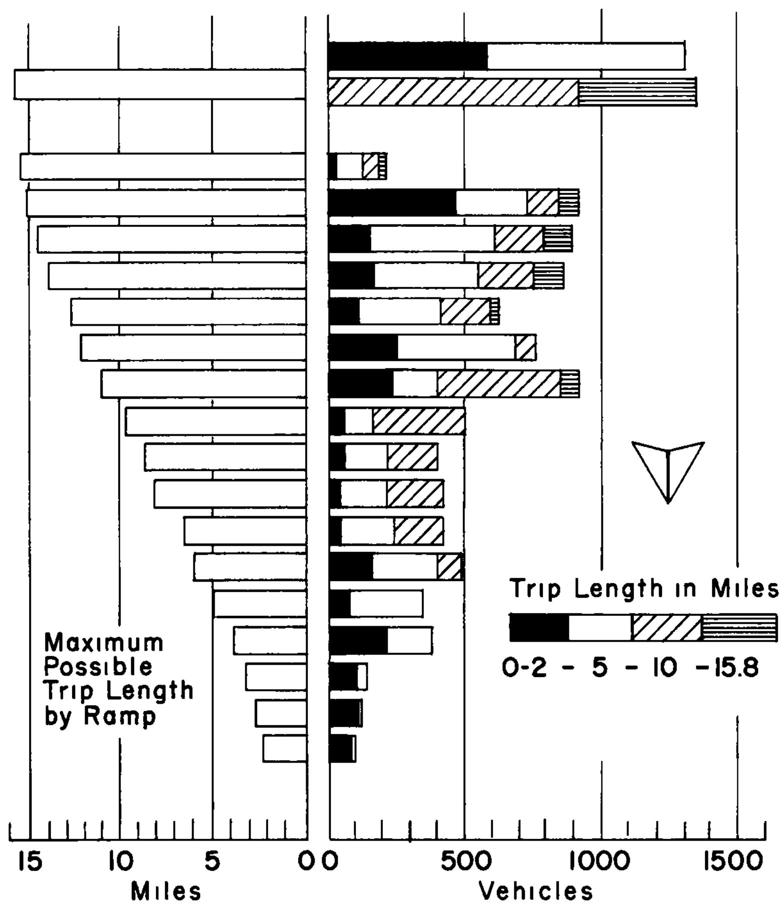

Figure 6. Westbound AM peak volumes and trip lengths, by entrance ramps.

Westbound Volumes and Trip Lengths

Figure 6 shows the volumes and trip lengths originating from each of the westbound on-ramps during the Wednesday AM peak hour. A sizeable portion of the total trips is less than 2 miles. These trips are easily discernible as they are shown in black. In general it is desirable to discourage these short trips from using the expressway.

The trips portrayed by the white portion of the bar graphs are from 2 to 5 miles in length and make up the greatest bulk of the westbound trips.

Eastbound Volumes and Trip Lengths

Figure 7 shows a marked difference in the number of short trips (0 to 2 miles) particularly at the west terminus. Of the trips entering at the west terminus almost ½ traveled 10 miles or more on the expressway. This type of trip length is most desirable.

These expressways like any other express service are intended to serve relatively long trips. It is, therefore, quite apparent that the elimination of short trips would be conducive towards increasing the efficiency and vehicle-mile productivity of an expressway facility.

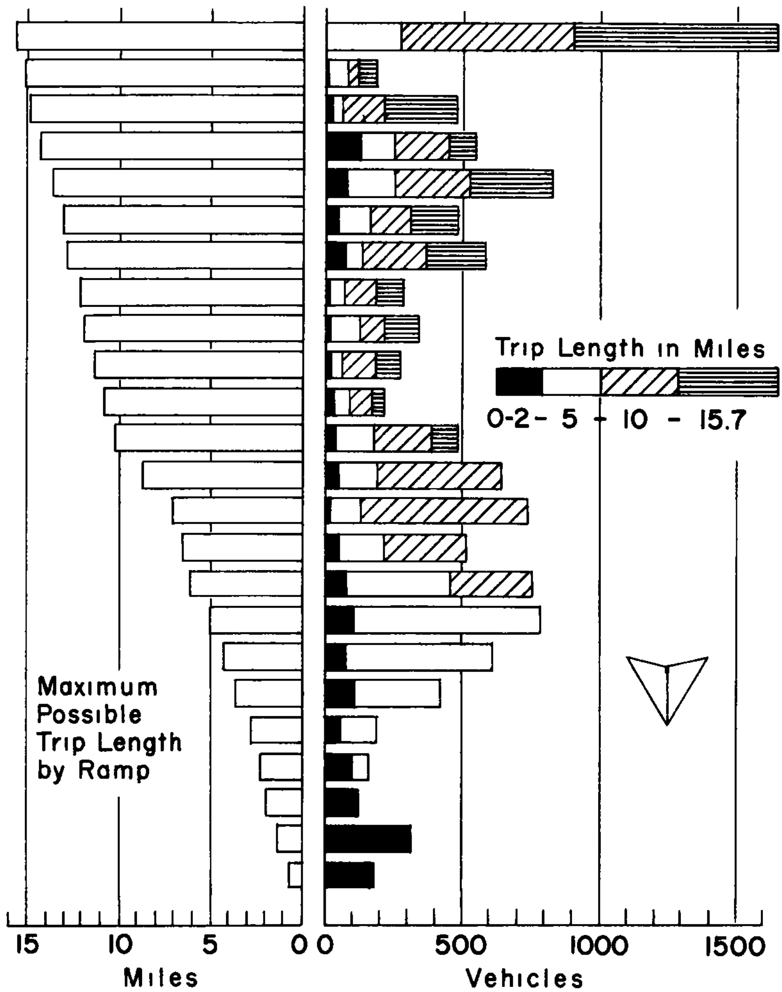

Figure 7. Eastbound AM peak volumes and trip lengths, by entrance ramps.

Vehicle-Miles

Figure 8 shows that a substantial portion of the expressway vehicle-miles can be accounted for by relatively short trips. For example, about 15 percent of the westbound vehicle-miles are produced by trips of 3 miles or less. From this it follows that a change in design for a new facility or a ramp control on an existing facility will result in an increased average trip length and consequently the lessening of congestion.

Trip Lengths vs Exit Opportunities

Up to this point in the report, trip lengths have been considered in terms of absolute length. However, ramp spacings are of primary significance. Figure 9 shows that

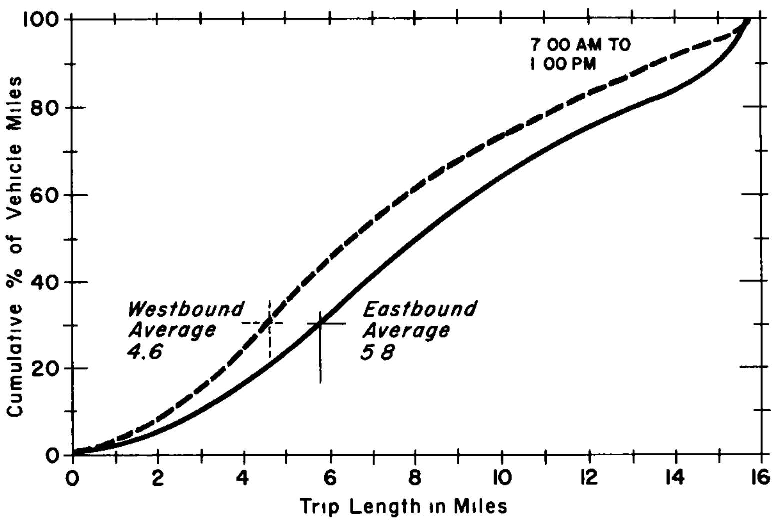

Figure 8. Cumulative distribution curve, expressway vehicle-miles related to trip length.

