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Subject Area

22 Highway Design
53 Traffic Control and Operations

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Foreword

The increasing number of completed miles of Interstate and other freeway types of highway have turned the attention of traffic operations and design personnel to the specialized field of traffic operations. What was once thought to be only a unique problem in a few scattered areas is becoming almost an everyday occurrence, i.e., the breakdown in traffic-carrying ability of freeways as heavy volumes exert their influence.

Research has been under way for many years to gain insight into the alleviation of freeway driving problems as well as other aspects pertinent to limited-access highways such as the need for increased communications for users. The Highway Research Board Committee on Freeway Operations was created so that those interested in this then-exotic topic could have a common area of interest. This RECORD is the result of some of the committee's work in the past year; it presents four papers concerned primarily with troublesome aspects of operating freeways to the satisfaction of the user.

The first paper, developed by a researcher at the Illinois Freeway Surveillance Project, reports on experiments with ramp metering devices. Favorable freeway operational benefits were found, such as reduced congestion, decreased travel times, increased speeds and flows and less troublesome "bottleneck" conditions. The placing of signals on ramps in producing the benefits increased delays at streets near the freeway and the author cautions against benefiting freeways to the extreme detriment of city streets.

The second report, using studies tested in Atlanta, evaluates the effectiveness of roadside radio communication on driver behavior. The research indicates that audio messages were as effective as visual messages and when used jointly, driver performance was better than for either type of message used alone.

Two New York researchers give an evaluation of an emergency telephone system in the third paper. Some 55 miles of rural Interstate highway in New York with such telephones were studied. It was found that false alarms were negligible and that over 75 percent of those using the emergency phones needed fuel or vehicular repairs. A trend indicating decreased use of phones with increasing traffic volume was evident.

The last paper reports research by four Texans on nationwide practices in freeway merging operations. Use of aerial photography made possible rather complete analyses. The effects of the various geometric elements as measured by factors such as volume, density, speed, and acceleration noise are shown.

This RECORD will be of prime interest to highway administrators, traffic operations personnel, designers, and those concerned with the everyday problems of congestion on freeways. Police and other agencies responsible for responding to emergencies will find some of the material to be of considerable help.

Contents

Operational Effects of Automatic Ramp Control On Network Traffic

JOSEPH M. McDERMOTT, Traffic Research Engineer, Chicago Area Expressway Surveillance Project

•ONE of the major features of the National System of Interstate and Defense Highways is the physical control of traffic access provided by grade-separated interchanges. Traffic research by the Chicago Area Expressway Surveillance Project of the Illinois Division of Highways has indicated that further control of access may be needed, particularly in urban areas, to prevent or reduce congestion caused by traffic demands in excess of expressway operational capacity (1, 2).

To adjust entrance ramp traffic volumes to the available expressway capacity, the Project developed an electronic ramp metering device which automatically controlled the magnitude and rate of traffic entering an expressway. The first successful use of modified traffic signals to automatically meter one vehicle at a time onto an expressway was initiated in September 1963 at an entrance ramp on the outbound Eisenhower Expressway in suburban Chicago (3).

Extension of the ramp metering technique to three additional entrance ramps in June 1965 produced a continuous expressway control system 2.5 miles long, designed especially to reduce expressway traffic congestion caused by overloading a sensitive

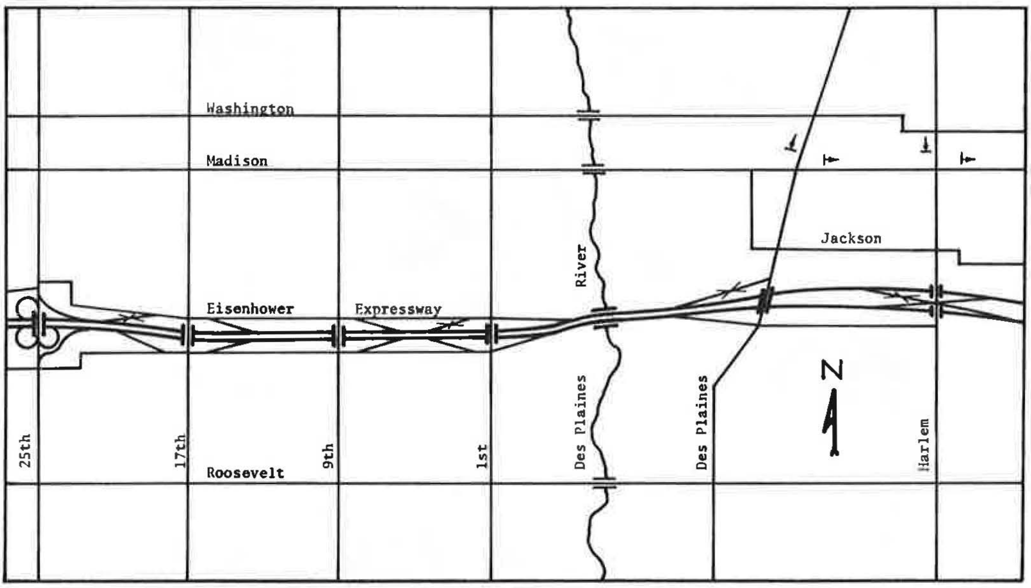


Figure 1. Locations of entrance ramp control signals and informational display signs.

Paper sponsored by Committee on Freeway Operations and presented at the 46th Annual Meeting.

geometric section. As a supplementary measure, because ramp metering devices generate ramp queues and traffic diversion, four display signs were installed at strategic surface street locations to provide some expressway-bound motorists with advance advisory information concerning existing ramp and expressway traffic conditions.

This report summarizes the overall operational effects on both expressway and surface street traffic flow produced by the automatic control system of four entrance ramp metering devices and four supplementary informational display signs. The report by no means contains all the ramp control research findings produced by the Project, but highlights areas of special interest to freeway operating and research engineers.

CONTROL STUDY DETAILS

The pilot expressway control system consists of entrance ramp metering signals at four successive outbound (westbound) Eisenhower Expressway (I-90) diamond interchanges: Harlem, Des Plaines, First and 17th Avenues. The four supplementary informational signs are located on the southbound and westbound approaches to two major surface street intersections handling expressway-bound traffic (Fig. 1).

Equipment Characteristics

The modified (red-green) traffic signals which comprise each ramp metering device maintain a constant green indication in non-peak periods. During "rush" periods (3:30

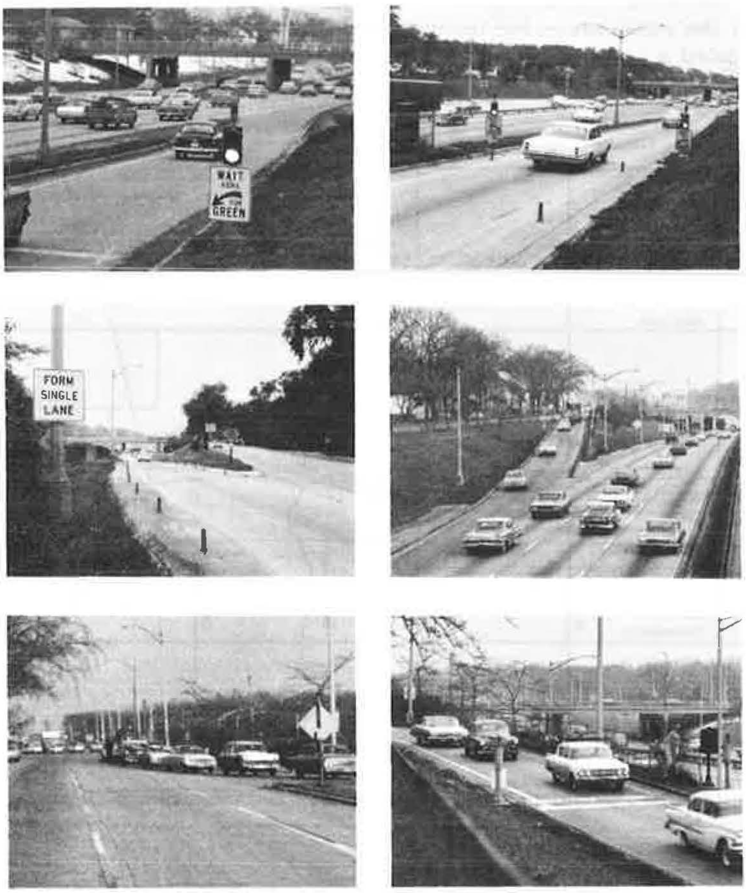
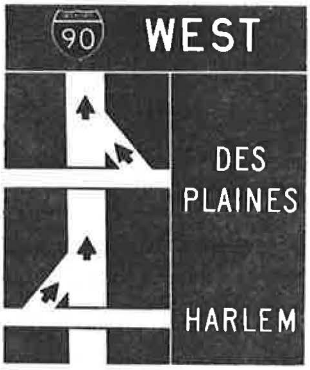


Figure 2. Ramp metering operations at First Avenue.



Westbound Madison
approaching Harlem

Southbound Des Plaines
approaching Madison

Figure 3. Informational display signs.

to 6:30 p.m. on commuting weekdays), the metering signals at each ramp rest in red and change to green for each ramp vehicle at minimum intervals ranging between 5 and 15 seconds. Thus, queued entrance ramp vehicles are released one-at-a-time into the expressway traffic stream (Fig. 2). Prevailing metering rates are automatically selected through a dectector-computer system which decreases entrance ramp flow rates

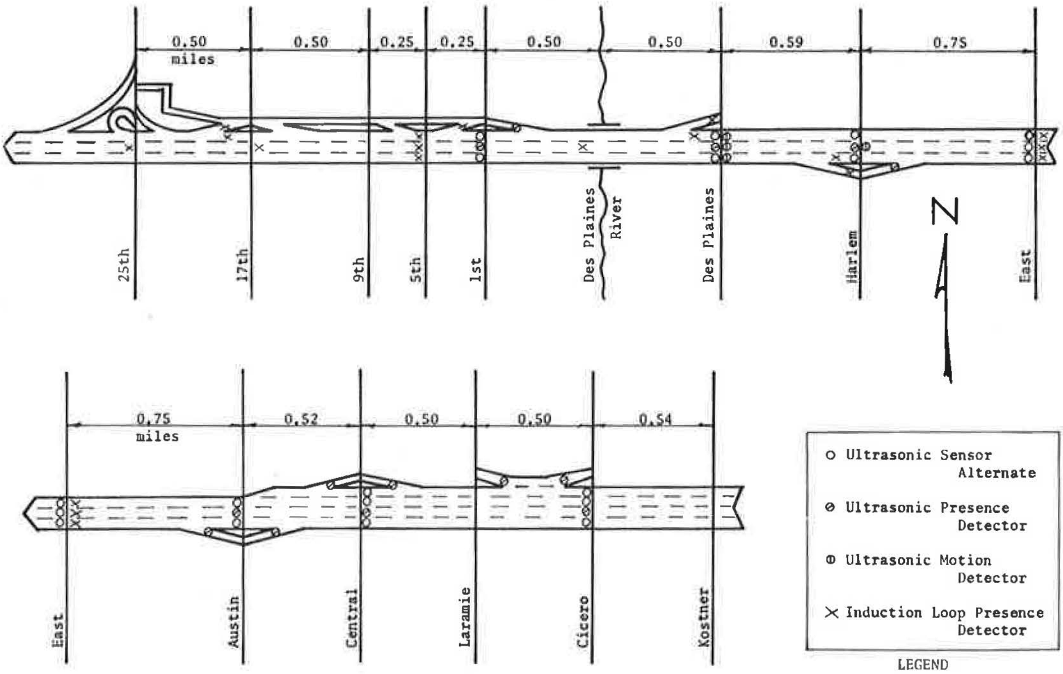


Figure 4. Electronic surveillance system on the westbound Eisenhower Expressway.

as measured expressway traffic conditions approach congestion levels (3, 4). Motorists disobeying red ramp signals trigger a violation alarm bell; signals of this type are enforceable under Illinois state law. Modified traffic signals of comparable appearance are in common use throughout the Chicago area at automatic toll collection stations.

To promote diversion to alternate routes when outbound expressway traffic conditions warrant, the supplementary informational signs display existing entrance ramp and expressway delays to some potential expressway users approaching or traveling on an arterial street serving as a major parallel expressway alternate. Each sign shows traffic conditions at the two nearest outbound expressway interchanges (to the west) by color-coded, changeable arrows: GREEN = NO DELAY; YELLOW = MODERATE; RED = CONGESTED (Fig. 3). If conditions at the nearest interchange are congested, motorists can use westbound Madison Street or other routes for travel to the next informational sign or to a more attractive ramp, thereby tending to decrease peak-period queuing behind ramp signals (5).