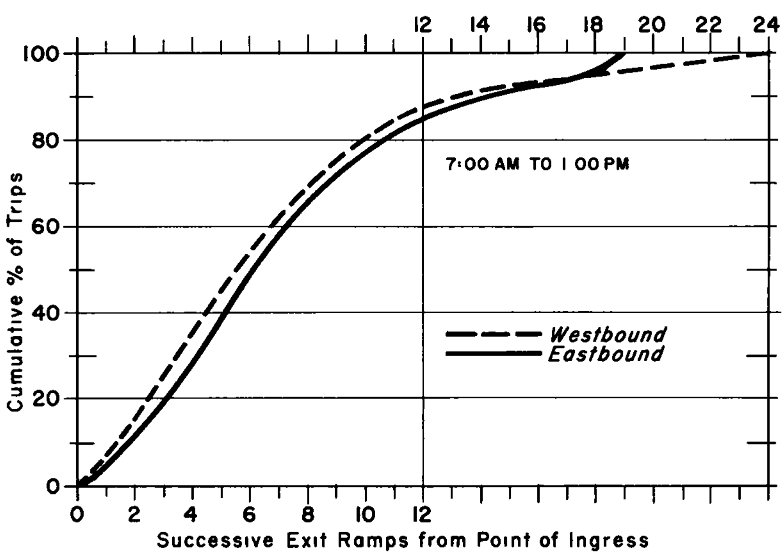

Figure 9. Trip lengths and opportunity for egress.

about 10 percent of the drivers left the expressway at their first or second opportunity to do so and 50 percent had left by the time the seventh opportunity presented itself. From this it is further emphasized that greater ramp spacings will tend to produce more nearly expressway-like trips, thereby reducing undesirable turbulence.

DENSITIES

Figure 10 shows the relationship between speed (dotted line) and density (solid line). When the density in vehicles per mile per lane exceeds the speed in miles per hour, congestion tends to develop. This cross-over occurs at a speed of 40 mph per lane.

There were only two sections on the expressway where this condition was found to exist. The more pronounced of the two was in the area of the two "center" interchanges. It was felt that these center ramps might very well be contributory to the congestion which was found. A further study of these ramps is indicated.

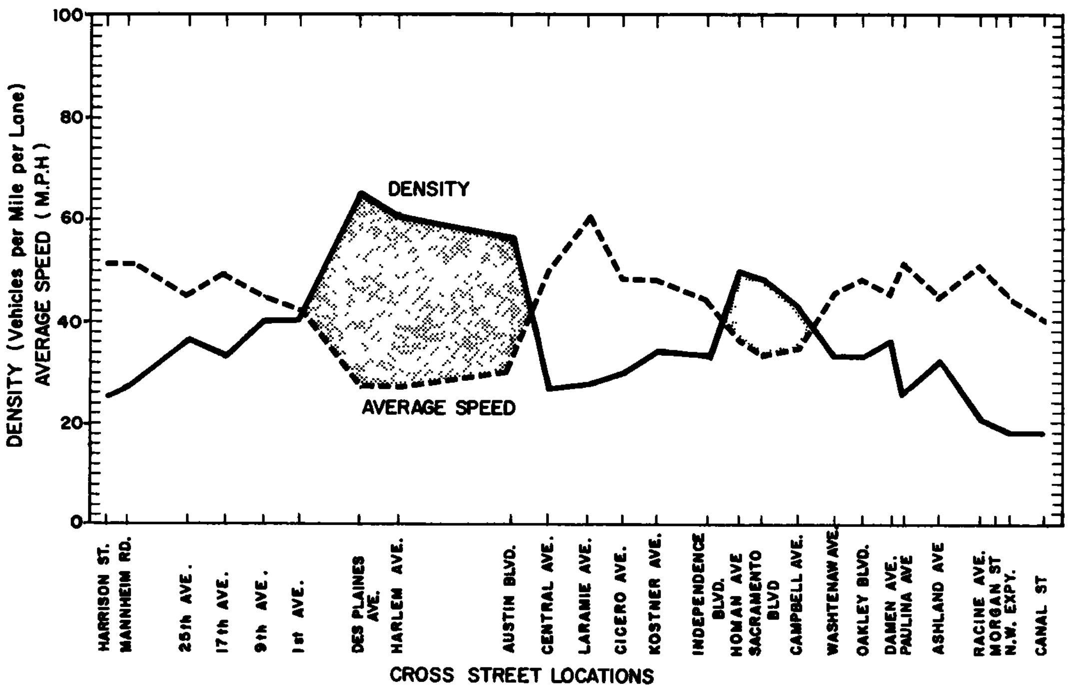

Figure 10. Density and speed characteristics, eastbound, AM peak hour.

SUMMARY

Expressways, most of which are operating at or above design capacity, are being choked up by sizeable numbers of short trips. This congestion is reflected in both the additional trips on the expressway as well as the added turbulence of the many interchanges. It does not appear sound economically to design for trips under 2 miles. Although a study such as this can point to better ramp spacings for future design, some sort of ramp control is needed to improve operating conditions on existing facilities.

The Cook County Highway Department is developing a simulation program to test various ramp controls. This program is based on the data gathered in this study. Testing various other hypotheses, such as the effect of eliminating all trucks from the expressway during the peak hours, will also be possible. Such a simulation technique is clearly desirable to augment the benefits from any proposed controls.

Ultimately, the economics of building and maintaining an expressway, and the economic gain realized by the users are the prime considerations. The economic gains are increased when the vehicle-miles of travel on the facility are at a maximum. The author is confident from the experience in this study that ramp spacings must be increased if this maximum is to be reached. The goal as designers and operators should be the highest productivity possible. When the output reaches the maximum level of vehicle-miles per hour, then that goal is reached.

Some Characteristics of Peak Period Traffic

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•TODAY serious or incipient congestion can be found on the freeways of any large metropolitan area, and the impression is growing that new urban freeways invariably become clogged with vehicles during peak hours within a few years of their completion. Experience of this sort has tended to breed skepticism among planners, and even highway administrators, about the ability of urban highway expansion to stay abreast of the automobile flood, except by unduly large investments of public funds in roads. One cause of this hesitation about highway improvement is the difficulty of predicting how long an urban freeway can provide effective service to traffic before it is snarled by peak-hour volumes.

The study described in this paper concerns the type of situation where (a) an urban highway route has been congested at peak periods for many years, (b) a substantial addition to road capacity is soon to be made, (c) a restraining limit on traffic growth will thus be removed, and (d) the change can be expected to alter peak-period traffic flow rates. Traffic surveys have been instituted on three important travel arteries in the San Francisco Bay area where notable highway improvements will soon be completed so that it will be possible to identify the time pattern of traffic flows before and after the new highway capacity is added. The surveys are being taken often enough to show trends over time. The findings of the Institute of Transportation and Traffic Engineering will not be available for two or three years, but it was thought it worthwhile to report some of the information collected about peak-period traffic.

The main object in this inquiry will be to determine how the highway improvements will affect the capacity standards used in the course of planning for the urban peak demand. Specifically, it refers to the designation of a "design hour volume" (DHV) to show the future road capacity required along a route, the expression of DHV as a function of total road usage in the familiar formula DHV = ADT x K/100 x D/100, and the concept of "practical capacity," which is based on an "acceptable" vehicle flow rate for a given amount of highway space. Although planning judgment is inherent in all of these values, they tend to be treated somewhat as the result of measurement and prediction; thus, the K-value chosen to designate the future DHV for a highway project is often taken from the existing percentage relationship of peak hour to total daily traffic.

Growing traffic congestion has the effect of lowering the peak percentage values measurable from present traffic flows. If roads were available in such plentiful supply that the rigors of the rush hour played no part in drivers' decisions, a curve depicting the rate of traffic flow over the peak period would probably resemble a mountain peak having a pointed summit, whose altitude would be the size of the peak demand. However, the typical traffic profile for the crowded urban arterial highway in the late afternoon of a weekday has more the appearance of a mesa than a mountain—a mesa with a nearly flat plateau at its maximum height, whose altitude is limited by the capacity of the road to move vehicles. As the peak is flattened, it spreads; and the more potential demand there is during the maximum time period, the wider the spread is likely to be. The spreading of the peak reduces the percentage of total traffic occurring in the maximum unit of time. Congestion both induces and forces drivers to adjust their time and place of travel, and it is reasonable to expect that the relief of congestion would produce changes in the opposite direction.

So far, there has been little specific study of changes in the urban traffic pattern as the result of providing space for vehicles where none before was available. The inquiry intends to record the changes as they occur, particularly in the peak percentage relationship which forms the factual basis of the urban K-value. Presently, it has become possible to describe the peak travel situation before the improvements are completed.

STUDY LOCATIONS

Three sites in the San Francisco-Oakland Bay Area were selected for this project (Fig. 1). They have in common the following characteristics:

1. The facilities themselves are access controlled by virtue of physical circumstances (a bridge, a tube, a tunnel). Traffic streams do not merge, diverge, or cross within the facilities, except for normal lane changing maneuvers. (Even these do not occur in the two-lane facility being studied.) Traffic is not interrupted by control devices, except on the lower deck of the Bay Bridge, where there is a traffic signal—soon to be eliminated. However, the facilities fail to meet freeway standards as regards provision of shoulders, widths of lanes and, in two cases, separation of opposing traffic streams.
2. The facilities were all constructed before World War II, and have remained substantially unchanged since their opening. At all sites, major construction projects are now underway to increase capacity.
3. The facilities have suffered from peak-hour congestion (as shown by reduced speeds, formation of queues, limitation on drivers' ability to maneuver) for at least ten years, but they have sufficient capacity to handle normal off-peak traffic flows at fairly acceptable standards of service.

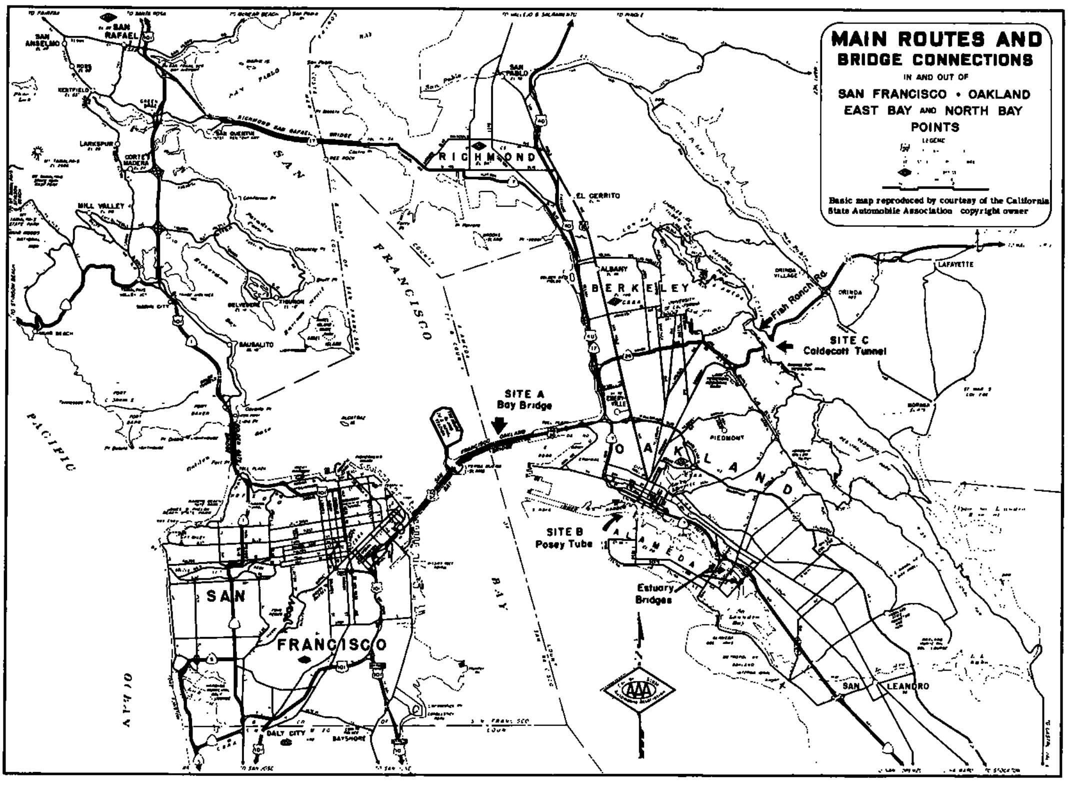

Figure 1. Location of study sites.

4. The capacities of the approaches to these facilities are greater than those of the facilities themselves for at least one direction in each case, permitting the possible capacity of each facility to be determined for at least one direction.

Site A—San Francisco-Oakland Bay Bridge

The San Francisco-Oakland Bay Bridge is the only vehicular facility connecting the two central cities of the metropolitan area across the Bay. The nearest parallel crossings are 10 mi to the north and 20 mi to the south; hence, alternate routes are available for only a minute proportion of trips using this bridge. It would appear that present traffic conditions on the Bay Bridge have had no significant diversionary effect on traffic movement, and the nearest parallel crossings are not being studied to detect such effects.

The geometric features of the bridge which determine its capacity are given in Table 1. The upper deck is restricted to private automobiles and pickup trucks at all times; commercial vehicles must use the lower deck. During peak periods, the center lane of the lower deck operates in the peak direction only, and passenger cars are permitted to use the lower deck during these hours in the peak direction, giving a total of 5 lanes for all peak traffic in the major direction. At other times, passenger cars are not permitted on the lower deck. (Since completion of this phase of the studies, operating rules for the lower deck have been changed to permit passenger cars at all times.)

TABLE 1

PHYSICAL DATA AFFECTING VEHICULAR CAPACITY

Item	Site				
	Bay Bridge[1]			Posey	Caldecott
	Upper Dk	Lower Dk	Total	Tube	Tunnel[2]
Traffic lanes:					
Total	6	3	9	2	4
No. for peak direction of flow	3	2	5	1	2
Widths per lane (ft)	$9^2/_3$	$10^1/_3$	-	11	11
Shoulders	None	None	-	None	None
Median type	Raised	None	-	Paint	Walls
Median width (in.)	8	-	-	10	-
Lateral clearance (ft):					
Outer lane edge to curb	None	None	-	None	None
Curb to wall or rail	1 - 3	1 - 3	-	4	4
Posted speed limit (mph)	40	35	-	35	45
Maximum grade:					
Amount (%)	3 up	3 up	-	4.5 up	4 up
Direction	To east	To east	-	Both	To east
Length (mi)	1	1	-	$^1/_4$ ea.	$^3/_4$

[1]Also, 2.7 percent upgrade for $1^1/_4$ mi west.
[2]Also, long upgrades on both approaches.

The present possible capacity of the bridge in the peak direction appears to be about 7,000 vehicles per hour (7.5 percent dual-tired), as given by the figures in Tables 2 and 3. However, because of the absence of shoulders and median over a distance of about 4.5 mi, traffic flow is drastically curtailed whenever a vehicle stops for any reason (out of gas, breakdown, accidents). Stoppages occur in a random manner, but frequently enough to cause wide fluctuations in flow rates measured.

TABLE 2

VEHICULAR CAPACITY DATA, 1961 WEEKDAY

Item	Site				
	Bay Bridge			Posey	Caldecott
	Upper Dk	Lower Dk	Total	Tube	Tunnel
24-hr vehicular volume, both directions:	99,765[a]	17,050[a]	116,815	29,645	50,302
AM peak hour:					
Time peak hr starts (AM)	6:54	7:00	6:54	6:30	7:30
Volume in peak direction:					
Vehicles	4,784	1,947	6,658	1,415	3,697
% of 24-hr direc. total	9.4	24.9	11.3	9.4	14.9
Dual-tired vehicles in peak direction (%)	0	20.6	5.9	2.1	2.0
Volume in opposite direc. (vehicles)	2,618	312	2,920	888	1,407
Volume in both directions:					
Vehicles	7,402	2,259	9,578	2,303	5,104
% of 24-hr total for both directions	7.4	13.2	8.2	7.8	10.1
Directional split (%-%)	65-35	86-14	70-30	61-39	72-28
PM peak hour:					
Time peak hour starts (PM)	4:42	4:36	4:42	4:12	4:54
Volume in peak direction:					
Vehicles	4,294	2,630	6,905	1,322	3,281
% of 24-hr direc. total	8.8	28.5	11.9	9.1	12.9
Dual-tired vehicles in peak direction (%)	0	18.6	7.5	3.5	1.7
Volume in opposite direction (vehicles)	3,512	374	3,867	1,257	1,264
Volume in both directions:					
Vehicles	7,806	3,004	10,772	2,579	4,545
% of 24-hr total for both directions	7.8	17.6	9.2	8.7	9.0
Directional split (%-%)	55-45	88-12	64-36	51-49	72-28
Two-Directional Peak hour:[b]					
Time peak hour starts (PM)				4:18	7:12
Volume in both directions:					
Vehicles				2,595	5,163
% of 24-hr total for both directions				8.8	10.3
Directional split (%-%)				50.2-49.8	71-29

[a]Approximate figure.

[b]For Bay Bridge, same as PM peak hour.