The ramp metering signals and supplementary informational signs are automatically controlled from continuous expressway and ramp traffic measurements. Since October 1962, expressway and ramp traffic has been monitored by an electronic detection system, which now covers a 6-mile outbound Eisenhower Expressway section between Cicero and 25th Avenues (Fig. 4). A total of 45 detectors provide inputs for the analyses of traffic operations on most ramps and at average expressway intervals of about ½ mile. A small, real-time, digital control computer and several analog computers in the Project office process detector data into system surveillance, control, and evaluation outputs (6). An office control console collates displays of expressway traffic conditions, prevailing metering rates, ramp signal changes and violations, sign conditions, and other information useful to the control observer.

Bottleneck Control Theory

The primary objective of the ramp control system is the prevention of expressway traffic congestion caused by overloading the three-lane Harlem Avenue to 17th Avenue section of the westbound Eisenhower Expressway. Should congestion develop despite ramp controls, or for reasons not attributable to entrance ramp turbulence, the secondary objective of the ramp control system is the restoration of non-congested traffic flow.

Each ramp metering device operates over the metering range from the maximum rate of 12 vehicles per minute (vpm) to the minimum of 4 vpm. As non-congested expressway flows rise towards maximum volume, the ramp flow rates are gradually decreased, thereby attempting to prevent the development of congestion from high ramp flow rates merging with high expressway flow rates. In the congested expressway situation, the minimum metering rate of 4 vpm is employed until non-congested operations return (3, 4).

The control section was selected to encompass the final outbound expressway bottleneck. A recurrent problem of traffic congestion existed east of the Des Plaines River bridge, where the otherwise depressed expressway rises to an at-grade cross section for passage over the Des Plaines River. The interplay of traffic demand and sensitive geometrics usually produced congested operations (from 4:30 to 6:00 p.m. each commuting day) which regularly extended upstream into the next major bottleneck, aggravating congestion where the expressway pavement reduces from four to three lanes (Austin Avenue).

Metering controls at the Harlem and Des Plaines entrance ramps were primarily intended to prevent bottleneck congestion. However, the sensitive geometrics in this area complicate the control problem. Peak-hour volumes of 550 vehicles per hour (vph) and 675 vph at the Harlem and Des Plaines entrance ramps, respectively, were loaded successively from the left and right sides of the expressway where a reverse curve, three closely-spaced overpass structures and a 3 percent upgrade decrease capacity.

Nevertheless, it was quite evident that Harlem and Des Plaines entrance ramp traffic often caused the introduction of congestion, whereas the geometrics were suspected of maintaining the congestion once it had developed. Although no overloading congestion existed anywhere downstream of the Des Plaines bottleneck, the downstream entrance ramp controls at First Avenue and 17th Avenue were deemed necessary to retain non-congested traffic flow west of the Des Plaines River bridge. It was anticipated that bottleneck improvements might increase flows through the downstream corridor and/or upstream ramp controls might divert traffic to downstream entrance ramps, thereby making it desirable to prevent the occurrence of congestion at locations previously trouble-free.

Evaluation Methods

Inasmuch as the ramp control system was expected to produce positive expressway benefits at the expense of some negative benefits on the contiguous surface streets and metered entrance ramps, the whole network traffic operations were analyzed for the 3:30 to 6:30 p.m. peak period before and after commencement of the ramp control system on June 21, 1965.

Expressway traffic measurements consisted basically of automatic digital computer loggings each minute of volume and occupancy for 36 detectors in the surveillance system. Manual expressway and ramp volume counts were used to supplement automatic detector data and to check detectors. Basic calculations of daily expressway performance were programmed to yield vehicle-minutes of travel time and vehicle-miles of travel. Individual travel-time samples were taken by matching license plates for certain-colored Volkswagens passing time-and-license plate recording stations. These travel times were utilized in one method of computing total travel time; an input-output technique, however, proved more efficient for data collection, analyses, reliability and accuracy (7).

Surface street traffic measurements were more difficult to obtain due to the permeable nature of a grid street system. Machine counters were employed to establish intersection approach patterns and to supplement manual intersection counts for 12 critical arterial intersections (8). Intersection approach delays (and entrance ramp queue delays) were estimated by recording the number of vehicles within a certain trap length every minute. Floating-car travel time runs were also made over numerous surface street routes.

EXPRESSWAY IMPROVEMENTS

Expressway operational measurements were recorded for 33 peak traffic periods both before and after the commencement of the ramp metering control system. Between 3:30 and 6:30 p.m. data were collected over a 5.40-mile expressway section from Kostner Avenue to 5th Avenue. Although the 2.59-mile East Avenue to 5th Avenue section comprises the basic control section, because traffic improvements over this distance could be directly attributed to the ramp control system, the longer data collection section extending back to Kostner Avenue was necessary to estimate the incremental control benefits occurring upstream at the Austin Avenue lane reduction bottleneck.

Extent of Congestion

A contour map showing the extent of expressway traffic congestion over time and distance was composed from the five best operational peak periods both with and without the ramp control system (Fig. 5). Congestion was defined as sustained traffic operations at reduced speeds, evidence by increased expressway lane occupancies accompanied by decreased lane volumes (volume-to-occupancy ratio less than 1.0).

Although the ramp control system did not completely eliminate congestion in the control section, both the severity and dimension of congestion were significantly reduced. The period of congestion on the Des Plaines upgrade was decreased 45 minutes (from 75 minutes to 30 minutes); the introduction of congestion was delayed by 35 minutes and recovery occurred 10 minutes earlier. The severity of congestion in the control section

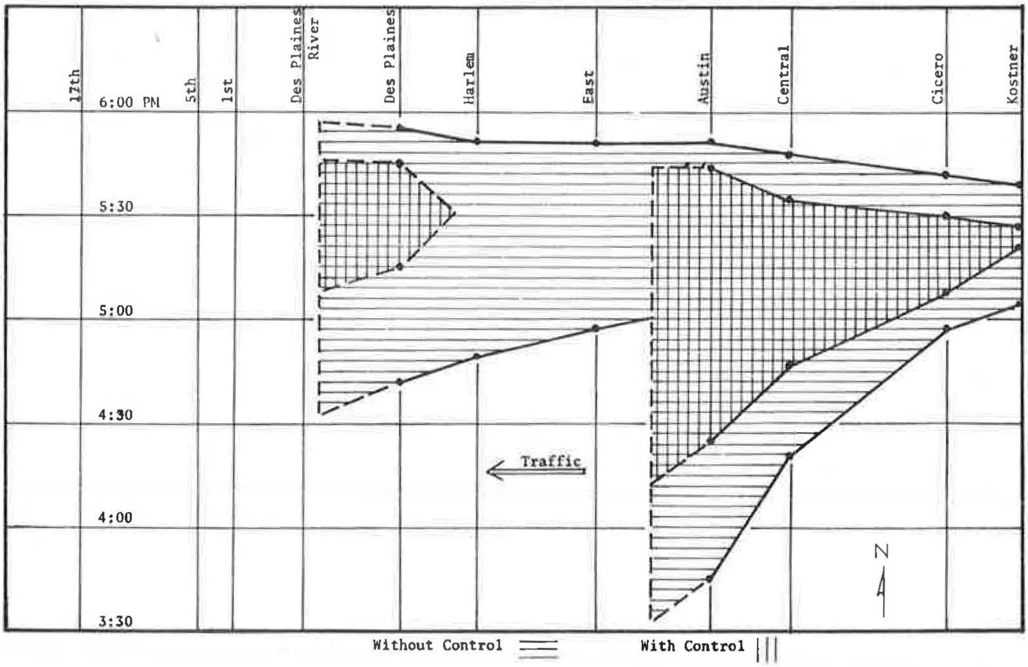


Figure 5. Expressway congestion contour map.

was reduced such that congested operations were confined to the immediate Harlem-Des Plaines area, a condition indicating expressway demands near the congested flow rate. As a result, the Des Plaines backup was prevented from influencing traffic operations at the upstream Austin bottleneck.

The improvements displayed at the Austin bottleneck, however, cannot all be assigned to the ramp control system, inasmuch as the "before" backup from the Des Plaines bottleneck did not compound Austin congestion until about 5:00 p.m. The changes east of Austin Avenue before 5:00 p.m., therefore, must reflect changes in traffic demand due to a redistribution of travel patterns and/or normal seasonal traffic variations. It should be noted in the contour map that congestion reductions were recorded well upstream. The improvements transmitted beyond the upstream data collection limits unfortunately went unmeasured, thereby lending a conservative tone to the measured changes.

Total Travel Time

The input-output technique for estimating total expressway travel time for all vehicles (the sum of all individual travel times) produces a plot of vehicles in the expressway section vs clock time. The area under this curve represents the total travel time of all vehicles in the section for the time period under consideration (7). Curves were plotted for two expressway sections (the control section and the upstream section) from the five best operational peak periods in each study phase.

The East-to-5th control section (Fig. 6) clearly shows direct control benefit between 4:35 and 6:20 p.m.; the area between the two curves represents a reduction in total travel time of 186 vehicle-hours. As noted previously in the congestion contour map (Fig. 5), control benefits are not realized until after 4:50 p.m. in the Kostner-to-East upstream section (Fig. 7). By the prevention of backups from the Des Plaines bottleneck between 4:50 and 6:10 p.m., however, an additional 110 vehicle-hours are saved.

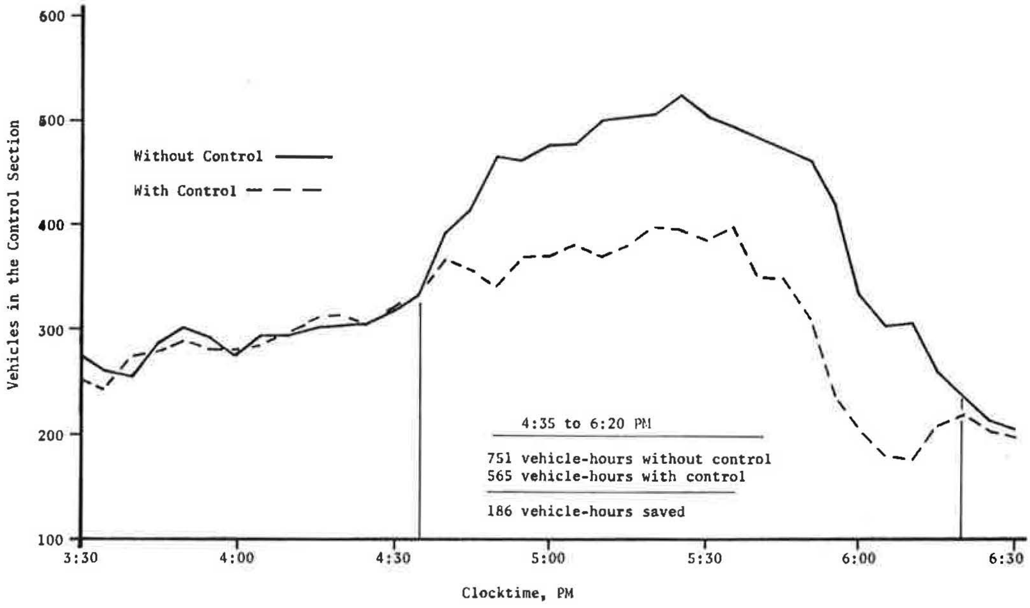


Figure 6. Number of vehicles in the control section (East to 5th).

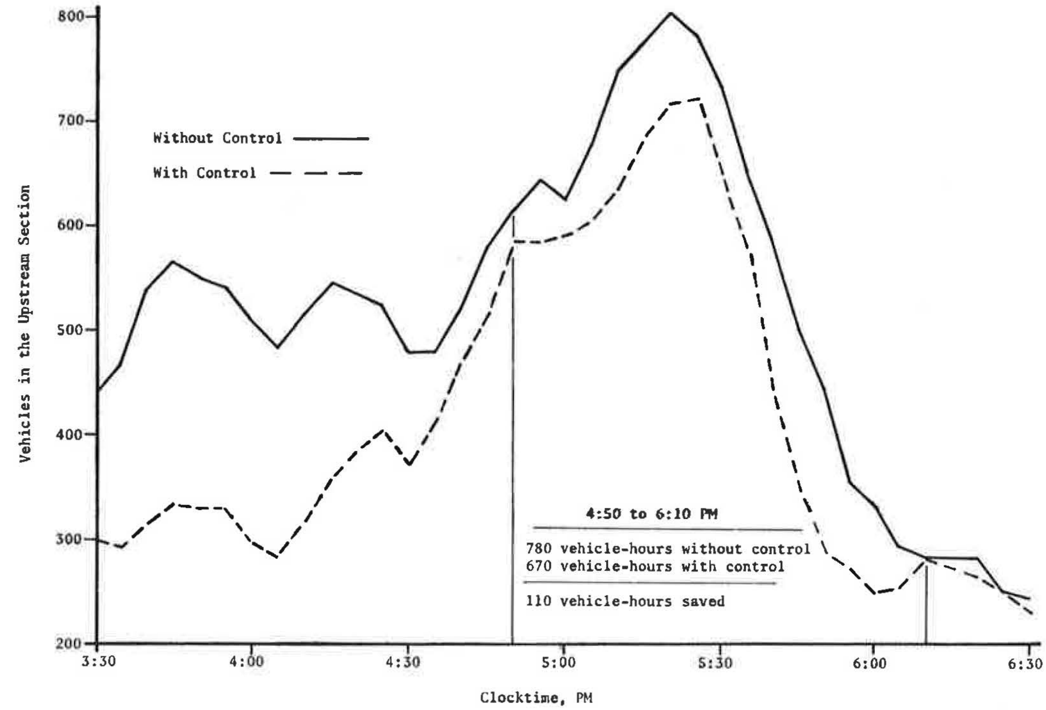


Figure 7. Number of vehicles in the upstream section (Kostner to East).