The bridge is now being reconstructed by removal of train tracks and reconstruction of both decks to provide 5 lanes, 11 ft 7 in. wide, for each direction; opposing traffic will be completely separated, westbound vehicles being on the upper deck and eastbound vehicles on the lower. This will no longer permit the operation of any lanes reversibly, as is the present procedure. Nevertheless, the design engineers have forecast an increase in capacity of from 30 to 35 percent, partly because the effect of stopped vehicles is expected to be much smaller.

TABLE 3

HIGHEST, LOWEST, AND AVERAGE PEAK VOLUMES OBSERVED DURING DIFFERENT COUNTING PERIODS

Facility	Direc of P F	Length of Counts	No of Obser-vations	Traffic Volume (veh)			Percent of 24-Hr Direc Total		
				Highest	Lowest	Average	Highest	Lowest	Average
Bay Bridge	Westbound	Peak hour	14	6,833	6,378	6,600	13 6	11 5	12 3
		Peak half-hour	14	3,718	3,428	3,606	7 3	6 3	6 8
		Peak 6 min	14	791	722	754	1 6	1 3	1 4
	Eastbound	Peak hour	15	7,115	5,673	6,625	13 5	11 3	12 5
		Peak half-hour	15	3,669	3,076	3,467	7 0	5 9	6.5
		Peak 6 min	15	793	625	736	1 5	1 3	1 4
Caldecott Tunnel	Westbound	Peak hour	4	3,894	3,439	3,725	16 3	14 4	15.5
		Peak 6 min	4	441	375	417	1 8	1 6	1 7
	Eastbound	Peak hour	4	3,281	2,995	3,163	13 8	12 9	13 4
		Peak 6 min	4	355	320	342	1 5	1 4	1 5
Posey Tube	Southbound	Peak hour	4	1,555	1,391	1,446	10.3	9 4	9 9
		Peak 6 min	4	180	157	165	1 3	1 0	1 1
	Northbound	Peak hour	4	1,356	1,321	1,332	10 5	8 9	9 6
		Peak 6 min	4	190	145	160	1 5	1 0	1.2

Site B—Posey Tube

The Posey Tube connects downtown Oakland with the western section of the City of Alameda under the "Oakland Estuary." (Alameda is actually an island only accessible by this tube and bridges, and the "estuary" is actually a tidal canal.) The tube provides the best connection from the western part of Alameda, which includes a naval air station, to most of the remaining metropolitan area, including San Francisco. It is also a good route out of other parts of Alameda. The nearest parallel crossings are three bridges located 2.5, 3, and 3.3 mi to the east. These bridges are located closer to the downtown sector of Alameda, but further from Oakland's downtown. On the Oakland side, a freeway parallels the estuary and provides a good connection between all the crossing facilities; on the Alameda side, several major streets provide a somewhat slower connection. Therefore, many trips crossing the estuary are offered a choice of facilities. It is probable that the present congested conditions at the Posey Tube have caused an appreciable diversion to the parallel bridges. Hence, they are also included in the long-range study (but not in this report).

The tube has two lanes with passing prohibited throughout its length. Therefore, it actually operates as two one-lane roadways. Vehicle stoppages are rare, none having occurred during any of the counting periods of this project. When they do occur, traffic flow is cut off entirely in the direction of the stoppage unless special police enter the tube to guide traffic around the halted vehicle.

The south portal of the tube is approached by 3 lanes, which reduce to one lane as the tube is entered. During the evening peak a police officer regulates traffic at this entrance, allowing one lane at a time to enter the tube. The north approach is fed from a four-lane city street. A nearby signalized intersection with a one-way street limits the capacity of this approach, but evidently not below the capacity of the tube itself. In fact, the tube can generally carry a heavier vehicle flow from the north approach southbound than in the opposite direction. Values for maximum traffic flows carried are given in Tables 2 and 3.

A new two-lane tube is being built 500 ft west of the existing tube. When the project is completed, each tube will operate in one direction only, and total capacity is expected to increase at least 100 percent.

Site C—Caldecott Tunnel

Caldecott Tunnel connects the central part of the San Francisco-Oakland Metropolitan Area with a section of Contra Costa County through the Berkeley Hills. These hills reach an elevation of almost 2,000 ft, and are a barrier to easy vehicular movement. While some roads cross this range of hills, only one of these (Fish Ranch Road) is of sufficiently high standards and close enough to divert a large volume of the traffic which would otherwise use the tunnel. This road is being included in the project, but not discussed here.

Traffic through Caldecott Tunnel during peak hours is largely of the commuter type. The region east of the tunnel is a "bedroom community," many of whose wage-earners work in Oakland, Berkeley, or San Francisco. Table 2 shows traffic at this location has a far heavier inbalance by direction during peak periods than at the other locations.

The tunnel consists of two bores, each two lanes wide, on a continuous 4 percent grade up from west to east. The top of the grade is near the east portal, and both highways approaching the tunnel do so on long uphill grades of 4 to 5 percent. The approach from the west is two lanes wide, that from the east three lanes wide. Fish Ranch Road joins the main highway just east of the east portal. Thus, traffic volumes on the east-approach highway are greater than through the tunnel itself. In the westbound direction this poses no capacity problem, because the extra uphill lane is available; however, there are only two lanes downhill from the tunnel eastbound, and these must carry all tunnel traffic plus the vehicles entering from Fish Ranch Road. The capacity of the tunnel in the eastbound direction is therefore not believed to be the critical one, and volumes measured are believed to be controlled by the merging just beyond the tunnel. In the westbound direction, tunnel capacity probably is the limiting factor.

The Fish Ranch Road alternate route, which joins the main highway at the east portal, diverges northwards on the west side of the hills, and is a possible, though inferior route for traffic originating or terminating in the northern part of the East Bay Area, and, perhaps, to traffic crossing the San Francisco-Oakland Bay Bridge. It is probably little used by traffic to or from Oakland and points south, nor by the majority of traffic to and from the Bay Bridge. The present role of this road as an alternate during periods of congestion can best be seen by noting that the westbound peak hour carries 36 percent of the 24-hr westbound total, and the eastbound peak hour 43 percent of the 24-hr eastbound total. The road sees relatively little use in off-peak periods.

A new bore, also two lanes wide, is being built just north of the existing bores, and the approach highways on both sides are being converted to eight-lane freeways. On completion of these projects, the center bore will be operated reversibly during peak hours, and there will be four lanes in the peak direction on the approaches and through the tunnel. In the direction opposite to the peak movement, traffic will be restricted to two lanes through the tunnel. Traffic capacity in the peak direction should increase about 100 percent.

FIELD STUDIES

The following studies are being conducted in the field:

1. Total Vehicle Volume Counts. —At all sites and parallel routes deemed to have a diversionary role. At the sites themselves, peak period data are subtotaled every 6 min, other data every hour. At parallel route sites, peak period data are subtotaled at either 6- or 15-min intervals.
   - (a) Bay Bridge—Counts are made by electronic machine connected to toll booths recording each transaction.
   - (b) Posey Tube—Counts are made by street traffic counters modified to print at 6-min intervals. Detection is by sonic vehicle detectors mounted overhead near one portal. On two of the parallel bridges, permanently installed photoelectric detectors are connected to street traffic counters; at the third bridge road tubes are used.
   - (c) Caldecott Tunnel—The same procedure as at Posey Tube is used. On Fish Ranch Road, road tube detectors are employed.

2. Vehicle Classification Counts. —At all sites, but not on parallel routes. These counts are made manually.
   - (a) Bay Bridge—Continuous counts are made during peak periods, using 6-min intervals for subtotals. During off-peak periods, 30 min are counted every hour, or data obtained by operating agency are used.
   - (b) Posey Tube—During peak periods, approximately 12 min are counted every one-half hour; during off-peak periods, about 24 min are counted every hour.

(c) Caldecott Tunnel—Same procedure as at Posey Tube.

3. Vehicle Occupancy Counts.—At all sites, but not on parallel routes. These counts are made manually.

(a) Bay Bridge—Counts are made at the toll plaza, by sampling three or four lanes at a time. Counting continues about 21 min per ½ hr in peak periods, and about 30 min per ½ hr in off-peak periods. This results in sample sizes of from 20 to 25 percent.

(b) Posey Tube—During peak periods, approximately 12 min are counted everyone-half hour; during off-peak periods, about 24 min are counted every hour.