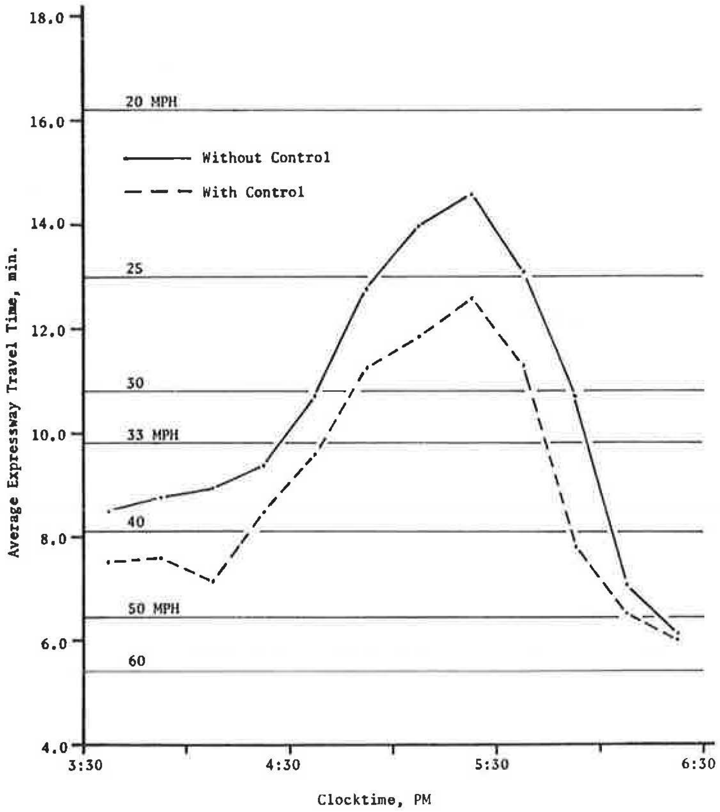


Figure 8. Individual trip travel times, Kostner to 5th (5.40 miles).

The total travel time changes prior to 4:50 p.m. in the upstream Kostner-to-East section cannot be attributed to the ramp control system unless expressway travel patterns were altered by control, an unlikely occurrence. Normal spring-to-summer, peak-period traffic demand decreases apparently produced most of these travel time changes. Slight demand reductions at the Austin lane reduction bottleneck seem ample to delay the introduction of peak-period congestion there in the summer months.

During the effective control period (4:35 to 6:20 p.m.), the total travel time savings due to ramp controls represent actual reductions in individual trip travel times, since the vehicle-miles of expressway travel were maintained or slightly increased. The direct control section benefits (186 vehicle-hours) combined with the incremental upstream Austin bottleneck section benefits (110 vehicle-hours) produce total ramp control benefits amounting to 296 vehicle-hours of reduced total travel time for expressway users between Kostner and 5th Avenues (5.40 miles).

Individual Travel Times

Since total travel time reductions are not evenly distributed among all expressway users, plots of actual trip travel times vs clock time provide estimates of control benefits experienced by individual motorists. Ramp controls produced trip travel time savings of up to 3 minutes for through (Kostner-to-5th) expressway users in the effective control period (Fig. 8), comparing the best operational peak periods with and without control.

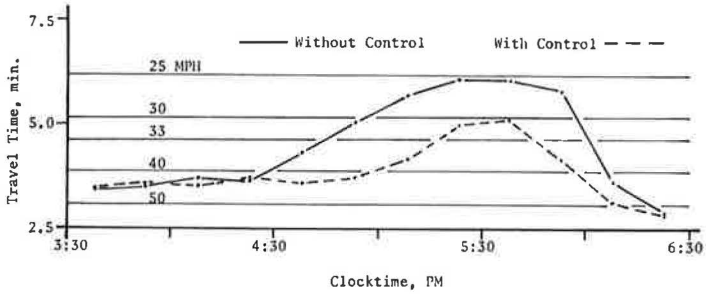


Figure 9. Control section trip travel times, East to 5th (2.59 miles).

The individual trip travel time graphs for the control section (Fig. 9) and the upstream section (Fig. 10) agree in general shape and overall magnitude with the total travel time graphs shown previously (Figs. 6 and 7), although the data collection and analysis techniques were basically independent. Favorable agreement with the congestion contour map (Fig. 5) is also possible by defining congestion on the trip travel time graphs as traffic operations at space mean speeds less than 33 mph, a quite reasonable definition.

Expressway Production

Regarding the effect of ramp controls on expressway traffic volumes, two areas of major interest are the actual expressway bottleneck section and the upstream expressway section. The upstream station at East Avenue (Fig. 11) shows non-congested volumes with ramp controls and congested volumes (beginning at 4:50 p.m.) without controls. The demand entering the control section was essentially the same on the best operational days prior to 4:50 p.m.

The prevention of the congestion backup from the Des Plaines bottleneck produced an increased expressway flow of 315 vph at East Avenue between 4:50 and 5:50 p.m. The volume differences after 5:50 p.m. are meaningless from a production standpoint, since the mainline demand at East Avenue decreases (non-congested flows with control),

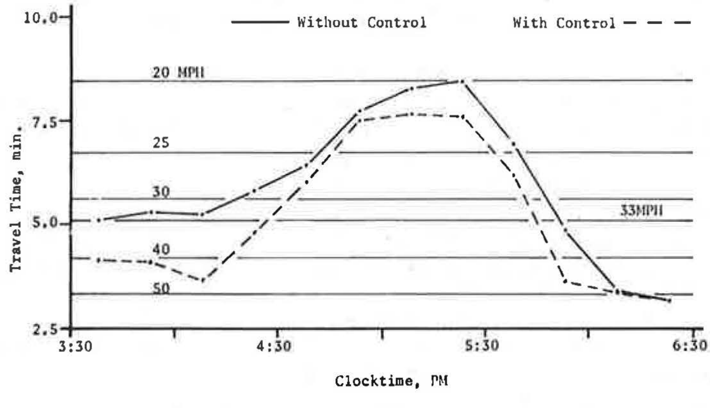


Figure 10. Upstream section trip travel times, Kostner to East (2.81 miles).

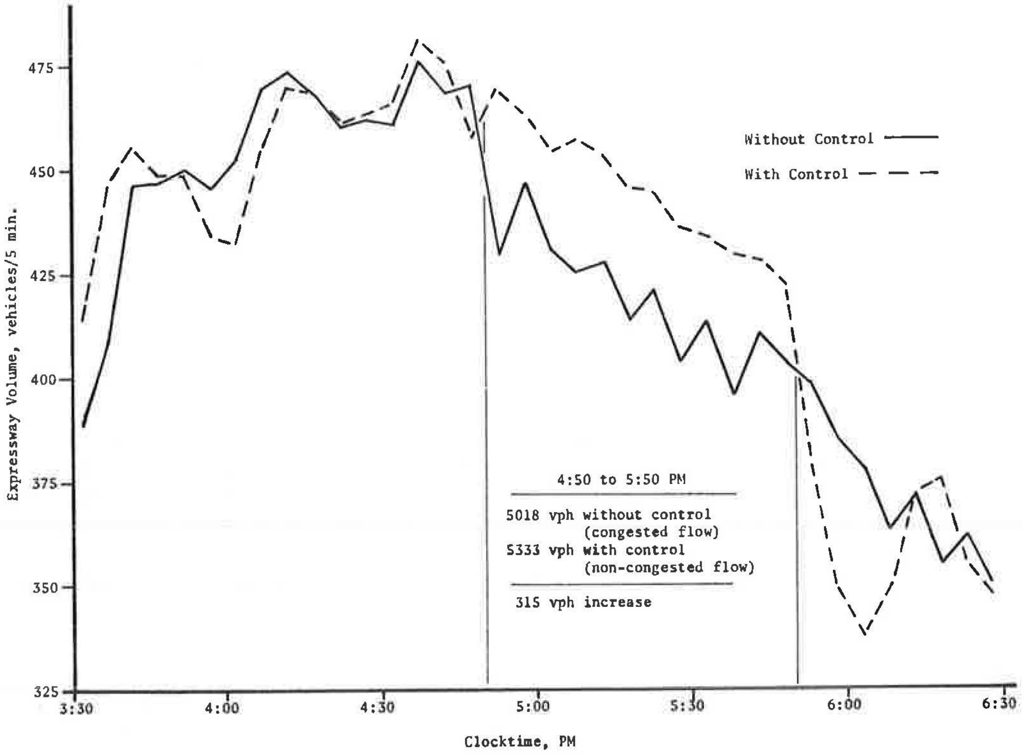


Figure 11. Expressway volume at East Avenue.

whereas the "before" volumes represent continued discharge from upstream congestion storage, a condition prevented by ramp controls.

The actual bottleneck output flow at the Des Plaines River (Fig. 12), meanwhile, decreased 49 vph in the critical control hour there (4:40 to 5:40 p. m.), indicating that upstream ramp controls do not necessarily increase long-term bottleneck outputs. The ramp controls, however, may have been slightly conservative from a bottleneck capacity standpoint.

A flow map for the critical control hour in the East-to-5th section (Fig. 13) shows the overall production effects produced by entrance ramp metering at Harlem and Des Plaines. Diversion of 325 vehicles between 4:45 and 5:45 p. m. at these ramps reduced expressway density and increased upstream flows without greatly changing the flow through the bottleneck and downstream. (Flow maps of this type require inclusion of density changes.) Under reduced expressway density conditions, the 325 diverted ramp vehicles were essentially replaced by 265 expressway vehicles (formerly stored on the congested upstream expressway) in the use of the expressway bottleneck in the ciritical control hour. Perhaps improved ramp control schemes will allow one expressway vehicle to replace each diverted ramp vehicle in passage through the Des Plaines bottleneck. It should be noted that the total outbound corridor output was increased, as diverted vehicles were absorbed on alternate routes.

The upstream flow increases generated through ramp controls were reflected through the Austin lane reduction bottleneck to the upstream study section limits at Kostner Avenue (Fig. 14). This graph also shows the slight demand decreases responsible for the delay in the introduction of congestion at Austin Avenue.

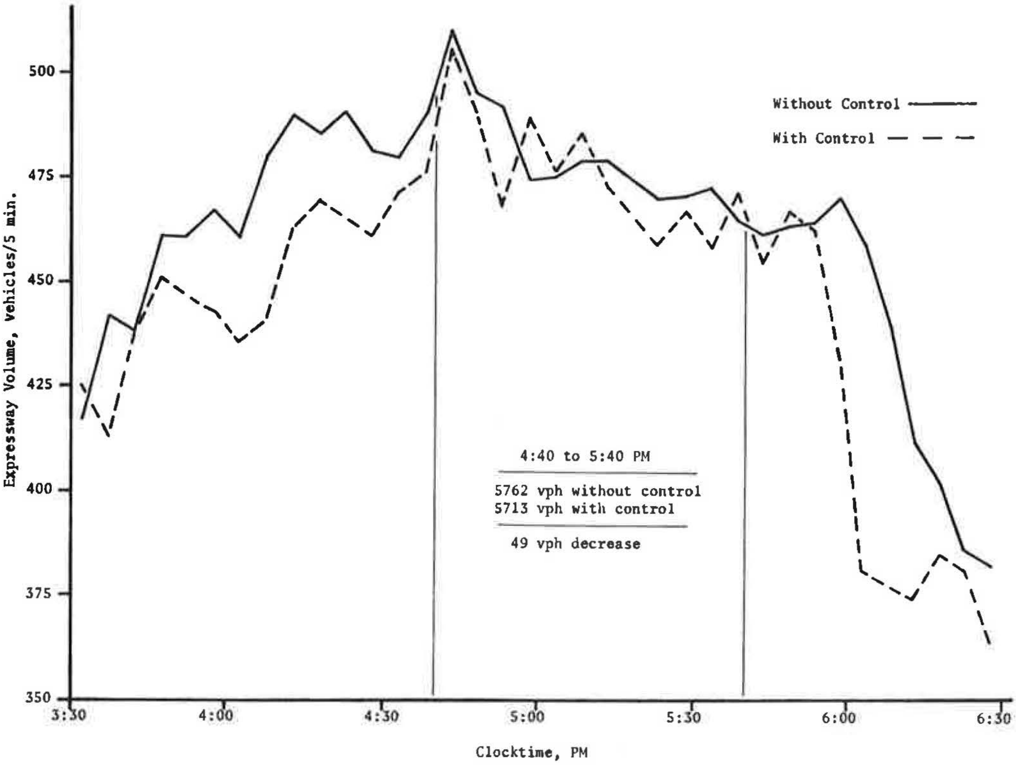


Figure 12. Expressway volume at Des Plaines River.

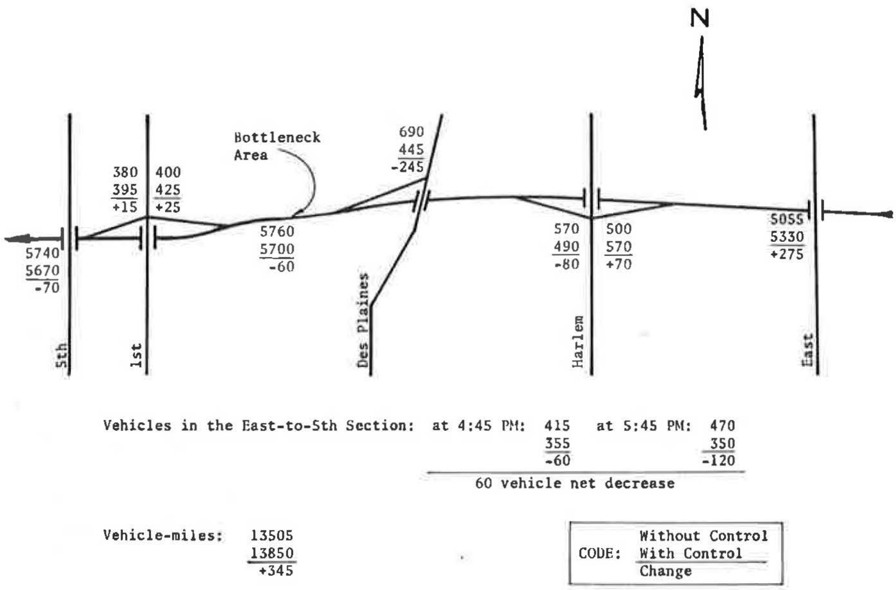


Figure 13. Expressway and ramp flow map, East to 5th, critical control hour (4:45 to 5:45 p.m.).

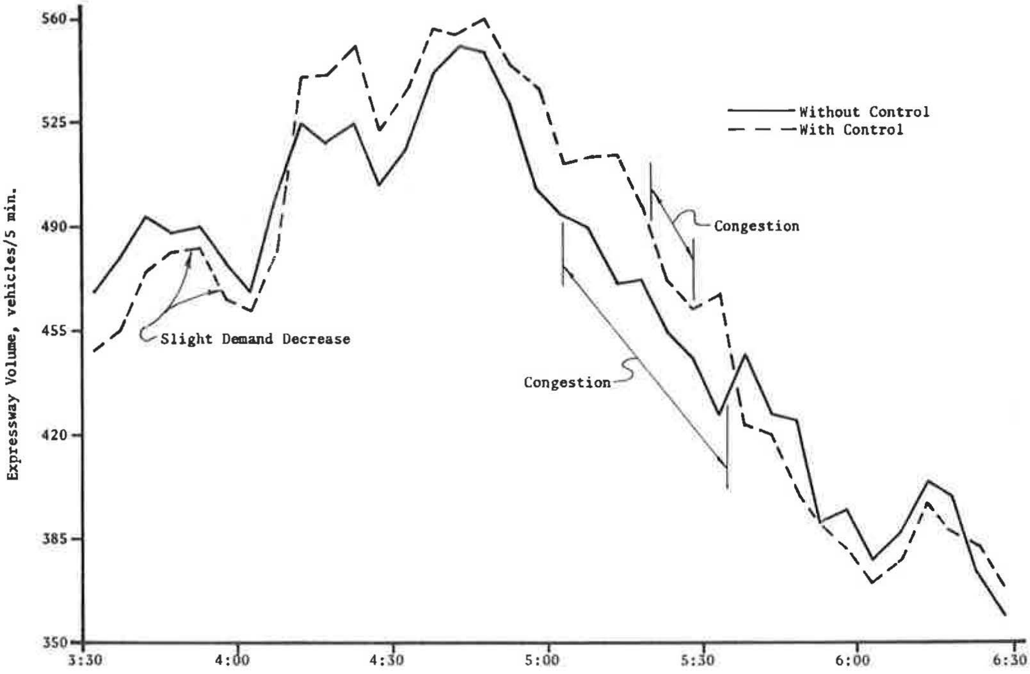


Figure 14. Expressway volume at Kostner Avenue.

Daily Performance Variations

It should be emphasized that the expressway findings presented thus far were derived by comparing only the best peak periods with and without ramp controls. Detailed analyses for all study days, however, demonstrated that the expressway does not operate at the same level each peak period. Even apparently comparable peak periods, free of obvious incidents and traffic disturbances, produced substantially different performance data, apparently due to subtle minor events, occurring within or outside the study section, which also complicate the basic expressway overloading problem.

The 1962 stopped vehicle study, for instance, pointed out that an overall average of one vehicle was stopped on the pavement or shoulders each hour per directional mile for some reason other than traffic congestion (9). Applying this overall factor to the 5.40-mile study section (even though the peak-period factor is somewhat higher), an average of more than 30 stopped vehicles in both directions could disturb traffic flow each outbound peak period (3:30 to 6:30 p.m.). Additional stoppages upstream or downstream of the study section could also cause backups and/or reduce flows within the data collection section.

It is not surprising, therefore, that the daily overall peak-period performance data exhibit a rather erratic pattern (Tables 1 and 2). The calculated vehicle-miles and vehicle-hours produce an overall "speed" parameter (vehicle-miles/vehicle-hours), which serves as a very useful indication of the operational level for each peak period. Ranking each peak period in descending operational levels (Figs. 15 and 16) shows that there were about ten peak periods with ramp control which were operationally better than the best peak period without control. The poorest peak periods are affected by the randomness of special expressway events (accidents, disabled vehicles, rain, etc.)

TABLE 1

OVERALL EXPRESSWAY PERFORMANCE WITHOUT CONTROL
(3:30 to 6:30 p.m.)

Date	Day	Kostner to Fifth (5.40 mi)			East to Fifth (2.59 mi)		
		Veh-Miles	Veh-Hours	"Speed"	Veh-Miles	Veh-Hours	"Speed"
4-27	Tu	—	—	—	40661	1117	36.4
4-28	W	—	—	—	38855	1150	33.8
4-30	F	75449	4403	17.1	35949	1157	31.1
5-3	M	78994	3804	20.8	37111	1446	25.7
5-4	Tu	84453	3243	26.1	39372	1034	38.1
5-5	W	83702	2788	30.0	—	—	—
5-6	Th	84406	2798	30.1	39663	1150	34.4
5-10	M	82834	3292	25.2	39269	1165	33.7
5-11	Tu	84321	2983	28.3	39822	1198	33.2
5-12	W	84392	2634	32.0	39580	1018	38.9
5-13	Th	75534	4185	18.1	35547	1656	21.5
5-14	F	82719	2660	31.1	39036	1234	31.6
5-17	M	86365	3636	23.7	40604	1324	30.7
5-20	Th	82761	2667	31.0	38732	1068	36.2
5-21	F	85596	2659	32.2	40093	1151	34.8
5-25	Tu	84579	2616	32.3	—	—	—
5-26	W	81693	2964	27.6	—	—	—
5-27	Th	82575	2672	30.9	—	—	—
6-4	F	85359	3140	27.2	40052	1335	30.0
6-7	M	86252	2845	30.3	40668	1152	35.3
6-9	W	84652	3311	25.6	39816	1188	33.5
6-10	Th	81732	3784	21.6	38579	1072	36.0
6-11	F	83706	3471	24.1	—	—	—
6-14	M	84735	2835	29.9	—	—	—
6-15	Tu	87191	2578	33.8	40984	1033	39.6
6-16	W	86843	2940	29.5	40609	1212	33.5

NOTE: Although attempts were made to analyze every commuting peak period in each study phase, unexpected detector malfunctions, computer problems, manpower shortages, and other limitations voided complete, accurate data collection for the peak periods omitted from Tables 1 and 2.

TABLE 2

OVERALL EXPRESSWAY PERFORMANCE WITH CONTROL
(3:30 to 6:30 p.m.)

Date	Day	Kostner to Fifth (5.40 mi)			East to Fifth (2.59 mi)		
		Veh-Miles	Veh-Hours	"Speed"	Veh-Miles	Veh-Hours	"Speed"
6-21	M	85632	2276	37.6	39696	1060	37.4
6-22	Tu	84772	2680	31.6	39257	1061	36.9
6-24	Th	85929	2707	31.7	40030	976	41.0
6-25	F	85919	2469	34.8	39985	1324	30.2
6-28	M	83823	2436	34.4	—	—	—
6-29	Tu	84597	2209	38.3	39356	940	41.8
6-30	W	83158	2725	30.5	—	—	—
7-1	Th	85612	2398	35.7	—	—	—
7-6	Tu	79315	3214	24.7	36137	1471	24.6
7-7	W	84951	2145	39.6	39215	830	47.2
7-8	Th	78555	1986	39.5	35872	944	38.0
7-9	F	84939	2295	37.0	39470	1011	39.0
7-12	M	—	—	—	39302	822	47.8
7-13	Tu	—	—	—	38821	831	46.7
7-14	W	86119	2574	33.4	40041	971	41.3
7-16	F	86304	2065	41.8	40030	1013	39.6
7-19	M	—	—	—	39916	841	47.5
7-21	W	—	—	—	39487	1054	37.5
7-22	Th	83709	2242	37.3	38511	870	44.2
7-23	F	—	—	—	38972	1071	36.4
7-26	M	83118	2641	31.5	38491	1132	34.0
7-28	W	85545	2671	32.0	39687	1063	37.3
7-29	Th	85271	2370	35.9	39537	916	43.1
7-30	F	—	—	—	40546	994	40.8
8-3	Tu	66630	3562	18.7	30999	1293	24.0
8-4	W	79845	2807	28.4	36991	1162	31.8
8-5	Th	82696	2928	28.2	38568	1007	38.3
8-6	F	84870	2369	35.8	39356	1107	35.5

See footnote to Table 1.

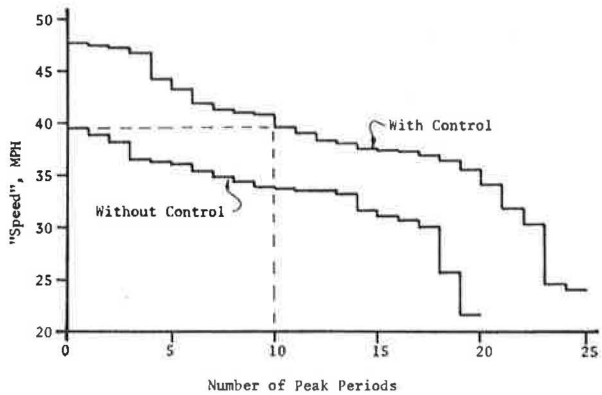


Figure 15. Ranked overall peak-period performance, East to 5th (2.59 miles).

occurring at the "wrong" time and place; the control benefits, if any, in these situations are difficult to measure because of radically dissimilar circumstances.

The long duration of the ramp control evaluation supplied a library of operational data heretofore not obtained for expressway traffic. The daily range of overall performance for "incident-free" peak periods points out the need for long study durations in order to avoid comparing "good" days with control and "fair" days without control, or vice versa (10).