(c) Caldecott Tunnel—Same time schedule as at Posey Tube. Both lanes are counted simultaneously, although during peak periods some vehicles may be missed.

4. Transit Riding Counts.—These are made at Sites A and B only.

(a) Bay Bridge—The two transit agencies carrying local passengers across the Bay Bridge both use a toll ticket system on which drivers enter the number of passengers carried on each trip. These data are tabulated for a 24-hr period against the time at which each trip is scheduled to pass through the toll gate. (If vehicles are not on schedule, no allowance is made for this fact.) Long distance bus schedules are not included in this tabulation, nor are buses that shuttle passengers between San Francisco and the various long-distance railroad terminals in Oakland.

(b) Posey Tube—Estimates are made of the numbers of passengers on every bus passing through the tube between 6:30 AM and 6:30 PM. In off-peak periods, these estimates are apt to be somewhat rough, because buses then may be traveling at 30 mph past the observer.

(c) Caldecott Tunnel—No transit counts are made at this site, because the speed of buses (up to 50 mph) and the type of equipment used would make counts very inaccurate. Well over one-half the buses passing through Caldecott Tunnel then proceed across the Bay Bridge (and vice versa) without intermediate stops; for this group of buses data are available at the Bay Bridge.

The series of field studies listed is performed at quarterly intervals (January, April, July, October) at the Bay Bridge, and at semi-annual intervals (Spring, Fall) at the other sites.

ANALYSIS OF PEAK PERIOD TRAFFIC PATTERNS

Basic data in these studies are the 6-min counts of all vehicles passing through the highway bottlenecks during peak periods. The picture of the peak they present before the forthcoming additions to highway capacity is portrayed in this section.

Inspection of the data for Site A, the Bay Bridge, is convincing that by 1961 the reconstruction of the lower deck had progressed to the point where enough of a change had been made in total Bridge capacity to affect the flow patterns. Therefore, the information for the Bridge in the remainder of this paper represents only the counts taken in 1959 and 1960. For the Tunnel and the Tube, the data presented here include all counts completed to date.

PEAK PERIOD VOLUMES AND PERCENTAGES

Generally, the values given in Table 3 represent the vehicle volumes that occur from the fullest use of the facilities during the entire peak hour; they indicate the maximum ability of the roads to handle traffic flows. This is not quite true for the Bay Bridge during the morning, westbound peak, when some additional cars could be carried on the lower deck in the maximum hour; it is also possible that slightly higher southbound volumes could be recorded for the Posey Tube if the city streets of Oakland permitted a more regular flow of vehicles. Otherwise, the peak situation is such that no additional vehicles could come onto the bridge, the tube or the tunnel in the peak hour without crowding others off.

Despite this fact, there is a certain amount of variation in the maximum volumes which are recorded upon separate days. Table 3 gives the extremes and the average values for each of the facilities, which is a rough way of noting the dispersion of the data. Any dispersion is due mainly to irregularities in the traffic stream caused by vehicle stalls, accidents, climate or driver behavior under highly congested conditions, rather than to differences in the total number of vehicles desiring to use the roads.

The range between the high and the low hourly values was found to be greatest for eastbound Bay Bridge traffic, mainly because of vehicle stalls. The finding has been that delays and accidents on the Bridge during the afternoon peak have caused a considerable fluctuation in the rates of vehicle flow. Because the counts are taken at the toll gates at the east end of the Bridge, there has been no opportunity to measure the effect of similar interferences to traffic during the morning peak, but the impression is that accident delays are not as severe as in the afternoon. About accidents, more will be said later.

Because traffic is at a maximum during the peak, the percentage relationships between peak and total daily traffic are the highest that could be calculated at the present time. Unless road capacity is increased, traffic volumes during the peak cannot rise. Therefore, traffic increases could occur only in off-peak periods, and this would reduce the percentage of total traffic in the peak.

The percentages are highest for the Tunnel and lowest for the Tube. The low values for the Tube reflect the directional factor: in both the morning and the afternoon, there is a peak flow in each direction in the Tube which reaches the full capacity of the facility for a sustained time period. Also, the Tube is located near the heart of the urban complex, and off-peak usage is relatively heavy. By contrast, the peak flow through the Tunnel is primarily composed of traffic coming out of bedroom suburbs in the morning and going home in the afternoon: it is highly one-direction, and the off-peak travel is moderate in size. The percentages for the Bridge show both influences: the Bridge connects the major central cities of the Bay area, but the primary flow is in and out of San Francisco, rather than Oakland, and the length of the Bridge has a restraining effect on off-peak movements.

Traffic volumes and peak percentages for shorter time intervals than the maximum hour have been recorded in order to have a measure of the "peaking" tendencies of the traffic stream. As the traffic flow profile acquires more of the aspect of a "peak," the values for the shorter intervals should rise in relation to the hourly values. If capacity were used so that there was a perfectly even flow of vehicles throughout the period, the volume in each 6-min interval would be precisely 10 percent of the peak-hour volume. However, even when there is a continuous queue of cars waiting to enter a bottleneck, the flow is not entirely smooth. This fact, previously observed by the California Division of Highways (1), is apparent in these studies. Referring to the average vehicle volumes in Table 3, each peak 6-min volume bears the percentage relationship to the peak-hour volume given in Table 4. This variation exists even when capacity is fully utilized.

If the addition to highway capacity along these routes were to cause a greater peaking of traffic, as is expected, the change should increase the peak percentages more for the shorter time intervals than for the maximum hour. This hypothesis will eventually be tested in the course of the investigation.

Peak Period Flow Profiles

Using the 6-min vehicle counts, an attempt has been made to determine the length of time during the rush hours that each facility is actually utilized to its full physical capacity. The traffic profiles over the total peak period are shown in Figures 2, 3, and 4. The traffic volumes represented by these curves are hourly rates of flow for

6-min intervals; that is, each 6-min volume is multiplied by 10, which converts it into an hourly rate.

TABLE 4

PERCENTAGE RELATIONSHIP OF 6-MIN VOLUME TO PEAK-HOUR VOLUME

Test Area	Traffic Flow	Percentage Relat. (%)
Bay Bridge	West	11.4
	East	11.1
Posey Tube	South	11.4
	North	12.0
Caldecott Tunnel	West	11.2
	East	10.8

The curves are averages of all of the individual days for which peak period vehicle counts were completed. The curves for the Bay Bridge are composed of counts taken between April 1959 and October 1960. Over this time, there was a mild increase in total peak period traffic, resulting in a slight lengthening of period of maximum flow, but the change is not of much significance. In the westbound direction on the Bridge, the curve represents 14 counts, and in the eastbound direction 19 counts. The Caldecott Tunnel curves are an average of 6 counts westbound, 7 counts eastbound, made during 1960 and 1961.

Figure 3 for the Posey Tube shows the traffic flow in each direction during the morning and afternoon peaks for reasons previously explained. Only 4 days are depicted by each curve (3 days in the southbound AM direction), which accounts for the irregularities in the lines.

The treatment of the traffic movements on the lower deck of the Bridge and on Fish Ranch Road at Caldecott Tunnel must now be explained.

Both the lower deck (at least at the time these counts were taken) and Fish Ranch Road are "inferior" routes to practically all of the passenger car traffic. They would be selected only because traffic congestion on the superior roads induced drivers to choose the less attractive alternative. This is much more true of Fish Ranch Road than the lower deck, but it is probable that enough additional capacity on the upper deck of the Bridge, or at the Tunnel, would divert nearly all of the passenger cars away from the inferior routes.

In Figure 2, the flow of vehicles on the upper deck of the Bridge has been estimated by deducting the lower deck traffic volumes from the totals. This procedure was required because the traffic counter at the toll gates does not distinguish between upper and lower deck vehicles. The lower deck figures were obtained manually, in the course of the vehicle classification counts. Only one classification count is made at each quarterly survey, so that the lower deck flow shown in the figure represents the average of seven days, rather than 14 or 19 days.

In Figure 4, the values for Fish Ranch Road were added to the total volumes for the Tunnel. The estimates for the Road are rough—only two days have been used in the westbound direction, and four days in the eastbound direction.

On each chart the average midday rate of flow has been shown, so that an impression might be gained of the extent that the maximum capacity of the facilities exceeds the normal traffic requirements of the time period between the morning and afternoon peaks.