Expressway Accidents

The ranked daily performance data establish that ramp controls allow the expressway to operate at levels higher than possible without controls. Expressway accident data, however, suggest that ramp controls actually cause "good" peak-periods on some occasions by preventing the occurrence of special events which inititate "poor" peak periods. Traffic accident data compiled by the Chicago Area Transportation Study (Fig. 17) indicate a 14.4 percent reduction (444 to 380) in outbound expressway (and

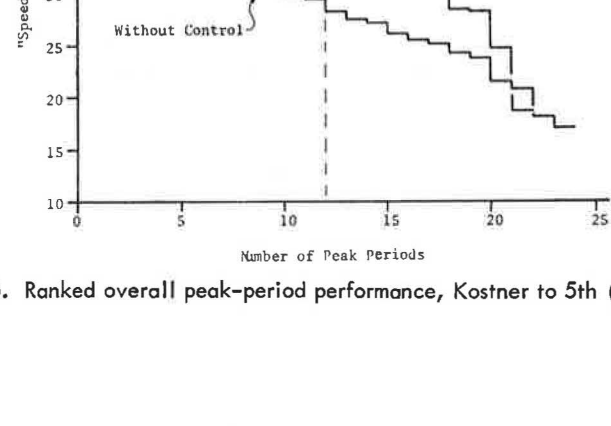


Figure 16. Ranked overall peak-period performance, Kostner to 5th (5.40 miles).

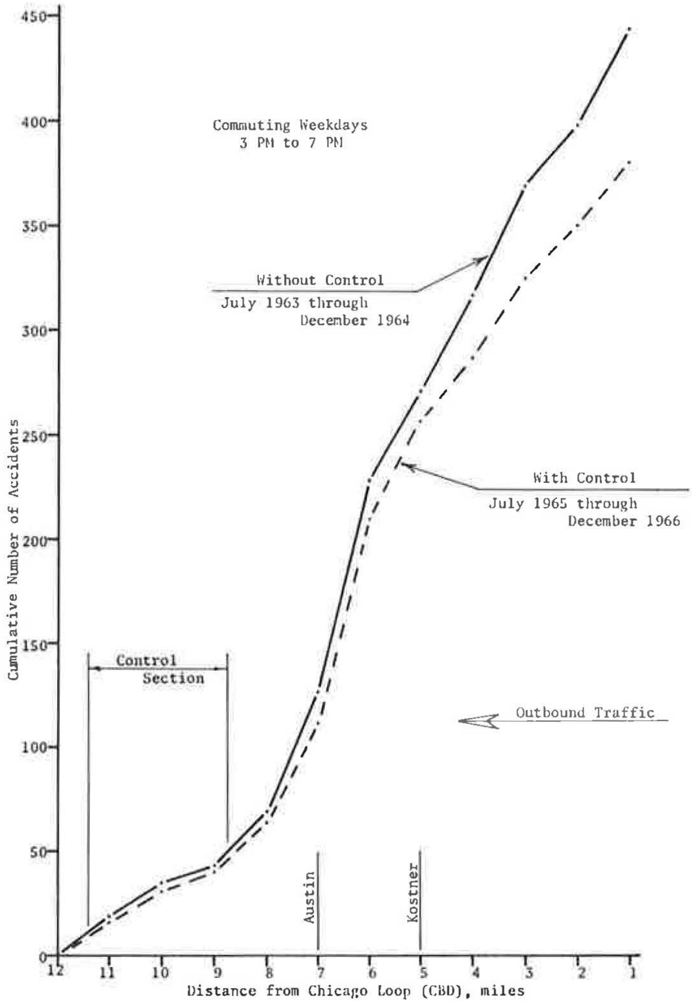


Figure 17. Peak-period accidents, westbound Eisenhower Expressway, commuting weekdays, 3:00 to 7:00 p.m.

ramp) peak-period accidents in the 18 months following commencement of the entrance ramp control system. All other Eisenhower Expressway accidents for the same length of roadway (including inbound peak-period accidents) increased 2.9 percent (2240 to 2306) during the same 18-month period. Assuming outbound peak-period accidents might have increased at this same rate without the ramp control system, the net prorated accident reduction amounts to 16.8 percent (457 to 380).

It is most probable that the reduced outbound peak-period traffic congestion accomplished through the ramp control system is responsible for the decrease in accidents recorded. The upstream propagation of ramp control benefits, such as more "good" peak periods, fewer congested sections, and shorter congestion periods, is reflected by accident reductions mainly in sections upstream of the control section. The favorable accident experience sustained through 18 months of ramp control offers statistically significant evidence of safer expressway traffic operations (11). Moreover, the favorable decrease in congestion and accidents suggests that other expressway operational benefits, such as the reduction of disabled vehicles and reduced air pollution, are also produced by effective ramp controls.

TABLE 3

DAILY ENTRANCE RAMP METERING STATISTICS
(Averages of 50 Control Periods)

Category		Harlem	Des Plaines	First	Seventeenth
Ramp traffic, vehicles		1272	1095	1036	1036
Signal violations, vehicles		49	32	26	30
Signal compliance rate, percent		96.1	97.1	97.5	97.1
Range	Low	93.9	94.8	92.7	93.4
	High	98.5	98.8	99.0	98.8
Control period, minutes		170	169	165	163
Ramp flow rate, veh/min		7.46	6.47	6.26	6.37

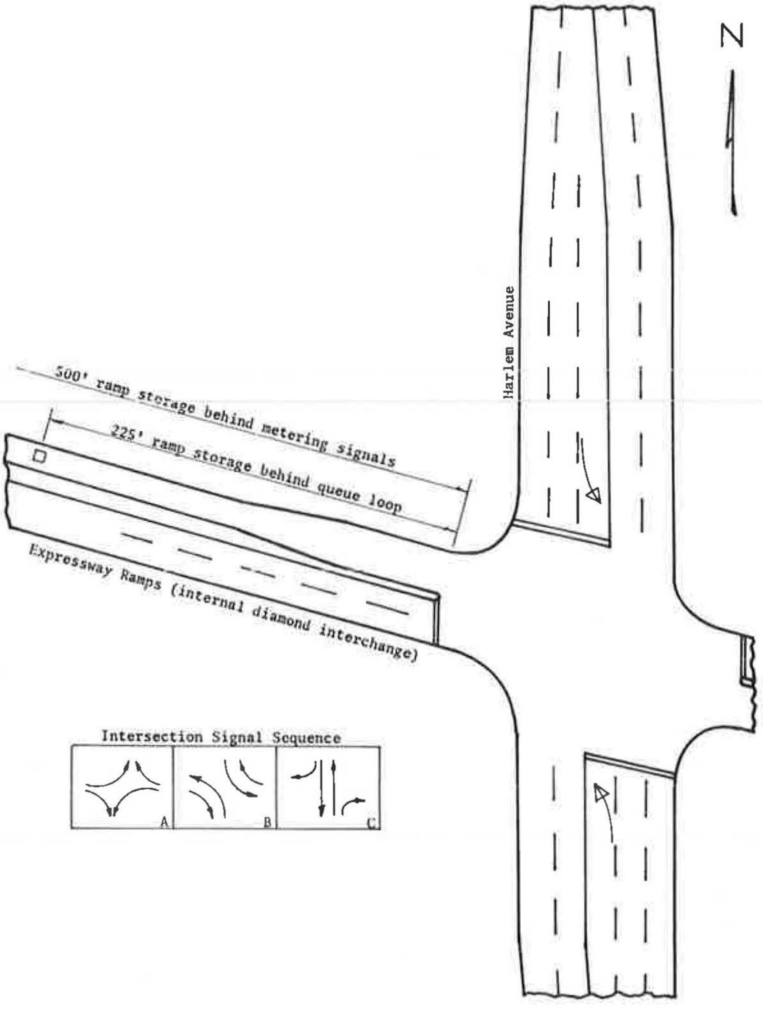


Figure 18. Harlem Avenue intersection with Eisenhower Expressway ramps.

ADVERSE CONTROL EFFECTS

Although ramp metering controls provide operational improvements in expressway traffic flow, ramp delays and surface street friction can result from ramp queues and diversion. Experience with the experimental control system has demonstrated that the major problem of peak ramp demands exceeding the general operational capacity range of each metering device is greatly lessened by gradual traffic adjustment to repetitive metering delays, after an introductory week or two of rather severe queuing problems. Thereafter, day-to-day variations in local metering patterns produce some ramp and surface street delays, especially when expressway traffic flow deteriorates.

Generally, there are no major ramp queuing and surface street problems resulting from ramp controls as long as expressway traffic operations remain non-congested. Inasmuch as the ramp metering rates automatically fluctuate up and down to accommodate the surges common to a high-volume, non-congested expressway traffic stream, a fairly predictable entrance ramp capacity and delay pattern becomes established. The existence of expressway congestion, however, produces sustained restrictive ramp metering, which in turn develops ramp queues extending back into the surface street network. Thus, the least interference with ramp and surface street traffic occurs during the best expressway peak periods, due mainly to the characteristics of the ramp control logic.

Ramp Operations

The effect of each metering device on entrance ramp traffic behavior is similar to findings reported previously for the initial First Avenue installation (3). Motorist acceptance of the "one-at-a-time" metering scheme is reflected by an average red signal compliance rate of over 96 percent at all four sites (Table 3). The queuing and diversion patterns differ for each ramp, however, due to the variation in geometric and traffic characteristics affecting the attractiveness of parallel alternate routes.

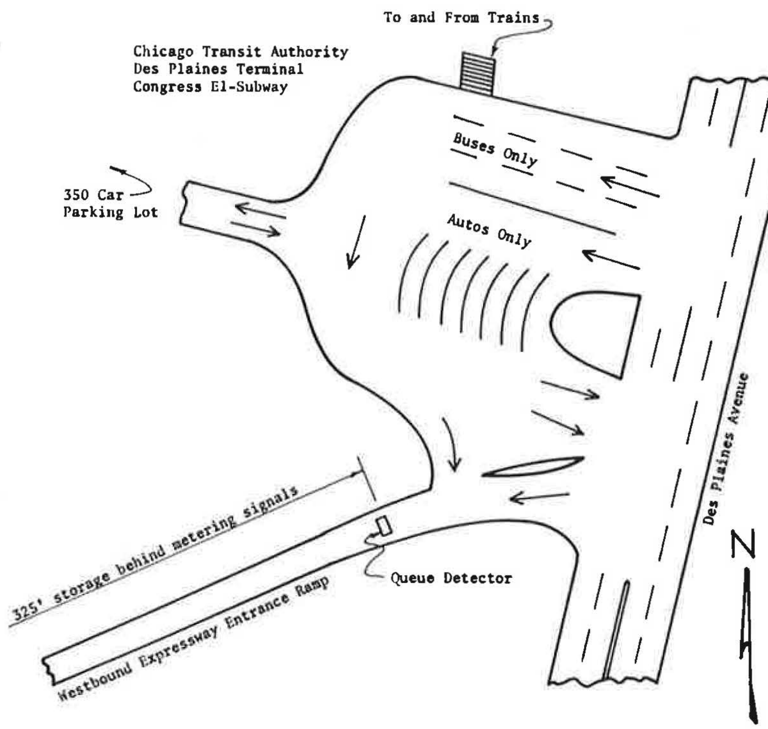


Figure 19. Des Plaines Avenue intersection with westbound Eisenhower Expressway entrance ramp.

The adverse effects of ramp controls at First Avenue and 17th Avenue are insignificant, inasmuch as the expressway flow through this corridor is usually non-congested, a condition producing no severe ramp queuing problems. At the critical Harlem Avenue and Des Plaines Avenue entrance ramps, however, stringent metering periods are usually required each day after congestion develops in the Harlem-Des Plaines expressway area. The lack of frontage roads or other convenient alternate routes at these sites forces ramp queuing, permanent diversion, and daily (optional) diversion.

The Harlem interchange geometrics (Fig. 18) and ramp demand approach patterns confine adverse effects to queue delays and diversion delays, if any. Queuing interference with surface street traffic is minimized by the ample storage areas on the ramp and in the right-turn bay. Left turns onto the metered ramp are favored by the intersection signal sequence, but the low traffic demand from this direction makes it practical to store left-turn vehicles on the ramp each signal cycle, thereby avoiding intersection friction by left-turn queue blockages.

The Des Plaines interchange geometrics (Fig. 19) are complicated by the presence of the rail-rapid-transit terminal parking lot at the entrance ramp head. Severely peaking traffic movements, generally following train arrivals, produce considerable intersection friction attributable to ramp queuing. In most cases, ramp delays are not overly severe, but queue blockages of turning movements into and out of the various driveways impose delays on non-expressway-bound traffic.

Ramp Delays

On good expressway operational days, all metered ramp demands are handled with little trouble; ramp queues are confined to the ramps or immediate approach storage areas. On poor operational days, queue lengths seldom exceed 30 vehicles, due to diversion from long and/or slow queues. Ramp delays, then, are minimized under non-congested expressway flow conditions.

Inasmuch as ramp delays for the 250 to 350 feet of storage area immediately behind each metering device were automatically included in the total expressway travel time computation by using queue detector volumes for the metered ramp inputs, the remaining ramp delays are estimated for periods of queuing beyond the ramp queue detector. On the better expressway operational days, only the Harlem and Des Plaines entrance ramp delays are significant enough to measure:

	Harlem	Des Plaines
Queuing Period	4:00 to 5:35 p.m.	4:35 to 5:40 p.m.
Total Time Queued	65 minutes	45 minutes
Estimated Delay	10 veh-hr	10 veh-hr

Although ramp queues extending beyond the queue detector occur more often at Harlem than at Des Plaines, the more severe Des Plaines queuing produces the same total delay as Harlem, 10 vehicle-hours.

Surface Streets

Besides imposing delays on motorists enduring ramp queues, ramp metering controls produce permanent and optional diversion to alternate routes. Diverted ramp traffic may not only experience increased travel times and/or trip lengths, but also may contribute to existing surface street congestion at critical intersections (2). Comprehensive studies of the surface street network, however, have failed to uncover any deterioration of surface street traffic operations attributable to the ramp controls except at the entrance ramp-arterial street terminals where queues interfered with traffic movements.

Because expressway traffic congestion compounds ramp queuing, the least surface street interference is caused during the best expressway peak periods. The following minimum diversion magnitudes (3:30 to 6:30 p.m.) for the critical Harlem and Des

Plaines ramps consist of permanent diversion and optional diversion, the latter being generated by inspection of ramp queues and/or utilization of the informational display signs, details of which are covered in another report (5).

	Harlem	Des Plaines
Total Diversion	123	384
Optional Diversion	109	68
Permanent Diversion	14	316

Optional diversion varies each peak period with expressway traffic conditions; repetitive delays eventually cause some optional diverters to select permanent alternate routes. The gradual increase of permanent diversion results from restrictive metering control on poor expressway days. Although the metering pattern eventually stabilizes the metered ramp demand, permanent diversion produces vacant ramps (especially at Des Plaines) for some periods on good expressway days. Thus, even though motorists comply with the ramp signals, widely varying controls do not appear to be tolerable. This characteristic, however, is more a reflection of the inaccessibility and inconvenience of alternate routes than an indication of control inflexibility. The permanent diversion component is practically nonexistent where continuous one-way frontage roads parallel the expressway (3).

It does not appear that ramp diversion greatly affects either surface street traffic or diverted traffic except at the complicated Des Plaines entrance ramp terminal. Diversion magnitudes at Harlem (Fig. 20) and Des Plaines (Fig. 21) are distributed over much of the peak period, such that diverted traffic is unlikely to be overloading any specific surface street links. The variations in trip origins and destinations also cause several alternate routes to absorb the diverted traffic (12).

It is suspected that many diverted expressway trips are short in length, particularly at the Des Plaines ramp, where the previous convenience of expressway ramp access

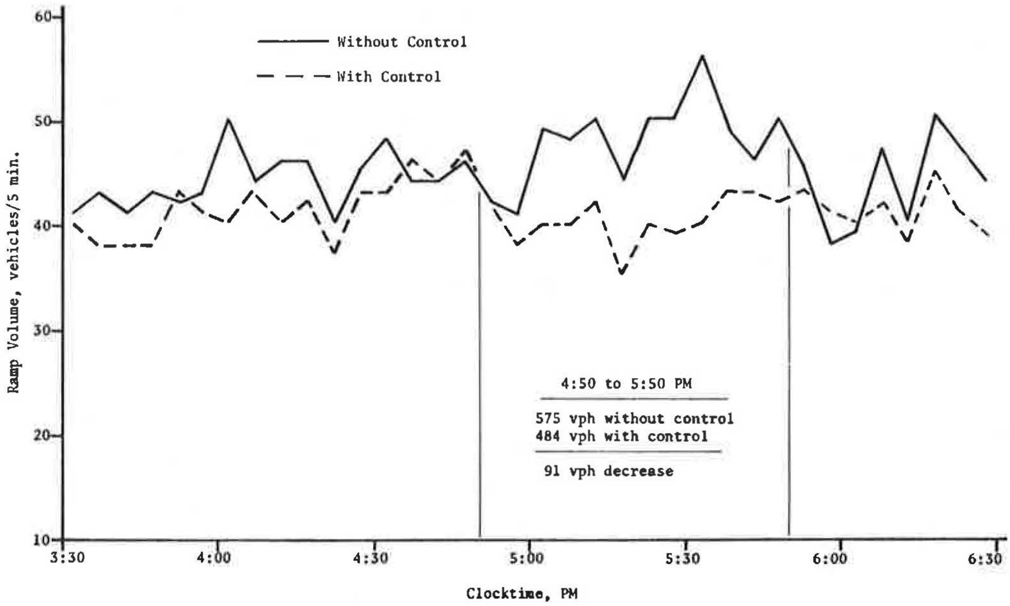


Figure 20. Harlem Avenue entrance ramp volume.

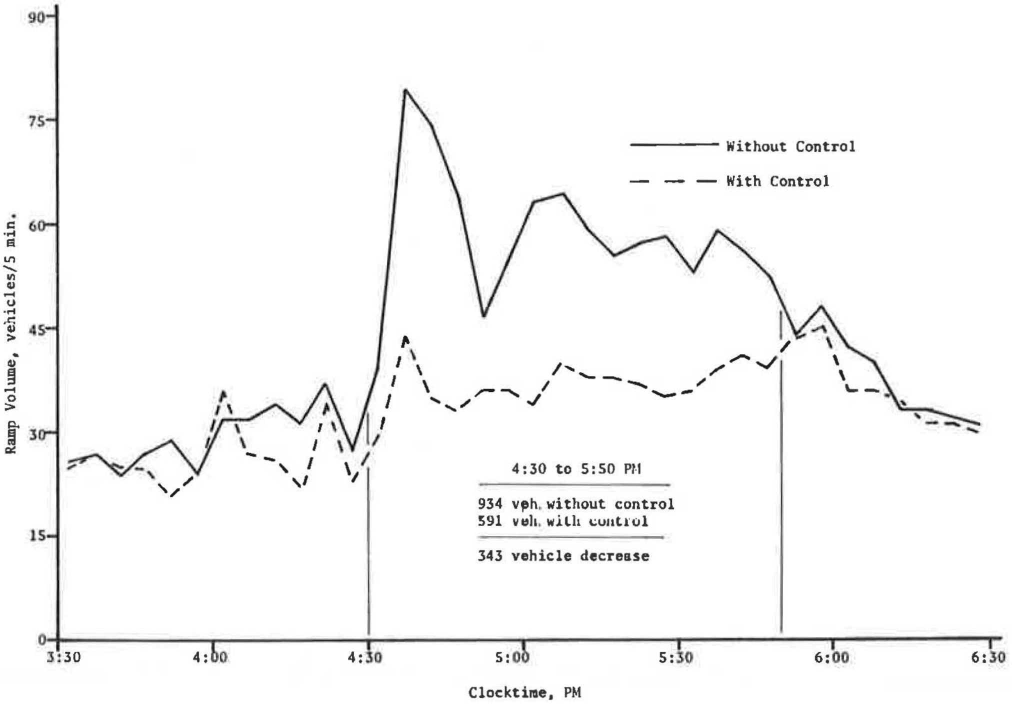


Figure 21. Des Plaines Avenue entrance ramp volume.

attracted "park-and-ride" rail transit patrons not necessarily inconvenienced by alternate surface street routes (12). Nevertheless, although many permanent diverters experience no increased travel time, it seems reasonable to assign a 2-minute increased travel time to all diverted vehicles for the purposes of delay computation, thereby consuming 20 vehicle-hours of total extra surface street travel time.

The major surface street problem is reflected by the response of the affected motorists. Fewer than ten complaints have been received from motorists annoyed at the ramp controls. Most of these complaints concerned delays at Des Plaines, not so much in ramp queues, but in the chaos of turning movements blocked by ramp queues and affecting both expressway and non-expressway users. This particular problem has been alleviated by the installation of conventional traffic signals at the ramp entrance-transit parking lot-Des Plaines Avenue intersection. Future research will coordinate signal timing with ramp queuing; emphasis will be placed on the reasonable allotment of entrance ramp right-of-way while minimizing interference with non-expressway-bound traffic.

CONTROL REFINEMENTS

Research with metering controls at four successive entrance ramps has reduced expressway traffic congestion caused by overloading the Harlem-Des Plaines section of the outbound Eisenhower Expressway. However, the combination of uncontrolled mainline flows and sensitive section geometrics prevents the elimination of peak-period congestion or recovery to non-congested operations once congestion has been introduced under peak flows.

Inasmuch as the advent of traffic congestion during peak expressway flows minimizes the obvious operational improvements produced by ramp metering, the adverse effects transmitted to the contiguous surface street network by stringent ramp controls can be reduced or eliminated by adjustments in the ramp control schemes. In order to prevent

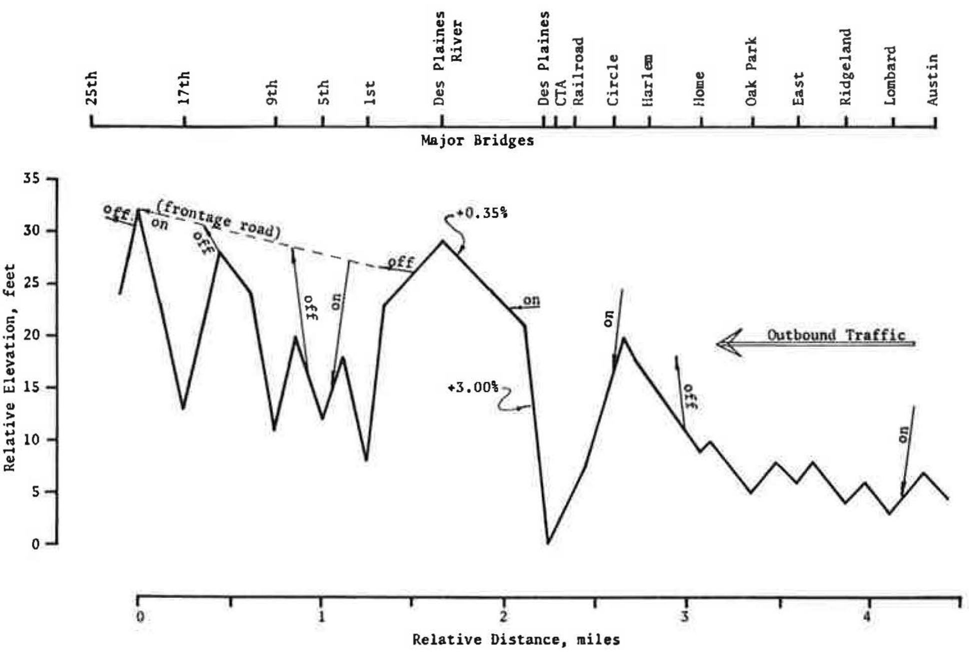


Figure 22. Westbound Eisenhower Expressway profile.

the introduction of expressway traffic congestion, however, more control of the expressway demand reaching critical capacity sections is needed. Thus, control refinements have been developed to provide an operational mode (rather than a research mode) for the present control system and to furnish an expanded control system providing upstream mainline demand reductions.

Operational Adjustments

The major operational problem in the present control section is now the sensitive Des Plaines upgrade (Fig. 22). As suspected, two bottlenecks had existed, the Des Plaines entrance ramp merging area and the critical upgrade. Elimination of severe entrance ramp surges and expressway overloading in the merging area has made the upgrade area the controlling capacity section (especially since the ramp controls increase flows on the upgrade such that the upgrade capacity is eventually exceeded). Once congestion develops in the upgrade area under peak traffic demands, the restoration of non-congested operations has not been possible due to the inability of restrictive metering at the only upstream metered ramp (Harlem) in overcoming the upgrade characteristic of maintaining congestion until the mainline demand decreases significantly at the end of each peak period.

Hence, whenever congestion exists on the Des Plaines upgrade after 4:40 p.m., regardless of the cause, non-congested flow cannot be restored by the present ramp control system (unless an unusual upstream event temporarily reduces the mainline flow reaching the upgrade bottleneck). Restrictive ramp metering, therefore, is now utilized only when congestion does not exist or when recovery from congested traffic operations appears possible (prior to 4:40 p.m. when congested flow rates can be greater than sustained mainline demand rates). In all other peak-period congestion situations, including accidents and other events that increase expressway density beyond the practical influence of the ramp controls, upstream ramps are metered to satisfy actual ramp demands, thereby avoiding most ramp and surface street problems.