<u>Bay Bridge AM Peak (Westbound)</u>.—If the vehicle flow has the appearance of a peak here, it is because the lower deck was opened to passenger cars during the peak period. When this availability of the lower deck began at 7:00 AM, the volume on the upper deck had almost attained the maximum flow rate of 5,000 vehicles per hour. This flow declined slightly as the uncongested lower deck attracted traffic, then rose as total volume increased. At 8:00 AM, the closing of the lower deck to automobiles coincided with an abrupt increase in the upper deck rate of flow.

Total volume on the Bridge reached a rate of almost 7,500 vehicles per hour on the average. This figure is probably the highest rate of flow possible for the five lanes open to peak traffic in the primary direction.

<u>Bay Bridge PM Peak (Eastbound)</u>.—At 4:00 PM, traffic on the upper deck was already at the maximum possible volume and well above the midday rate. (Further in-

vestigation showed a sharp rise in traffic beginning around 3:30 PM and continuing until full capacity is reached.) At 4:30 PM, the lower deck was opened to passenger cars, and the increased flow of vehicles arrived at the toll gates some 10 min later. Then there was a sharp increase in volume until the Bridge was operating at full capacity on both decks.

The maximum total rate of flow is only about 6,700 vehicles per hour, compared with 7,500 in the morning peak. On the upper deck alone, the flow averaged about 500 vehicles per hour less than the westbound flow in the morning. This difference reflects the effect of numerous stalls, delays, and accidents that occur on the Bridge. On each individual day, the maximum rate-of-flow usually exceeded 7,000 vehicles per hour for at least a short-time interval, but it was difficult for this rate to be sustained without interruption. Because stalls and other delays are distributed over the whole congested period of the peak, they tend to pull down the general average values for all days for the entire time period.

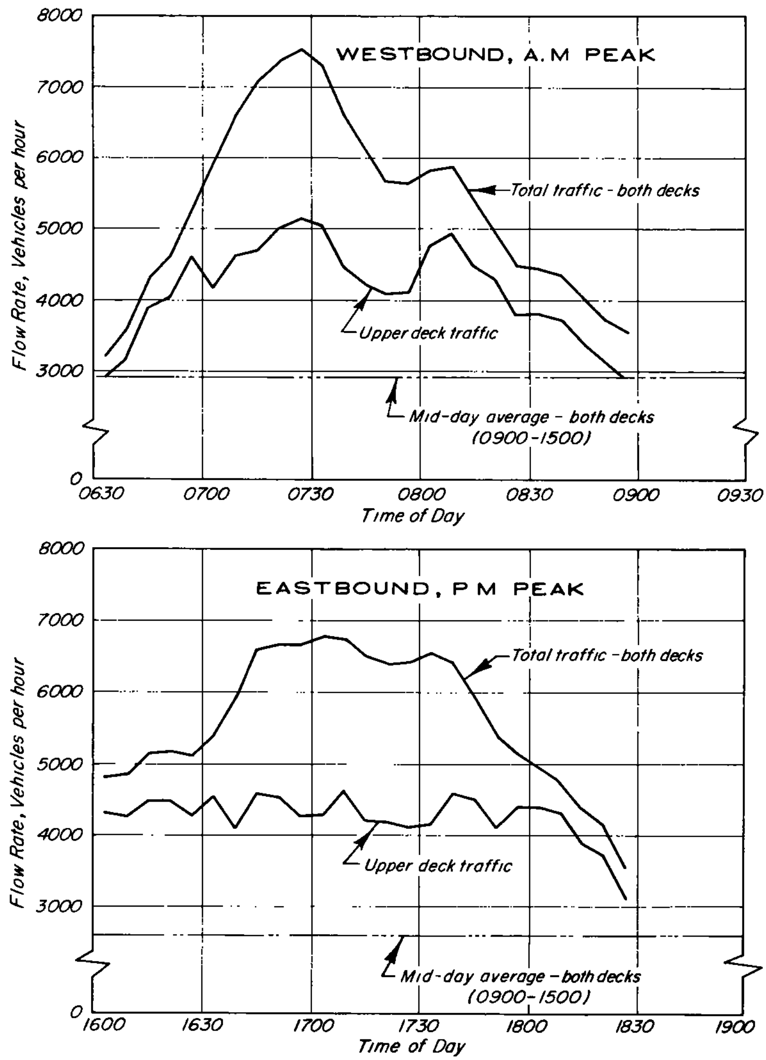

Figure 2. Peak period traffic flow, Bay Bridge.

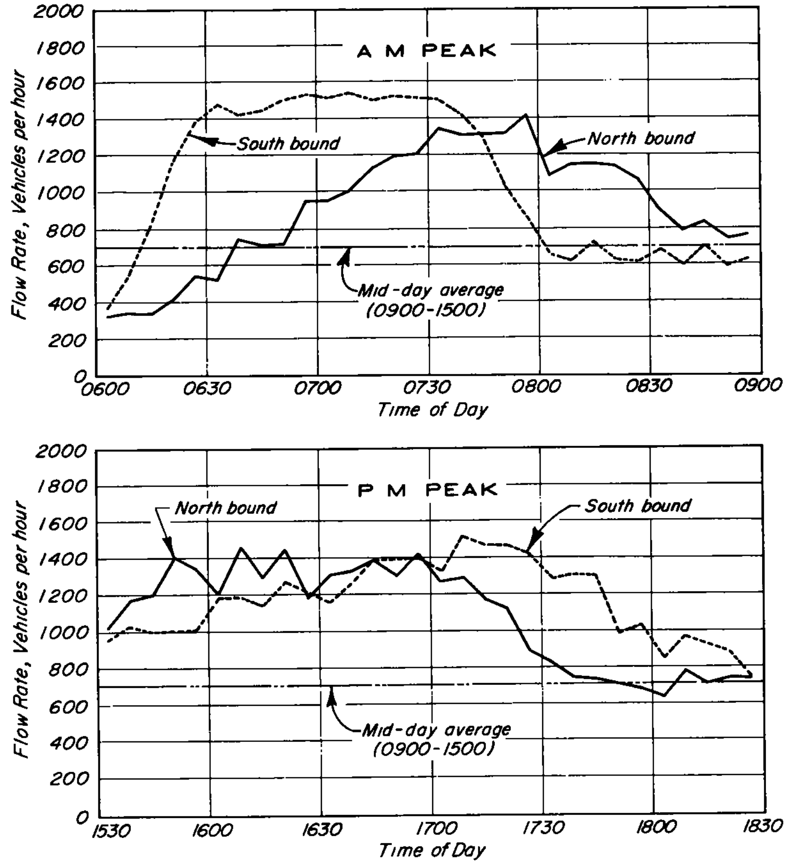

Figure 3. Peak hour traffic flow, Posey Tube.

It might be objected that the inclusion of volume data which are affected by stalls does not give a true picture of the maximum capacity of the Bridge. However, if the probability that accidents will happen is a function of the size of the total flow and the degree of congestion, then the capacity-reducing effect of accidents cannot be disregarded. It is certainly not disregarded by users of the Bridge—stalls of one kind or another are such frequent events that the possibility of being delayed by them undoubtedly influences driver decisions about when and whether to use the Bridge. Accidents tend to lengthen the peak period, as well as reduce the average rate of flow. Because individual delays cannot be foreseen by motorists, a peak accident causes a queuing of vehicles until the removal of the obstruction, then flow continues at a maximum rate beyond the time when it normally declines.

One attempt was made to measure the effect of accidents on Bridge capacity and the time pattern of traffic for the afternoon Bridge peak. Of the 19 days constituting the sample of the eastbound peak, there were 7 on which serious interruptions to the flow of vehicles occurred. These were deleted from the vehicle flow data, and the peak curve for the remaining 12 "accident-free" days was calculated.

At shortly before 5:00 PM, the average hourly rate of flow for the accident-free days was at 7,100 vehicles, compared with 6,700 vehicles for all 19 days. The difference narrowed until about 5:45 PM, when the curve for the accident-free days dropped below the curve for all days. From this time, the accident-free hourly rate was from 200 to 400 vehicles below the total curve until 6:30 PM when the counts ceased.