Metering Schemes

Several control schemes for operating individual ramp metering devices and systems composed of two or more consecutive metered entrance ramps have been proposed, formulated and tested (1, 2, 3, 4, 13, 14). All schemes decrease allowable entrance ramp inputs upon detected evidence of increased impending expressway traffic congestion. The object of most control schemes is the maximization of expressway-ramp merging volumes and/or critical bottleneck outputs without causing congested traffic operations. Inasmuch as the present degree of control has not sufficiently reduced the expressway mainline demand below the geometric capacity of the critical Des Plaines upgrade, the resulting daily upgrade congestion makes it difficult to measure operational differences between most alternate control schemes, especially since limitless technique refinements are within the range of day-to-day operational variations.

Uncontrollable daily variables, such as prevailing weather conditions and visibility restraints (sun glare, winter darkness, etc.) can impose a lower than ideal operating capacity on the Des Plaines upgrade. Operational traffic factors, such as seasonal and daily demand variations and the prevailing distribution of commercial vehicles, produce additional effects in the control section. (Trucks are restricted to the two right expressway lanes; the left-hand Harlem entrance ramp forces truck lane-changes on or immediately preceding the critical Des Plaines upgrade.) The most significant operator on the critical Des Plaines upgrade, however, is the uncontrolled output of the upstream Austin lane reduction bottleneck, which supplies most of the traffic demand entering the present control section. All these factors produce daily variations in the interaction between traffic flow and physical geometrics which permit the present control section to be operated according to numerous metering schemes without producing measurable evidence of the most effective technique.

Alternate automatic control schemes presently used include local metering control based on expressway lane occupancy measurements upstream and/or downstream of the merge area and system control based on upstream non-congested expressway volumes and occupancy levels throughout the control section (4). Analog computer methods are usually employed for local control schemes; a small, real-time, digital control computer provides the flexibility needed for the more sophisticated system control techniques. Whether the automatic metering schemes are under analog or digitial control, manual override capabilities allow selection of any desired metering rate at any or all metered ramps. In normal (non-research) day-to-day operations, one of the automatic control modes is employed until non-recovery type congestion occurs; manual override control is utilized thereafter at ramps feeding congested expressway sections to prevent ramp queues from seriously disturbing surface street traffic operations.

Although the present ramp control system has been unable to prevent eventual daily peak-period expressway overloading congestion, the introduction of congestion has been effectively delayed. However, any Des Plaines upgrade disturbances in peak flows can easily trigger shock waves to change traffic operations from a "super-sensitive," non-congested state to the inevitable congested state. Thus, both macroscopic and microscopic control parameters must predict or reflect traffic conditions on the brink of breakdown, so that entrance ramp merging turbulence can be minimized by restrictive metering.

All but one of the tested metering schemes have been reasonably effective in responding to critical conditions of impending congestion (even though enough physical control is not available at all times to prevent congestion). Inasmuch as traffic entering the expressway stream at right-hand entrance ramps must initially merge with the shoulder lane, traffic measurements in the shoulder lane preceding the merge area seem intuitively obvious as one-lane control parameters. Metering attempts based on shoulder lane measurements, however, have been relatively ineffective in responding to mainline traffic surges (possibly due to the Chicago area lane-use restrictions which confine trucks to the two right lanes). It has been observed that initial overloading shock waves tend to occur on Chicago area expressways from left to right, even though merging ramp traffic initially enters the shoulder lane. It is suspected that downstream

lane changes by entrance ramp traffic into the high-volume, high-speed left lanes initiates overloading congestion; the unstable volume fluctuations in the low-volume shoulder lane do not reflect these impending problems.

System Extension Upstream

Entrance ramp metering systems apply to expressway situations where mainline demand exceeds capacity. Expressway capacity, however, depends not only on the physical geometrics along the traveled way, but also on the uncontrollable variables (percent trucks, weather, etc.) encountered in day-to-day traffic operations. Hence, in order to prevent expressway overloading congestion, enough control flexibility must be provided to insure that demand flow rates remain lower than prevailing capacity flow rates throughout the control system. Moreover, should congestion develop despite ramp controls, sustained demand flow rates lower than actual congested flow rates must be provided to restore non-congested traffic operations. Inasmuch as non-congested operating volumes can approach prevailing capacity levels, whereas congested expressway sections necessarily exhibit reduced operating volumes, it is not only more desirable to prevent overloading congestion than to recover from congested operations, but also easier to accomplish (less restrictive ramp controls). It follows, therefore, that locations regularly encountering overloading congestion despite ramp controls will not likely experience recovery from congestion via the same ramp controls.

In the case of the Des Plaines upgrade, by delaying or diverting expressway-bound traffic, Harlem and Des Plaines entrance ramp controls increased non-congested upgrade flows to the extent that upgrade capacity was regularly exceeded. Although the sensitivity and capacity characteristics of the upgrade had not been fully realized prior to the ramp control study (due to the previous overwhelming merging problem), it is now obvious that more upstream ramp controls are needed to prevent over-capacity

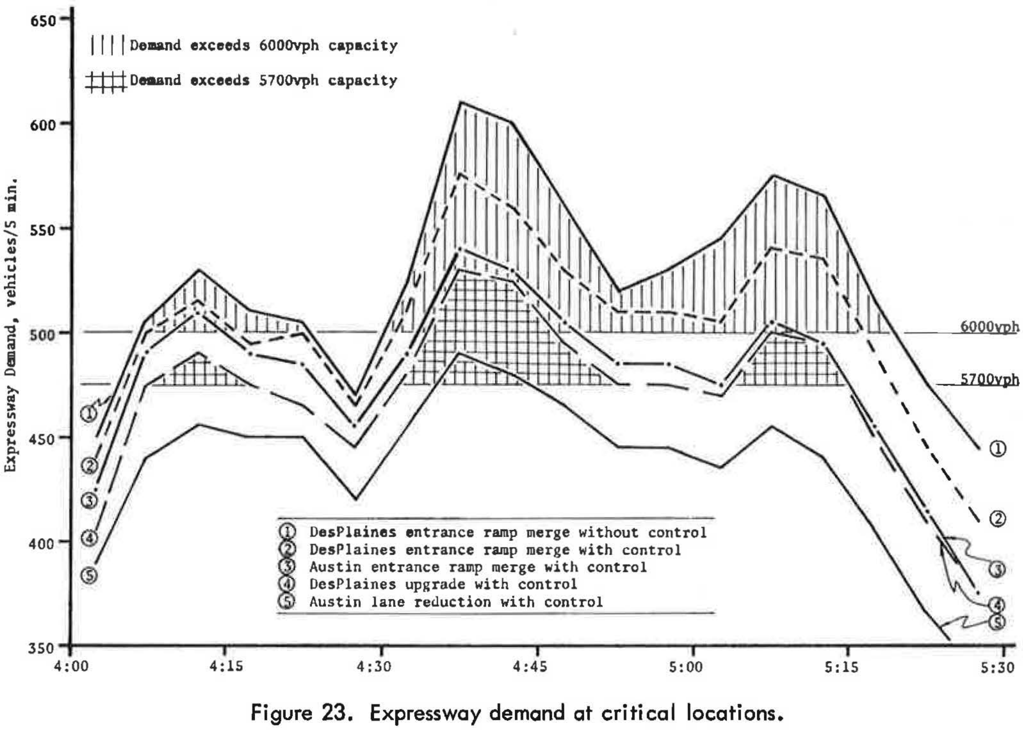


Figure 23. Expressway demand at critical locations.

mainline flows from reaching the critical upgrade. Even subtle mainline surges (independent of ramp merging friction) can develop upgrade shock waves which can propagate upstream to produce sustained peak-period congestion.

The theoretical demand for several expressway sections under the present control system (Fig. 23) was estimated by projecting non-congested upstream flows (at Kostner Avenue) downstream with adjustments for each entrance and exit ramp. The effect of present ramp controls (Harlem and Des Plaines) on the demand at the Des Plaines entrance ramp merge is also presented. The maximum sustained capacity for three-lane expressway sections is 6000 vph, except at the Des Plaines upgrade and Austin lane reduction bottlenecks, where geometric limitations impose a maximum sustained capacity of about 5700 vph. Again, the sustained level of service can be lower on days when uncontrollable variables override the ideal geometric capacity.

Three distinct, critical demand surges occur in each peak period, each surge following each half-hour beginning at 4:00 p.m. Generally, operational problems develop at the first outbound friction point where demand exceeds capacity; the flow downstream and on congested upstream sections is then reduced, as demand for downstream locations becomes delayed temporarily by storing on the expressway. Once congestion has developed, storage queues on the expressway increase until the demand no longer exceeds the congested flow rate.

The 4:00 to 4:30 p.m. demand surge presents potential problems at the Austin entrance ramp merge, the Des Plaines upgrade and the Des Plaines entrance ramp merge. The slight overloading, however, actually occurs initially at the Austin entrance ramp merge, the overloading increment having been introduced by Austin entrance ramp traffic. Although seemingly minor in degree, this merging turbulence is sufficient to trigger premature sustained congestion at the upstream Austin lane reduction bottleneck, illustrating the tendency of most major bottlenecks to bog down when subtle downstream events introduce congestion, thereby masking the real causes. It is expected that ramp metering at Austin Avenue would often prevent this minor overloading. Slight summer demand reductions were ample to delay Austin congestion during the control study.

If the 4:00 to 4:30 p.m. flow rate through the Austin entrance ramp merge is high enough, overloading can occur on the Des Plaines upgrade or at the Des Plaines entrance ramp merge. Usually the present control system can either prevent this problem or aid recovery of non-congested flow, inasmuch as the expressway demand drop prior to 4:30 p.m. is sufficient to dissipate mild congestion.

The major demand surge between 4:30 and 5:00 p.m. places all expressway sections (Fig. 23) potentially over capacity, but the two major bottlenecks at Austin and Des Plaines control the actual flow rates. Sometimes the present control system prevents upgrade congestion through this second surge (under poor Austin bottleneck operations), but the third mainline demand surge between 5:00 and 5:30 p.m. has been heretofore insurmountable.

In terms of theoretical demand exceeding maximum sustained capacity, the expressway section in the Des Plaines entrance ramp merging vicinity presents the greatest overloading potential. Although the present Harlem and Des Plaines entrance ramp controls have cut mainline demands somewhat, additional controls are needed upstream to further reduce mainline demands, such that, with enough control, no prevailing expressway capacity levels will be exceeded. Thus, by preventing overloading congestion in the present control section, an extended control system should also prevent congestion at the Austin lane reduction bottleneck, thereby eliminating the two major expressway bottlenecks during many outbound commuter peak periods.

SUMMARY OF FINDINGS

The experimental expressway traffic control system demonstrates that automatic entrance ramp metering can be a practical remedy for congestion caused by expressway overloading. Excellent motorist compliance with the one-vehicle-at-a-time scheme of the metering traffic signals permits a significant reduction in expressway congestion by delaying and diverting entrance ramp traffic. Although some geometric restraints

and uncontrollable variables are insurmountable with the present control system, significantly higher and safer overall expressway operational levels are provided than without control.

A net overall control savings of 256 vehicle-hours of total travel time (296 vehicle-hours saved by expressway users; 40 vehicle-hours additional delay to metered and diverted ramp traffic) was produced on the expressway, ramps and contiguous surface streets, comparing the best peak periods with and without control. Ramp controls substantially increased upstream expressway flows, without greatly changing downstream bottleneck outputs. Adverse metering effects attributable to ramp queuing and diversion were noticeable only at the metered ramp heads; restrictive metering controls tended to promote permanent diversion to alternate routes.

Inasmuch as the greatest control benefits are probably realized when non-congested expressway flows are maintained, the feasibility of extending the present control system to include enough upstream metered ramps to allow the further reduction of overloading congestion should be thoroughly investigated. Although careful and seemingly superfluous evaluation has demonstrated the value of ramp control systems to urban peak-period expressway operations, further detailed research is needed prior to all practical applications to insure that the degree of ramp control required does not produce adverse traffic effects negating the derived expressway benefits.

ACKNOWLEDGMENT

This paper also constitutes Report 17 of the Chicago Area Expressway Surveillance Project, sponsored by the Illinois Division of Highways in cooperation with the Bureau of Public Roads, Cook County, and the City of Chicago. The author wishes to express his gratitude to the Project staff, especially to Patrick J. Athol, Project Supervisor, for contributions to this report.