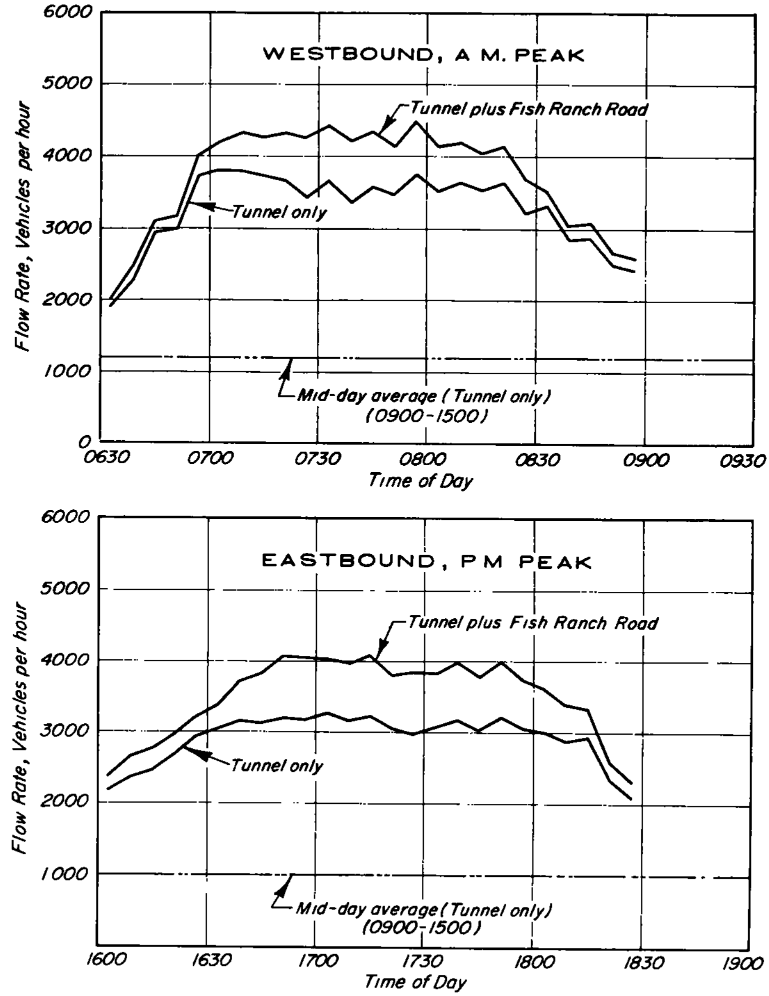

Figure 4. Peak period traffic flow, Caldecott Tunnel.

The "accident-free" days were not, of course, free from the normal stalls and minor delays which daily ration the flow of traffic. It seems reasonable to say that the effect of serious accidents, distributed over both accident and accident-free days, is a reduction of at least 5 percent in capacity; the total effect of all kinds of obstructions is a reduction of at least 10 percent in capacity. The latter conclusion is reached by comparing morning and evening flow rates.

The total volume curve declines slightly after reaching a maximum at 5:00 PM. It is believed that this was caused by the increase in the number of heavy trucks and buses on the lower deck after 5:00 PM; this subject will be further explored in the next section. Total volume fell abruptly after 5:30 PM when the lower deck was closed to passenger cars. The upper deck flow remained fairly constant until after 6:00 PM.

Posey Tube AM Peak.—Figure 3 shows an overlapping of the southbound and northbound peak flows on the Tube, but the southbound peak is largely finished by the time that northbound traffic reaches its maximum size.

Traffic headed south is moving out of Oakland, away from the central city core. The commuters are traveling mainly to the large naval air station and industrial plants in Alameda. In the opposite direction, the central district of Oakland is the principal destination.

Posey Tube PM Peak.—In the afternoon, the northbound peak, representing the departure of naval and industrial employees from their jobs, occurs first. From about 3:45 PM until after 5:00 PM, the Tube operates at full capacity in this direction, a rate of about 1,350 vehicles per hour. As southbound traffic increases to its maximum size, the northbound flow declines rapidly.

At their maximum, the peak volumes are no more than double the midday rates of flow, much less of an excess than at the Bay Bridge or the Caldecott Tunnel. One can imagine that if both peaks occurred in the same direction each morning and afternoon, the size of the combined peak would be very substantially greater—if road capacity permitted.

Caldecott Tunnel AM Peak (Westbound).—Unlike Posey Tube, the peak flow through the Tunnel is practically all in one direction. The morning traffic in the Tunnel reaches the capacity of the highway at 7:00 AM and continues at this rate until almost 8:30 AM. There is a constant queuing of cars at the eastern approaches during this entire period, which serves to divert vehicles to the inferior Fish Ranch Road. Even with the severe restraint of Tunnel capacity upon the peak rate, the maximum flow (including Fish Ranch Road) is 3½ times the midday rate, illustrating the importance of the Tunnel as a commuter facility and explaining why the peak percentages of daily traffic are exceptionally high despite extended traffic congestion.

Caldecott Tunnel PM Peak (Eastbound).—The Tunnel cannot operate at full capacity in the eastbound direction because there is no third traffic lane at the eastern end to absorb the vehicles from Fish Ranch Road. Therefore, the maximum flow, including Fish Ranch Road, is lower than in the westbound direction. The rate of flow, about 4,000 vehicles per hour, indicates the capacity of the two downhill lanes east of the Tunnel to move vehicles after Fish Ranch Road has merged.

The peak is spread out for a period lasting almost two full hours. One of the worst cases of traffic congestion in the Bay Area may now be found on the western approaches to the Tunnel. Long queues are a daily event on all approach roads.

It has actually been recorded, in each of the surveys, fewer total vehicles during the afternoon peak period between 4:00 and 6:30 PM than in the morning peak between 6:30 and 9:00 AM, including the Tunnel and Fish Ranch Road. This is most unusual. Typically the afternoon journey-from-work commuter travel is swollen by a variety of other travel purposes such as shopping, whereas the morning peak is almost exclusively a journey-to-work movement. The data suggest that the severe afternoon traffic conditions tend to divert nearly all travel which is not tied to employment hours to off-peak travel periods; it also may induce a number of commuters to use the extremely roundabout highways passing through the hills many miles to the north or south of the Tunnel.

OTHER DATA

A brief review of other information collected about various factors that affect total vehicular traffic volumes in peak periods is presented here. These are (a) the proportion of heavy vehicles, (b) the occupancy of automobiles, and (c) public transit riding.

Heavy Trucking

The presence of heavy vehicles in the traffic stream is an important factor in the capacity of the highways in this study because of the grades on all three facilities. The time and directional pattern of the heavy truck movement has some interesting implications for capacity standards.

As used in this section, the term "heavy trucks" refers to single-unit freight vehicles with more than 4 tires and to truck-trailer combinations. The lightweight trucks usually move fast enough to use no more road capacity than passenger cars, and a large number of them carry commuters rather than freight during the rush hours.

On all three facilities, the percentage of total vehicles that are heavy trucks drops considerably during the peak period. This is a common observation made on urban highways and needs no comment here.

Bay Bridge Trucking.—The Bridge's strategic location in the heart of a large metropolitan area, and as a link in a major intercity highway network, is responsible for a large volume of truck travel throughout the day. The peak rates of flow are shown in Figure 5.

In the morning, no sooner was the lower deck closed to automobile traffic than a sudden upsurge occurred in truck volume. Over a 20-min period, the rate of flow doubled and, at its maximum point, far exceeded the midday average. At 9:00 AM, the flow was declining; it leveled off at around 9:30 AM.

In the afternoon, the truck volumes rose in moderate degree just before the lower deck was opened to automobiles. Then, during the height of the automobile peak, the volume dropped sharply. By 5:00 PM, the volume had returned to the midday average—a fact which, combined with the increased volume of buses about this time, was probably the cause of the mild decline in total Bridge vehicular traffic shown in Figure 2.

After the lower deck was closed at 5:30 PM, there was a rise in heavy truck traffic. This time pattern was observed on each of the individual days on which classification counts were taken, but the curve in Figure 5 understates the magnitude of the fluctuation because it does not occur at exactly the same time each day. We averaged the minimum and maximum 6-min heavy truck volumes recorded on each day between 4:00 and 6:00 PM were averaged and found to range from 100 to 500 trucks per hour.

Congestion is costly to motor trucking, in wages and other operating expenses, and it is logical to find evidence that truckers try to avoid the worst of the peak travel conditions. In this particular instance, it does not appear that the afternoon fluctuation existed before the lower deck was made available to automobiles in 1953. At that time,

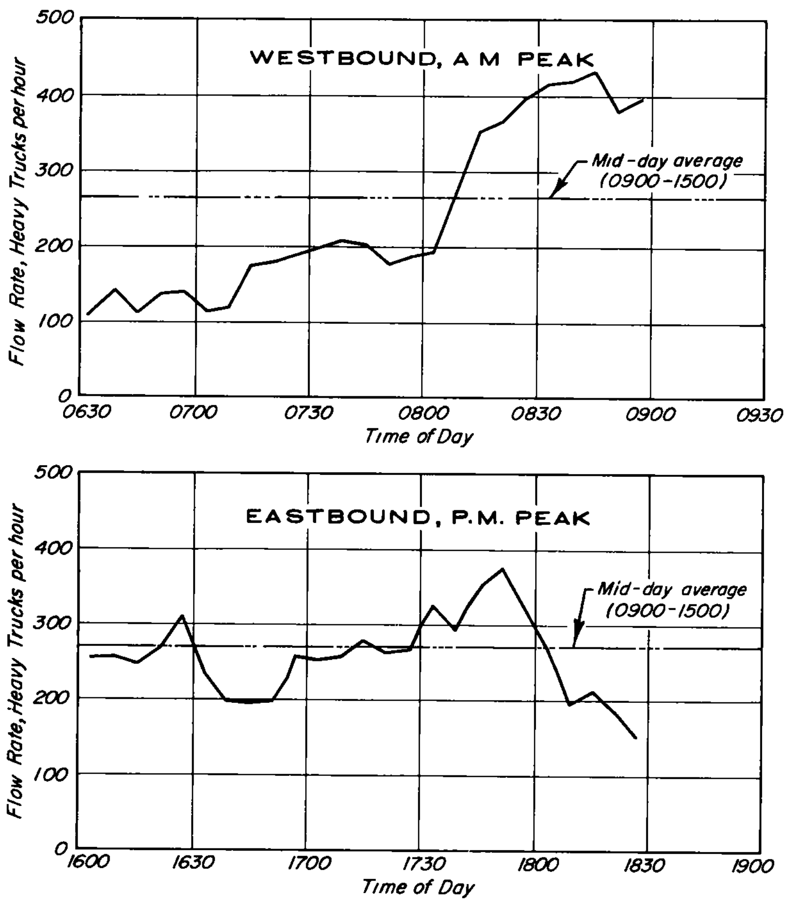

Figure 5. Flow of heavy trucks and combinations, Bay Bridge.

which was in the midst of a transit strike, the motor carriers voluntarily agreed to stay off the Bridge, if possible, during the maximum travel periods. Possibly this arrangement was continued unofficially after the strike was ended and the lower deck remained open to passenger cars.

Caldecott Tunnel Trucking.—On this suburban route, trucking is relatively light in volume; it is composed mainly of local commodity distribution. In the morning the majority of heavy trucks are moving outward from central city origins to the suburbs; in the afternoon they are returning. This movement is in the opposite direction to the primary direction of person trips.

At the rush hours, this directional difference is considerable. The average number of heavy trucks observed during the $1\frac{1}{2}$-hr periods in the morning and afternoon when Tunnel capacity is used to its absolute maximum in the peak direction was calculated. (Table 5). There are almost three times as many heavy trucks moving in the direction counter to the main vehicle peak flow as in the peak direction. Again, the conclusion seems warranted that rush-hour congestion discourages truck usage.

TABLE 5

AVERAGE NUMBER OF HEAVY TRUCKS USING CALDECOTT TUNNEL

Time Period	Heavy Trucks (avg.no.)	
	Westbound	Eastbound
7:00-8:30 AM	28	75
4:30-6:00 PM	83	31

Posey Tube Trucking.—There is no notable volume of heavy trucks through the Tube at peak periods. After 4:00 PM, the hourly rate of flow is less than one-half the midday rate in both directions.

Person Travel

The rate of automobile occupancy and the percentage of people using public transport determine how many persons travel in relation to the number of vehicles. Only short mention is made of the data collected on these subjects, pending a more thorough analysis.

Occupancy.—If there are fewer persons per car during the peak period, the size of the vehicle peak flow will be greater, or the spread of the peak wider. On the three roads in this investigation, occupancy samples were taken between 6:30 AM and 6:30 PM, and the peak period occupancy rate was generally found to be more than in the off-peak. But a notable difference between peak and off-peak rates was found only for the Bay Bridge.

The average occupancy over the entire 12-hr period on each facility is given in Table 6. For the peak direction, the Bridge rate rises to about 1.95 persons, both in the morning and in the afternoon peaks. Toll charges and the cost of all-day parking in San Francisco probably influence these rates.

TABLE 6

AVERAGE OCCUPANCY OVER ENTIRE 12-HR PERIOD

Test Area	Persons per Vehicle
Bay Bridge	1.67
Posey Tube	1.52
Caldecott Tunnel	1.46

On all three highways, the occupancy rate drops sharply in the later stages of the morning peak and does not regain the average level until after 10:00 AM.

Transit Riding.—The transit peak flow at the Bay Bridge occurs in the last half of the hour in which the highways are used by vehicles to their maximum capacity. Owing to this fact, the profile of persons movement presents much more the aspect of a peak than the curve for vehicle flow. On the Bridge, there is an actual increase in total person movement between 5:00 and 5:30 PM at the same time that the volume of vehicles, and the number of passengers in automobiles, is decreasing. At the Caldecott Tunnel, transit passenger counts were not feasible, but the record of bus movements there leads to the same finding in the afternoon peak.

There appears to be a fairly good coincidence between the transit and vehicular time patterns on Posey Tube in both directions. However, in the morning there is a much larger volume of persons on transit traveling northbound, into Oakland from Alameda, than in the opposite direction. In the afternoon, the outflow, southbound, from Oakland is considerably larger than the northbound travel from Alameda. Thus, although the total volume of vehicles over the whole peak period is not greatly different in each direction, the primary movement of persons is in the direction of Oakland in the morning and away from Oakland in the afternoon.

CONCLUDING REMARKS

This presentation has been largely descriptive. It was meant to show the peak patterns of vehicle movement on highway facilities that are utilized to their fullest capacity over lengthy time intervals during the regular morning and evening urban commuter rush periods. At this stage of the study, there is no wish to comment on the possible changes that might occur when the ceilings on road capacity are raised; however, several suggestions may be offered about the effects of sustained traffic congestion on the spread of the peak and the total volume of peak period traffic.

1. Vehicle congestion has caused a rather considerable use by passenger cars of "inferior routes" on the Bridge and at the Tunnel. The upper deck of the Bridge, which was intended to carry all automobile traffic, is almost completely filled in the peak direction from before 7:00 until after 8:00 AM, and from 4:00 until after 6:00 PM. Tunnel capacity is fully used for approximately the same lengths of time during the rush hours.

   It is difficult to imagine how far peak usage might have spread if neither the lower deck of the Bridge nor Fish Ranch Road had been available to absorb some of the traffic excess. These alternatives have made additional vehicle peak volumes possible; and it is reasonable to think that if they were not open to traffic, the total number of vehicles appearing during the peak period would be lower, particularly on the Bay Bridge.
2. Stalls and delays associated with intense vehicle congestion act to spread the peak by causing sporadic queuing of cars. They may also reduce the total peak volume on the Bay Bridge because they are frequent occurrences whose probability enters into driving decisions. It seems important that any concept of "possible" highway capacity include an allowance for the frequent repetition of delays and stalls as one of the average conditions of extreme traffic congestion.
3. The peak period data suggest that extreme congestion tends to shift travel in some trip-purpose categories. This seems evident for motor trucking, for whom the costs of congestion are immediately apparent in monetary values. An absolute decline in truck flow, as well as a reduction in the percentage of trucks in the total flow of vehicles, has been found on all three of the study facilities during the maximum vehicle peak.
4. It should be recognized, however, that there is a natural spread of the peak which is fairly independent of transport media. For example, factor employees tend to arrive and leave their jobs earlier, and retail store employees later, than the average. It is likely, too, that the spread of the peak has been encouraged by the convenience of commuting by automobile, as compared with public transit. These inferences seem warranted by the different times of the northbound and southbound peaks at the Posey Tube and by the relation of the peak in transit passengers to the vehicle peak found on the Bridge.

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1. Webb, G. M., and Moskowitz, K., "Freeway Capacity Studies 1956." Calif. Division of Highways, Sacramento (1957).

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