REFERENCES

1. Wattleworth, Joseph A., and Berry, Donald S. Peak-Period Control of a Freeway System—Some Theoretical Investigations. Highway Research Record 89, pp. 1-25, 1965; also, Chicago Area Expressway Surveillance Project [CAESP] Rept. No. 9, Aug. 1963.
2. May, Adolf D., Jr. Improving Network Operations With Freeway Ramp Control. CAESP Rept. No. 10, Sept. 1963.
3. McDermott, Joseph M. The Effects of a Metering Device on Entrance Ramp Traffic. CAESP Rept. No. 15, Aug. 1965.
4. Cooley, Philip C. Control Considerations. CAESP Summary Rept., Feb. 1966.
5. Hoff, Gerald C. Development and Preliminary Evaluation of Experimental Informational Signs. CAESP Rept. No. 18, Nov. 1965.
6. May, Adolf D., Athol, Patrick J., Parker, William, and Rudden, James B. Development and Evaluation of Congress Street Expressway Pilot Detection System. Highway Research Record 21, pp. 48-68, 1963; also, CAESP Spec. Rept., Jan. 1963.
7. Wattleworth, Joseph A. System Demand-Capacity Analysis on the In-Bound Gulf Freeway. Texas Transportation Inst. Research Rept. 24-8, Oct. 1964.
8. Kolenko, Stephen. Surface Street Network Volume and Intersection Capacity Study. CAESP Rept. No. 13, Dec. 1964.
9. Disabled and Non-Disabled Vehicle Stoppage on the Congress Street Expressway. CAESP Rept. No. 8, Nov. 1962.
10. McDermott, Joseph M. Daily Variations of Overall Peak-Period Expressway Performance. CAESP Internal Rept., April 1966.
11. Michaels, Richard M. Two Simple Techniques for Determining the Significance of Accident-Reducing Measures. Traffic Engineering, Sept. 1966.
12. Characteristics of Traffic Entering the Westbound Congress Expressway Within the Pilot Detection System Study Section. CAESP Rept. No. 5, March 1962.

13. Athol, Patrick J. Interdependence of Certain Operational Characteristics Within a Moving Traffic Stream. Highway Research Record 72, pp. 58-87, 1965; also, CAESP Rept. No. 11, Dec. 1963.
14. May, Adolf D., Jr. Gap Availability Studies. Highway Research Record 72, pp. 101-136, 1965; also, CAESP Rept. No. 12, May 1964.
15. Athol, Patrick J. Headway Groupings. Highway Research Record 72, pp. 137-155, 1965; also, CAESP Rept. No. 14, Jan. 1965.

Discussion

DONALD F. PETTY, Engineer of Traffic Research, Indiana State Highway Commission—The findings presented in this report are well documented and are another large step toward justifying the use of freeway traffic controls on an operational basis. My limited experience in the field of traffic surveillance and control has been related to the National Proving Ground for Freeway Surveillance Control and Electronic Traffic Aids in Detroit. This discussion covers my reactions to the investigation as viewed from experience on the John C. Lodge Project. The National Proving Ground has used considerably different equipment and techniques for accomplishing similar purposes as the Chicago Area Expressway Surveillance Project. However, one area of the project parallels this study to a great extent—the study on the effect on freeway traffic from closing on-ramps. Although the technique is different, it attempts to reduce congestion on the freeway and also diverts traffic to surface streets. The following conclusions resulted from two studies, and each supports or reinforces the findings made in this study:

1. General increases in volumes of traffic and decreases of travel time were found to result from the closing of on-ramps. More specifically, it was found that volumes were increased as much as 13.7 percent.
2. Lane stoppages were reduced as much as 92.5 percent.
3. It was found that non-congested traffic flow can be restored much faster with ramp control. One incident was reported in which a stalled vehicle affected three lanes of traffic. By closing two ramps, the congested traffic was cleared and all traffic was moving in 22 minutes. Normally, this would have required over one hour when relying on natural processes.
4. In each study a congestion contour map was prepared, and in each study it was shown that the time-space relationship of congestion was greatly reduced.
5. On the John C. Lodge Project, a study was conducted in which driver comfort and service were evaluated as related to the freeway and alternate surface streets. It was found that as volumes approach capacity the freeway provides no more comfort or service than the surface street alternate route. In addition, it was found that the freeway becomes a less comfortable place to drive than the surface route as congestion begins to occur on both routes. From these findings it was concluded that, at least from the comfort and service evaluation, traffic should be diverted from the freeway to permit it to flow as freely as possible, and preferably, to maintain a near-capacity condition.
6. From a public relations standpoint, much less resistance was found in the Chicago study than in the Detroit study. This was because, even though this problem may not be a large one, it is still possible, and even probable, that the wrong person may be inconvenienced and cause a considerable amount of bad public relations. The problem is certainly reduced to a minimum when the ramps are metered and not entirely closed, because the motorist has a choice to make.

As mentioned in the report, the effect of ramp control on accidents was found to be favorable. However, it was pointed out that it would be premature to claim that accidents were significantly decreased as a result of this control. This implies that

further research needs to be done to document this finding on a statistical basis. I strongly recommend that studies of this type be initiated in Chicago as well as at other surveillance and control projects.

Since it has been shown in this study, and reinforced by the John C. Lodge Project, that this type of traffic control can substantially increase the capacity of a freeway, the question arises: "When should I install this or some similar system on a freeway in my state?" When I first became involved with freeway surveillance and control, I asked this question and soon found that the state-of-the-art is not such that this question can be easily answered. It probably will not be easily answered even after many more studies have documented the improvement of traffic flow with these systems. It is possible that a cost-benefit study can be made of any freeway to determine if it is economically feasible to install this or some other system. Although the cost-benefit study may indicate a favorable answer for freeway surveillance and control, many other factors must be considered before making a decision to install such a system. These would include a complete volume-capacity analysis of alternate surface streets and an evaluation of the enforcement, education, and public relation implications of the controls. However, the first consideration must be the availability and/or selection of an alternate surface street route for the motorist. Also, storage space must be available on the ramps being considered for metering. In addition, much more care must be taken in this decision since a considerably greater expense is incurred than with any other traffic control device now used. This implies that as these installations become standard, the location and design of freeways must take this into consideration. The first step toward this goal has been accomplished since the Bureau of Public Roads has shown willingness to participate in the design and installation of conduit for surveillance purposes.

Upon completion of several of these research projects, I am confident that ramp control, as well as other freeway traffic controls, will be found to be justified under certain conditions on an operational basis. When this step is reached and a decision is made to install such a system, it must then be decided who will operate these devices. As was mentioned, this decision will be made considering the three E's of traffic safety—Engineering, Enforcement, and Education—even though the decision may be made by a highway department. Therefore, the operation of such a system should continue to consider these three functions. This implies that management, on a policy-making level, should include representatives from each of these functions since the system will not function at a maximum level without the proper design, appropriate enforcement, and adequate driver education (public relations). This creates a problem since the education or public relation function has not been adequately established by law in most jurisdictions, whereas the engineering and enforcement functions are well established. This means that special care must be taken when choosing a public relations representative for this policy group.

The administration of such a system is appropriately a traffic engineering responsibility. This is particularly true when ramps are metered since changing the controls results from traffic engineering judgment based on an evaluation of various traffic measurements. Also, these systems, as set up for research and operations, have traditionally been administered by traffic engineers.

To summarize, this report has shown that metering traffic on freeway on-ramps will substantially increase volumes on the mainline while decreasing travel times for most motorists. This conclusion has been substantiated in other similar studies, such as the John C. Lodge Project in Detroit.

JAMES L. FOLEY, JR., Commissioner of Transit and Traffic, Baltimore—The 41,000-mile Interstate System includes approximately 6,700 miles of urban freeway, which amounts to about 16 percent of the system. It is expected that this 16 percent will have to carry 52 percent of the traffic.

It is obvious that the urbanized section cannot operate at the same level of convenience as the rural portion. The increasing shortage of adequate gaps in the mainline traffic stream requires the development of additional control techniques. The need for these controls cannot be overemphasized.

In cities and suburban areas, expenditures of considerable amounts of money for traffic control to improve the operational characteristics can readily be justified when we look at the high cost of the land area requirements and the impact of the expressways upon the community. These factors make it obvious that it is impossible to construct all of the expressways which cities could use effectively.

The technique described in this report expands the capacity of the freeway by applying traffic control to one of the significant elements in the system. The report shows that even though significant benefits have been attained, additional investigations are still ahead of us and suggests areas of further research.

It was not too long ago that we felt we knew all of the answers. For example, adequate acceleration lane length was felt to be the solution to on-ramp problems. This report shows us how much we really do not know.

The successes of ramp metering emphasize the importance of preventing congestion build-up. Figure 5 of Mr. McDermott's report presents this relationship graphically. In a somewhat similar situation, invoking evening peak-hour restrictions on Calvert Street in Baltimore one hour earlier than the key demand resulted in lowered delays (shorter queues) to traffic one to one and one-half hours later.

There were several references in the study to use of only the five best operational peaks in each phase. This raises the question of why the analysis was limited and also what the results might have been had all peak hours been studied, excluding perhaps those with known congesting events such as accidents.

Tables 1 and 2 show 20 to 25 days on which data were collected. Would the inclusion of all of these days have altered significantly the outcome or conclusion of the study?

Reference was made to the enforceability of the ramp control signals and the high level of obedience. The enforcement of such controls raises the question of techniques used by the enforcing officer. Stopping vehicles on the expressway for arrests could easily become an event which would reduce the capacity. The report does not indicate whether or not the control equipment presents a green indication to a waiting vehicle in the event that the preceding vehicle is waiting at the end of the ramp.

The expressway condition signs shown in Figure 3 are a technique which we can look to for the improvement of surface, as well as expressway, operations. Expansion of this technique could be a significant contribution to traffic operations and control. The report indicates that diversion characteristics were enhanced by this technique but does not set forth these results in measurable detail. Their use raises the basic question of how well the motoring public understood and used this device.

The improvements at the Austin bottleneck were felt to reflect seasonal variation; however, the Baltimore experience with the Calvert Street restrictions seems to suggest that perhaps the effect of metering downstream is a little more subtle and perhaps exerts greater influence than the measurements would indicate. Thus, metering ramps downstream may have improved the operation at Austin prior to the development of congestion. The effect may be as great or greater than the seasonal variation. This suggests the need for continued research to eliminate the seasonal variation.

In regard to daily performance variations, reference was made to random events. Are these events identifiable, or are they too minor and subtle? Missing dates in Tables 1 and 2 seem to indicate that, at least, major or catastrophic events were deleted.

The discussion of the effect of ramp controls on accidents indicates an area where further study could be fruitful. It is possible that this benefit could, in the long run, be the most significant gain resulting from improved expressway operation. It is assumed that the accidents cited were "reportable" accidents only. The 310 fewer ramp movements could possibly account for some of the reduced accidents. The motorists' acceptance of the "one-at-a-time" metering scheme is indicated by the obedience to the red signal. Does this technique extend to increased courtesy on the part of the mainline driver? This habit adjustment could reflect in reduced accident rates.

A surface street delay may not necessarily be real delay in that in many cases adverse trip length would be avoided. In the early days of Chicago's expressway system, many drivers went miles out of their way to take advantage of the Outer Drive even though the travel time was not reduced. As the short- to medium-length trips are discouraged from using the expressway, the lost time would be further decreased.

Mr. McDermott's report is another vote for continuous frontage road paralleling the expressways. This particular recommendation appears to be cropping up more and more frequently as experience grows.

The assumption of 2-minute increased travel time to diverted vehicles seems to be unduly severe. Inclusion of the calculation of this value would be helpful. It seems that this value is high when the total trip length is considered, inasmuch as expressway operating speeds of 25 to 30 mph do not have the high time-benefit over peak-hour surface operations that expressways have in the non-peak periods.

For the operating traffic engineer, the operational mode is the most exciting. There are presently thousands of miles of urban freeway which are at, or near, operational capacity. The ramp metering technique will make the operation of these facilities more efficient. The inability of even this tool to effectively handle special events such as accidents shows the need for additional types of control or advisory techniques to warn motorists of these problems. Expansion of the expressway sign system described would be one way of advising drivers. Automatic turn prohibitions at the beginning of ramps and advisory signs on the freeway upstream from the problem area could help unload the freeway during such events.

The report does not detail the alternate criteria used for the several control schemes. Of particular interest is the scheme which was apparently ineffective in anticipating the onset of congestion.

One general conclusion of the report is the implication that for maximum effectiveness, complete control of urban ramp systems is likely to be required or, at least, at those ramps feeding the expressway with volumes in excess of a certain value. It would be desirable if this study could determine what this volume level might be.

The inability of the ramp controls to completely solve the problem indicates that perhaps some form of speed control upstream on the mainline would render the ramp controls more effective. For example, automatic speed control, if used east of Austin, might adjust the arrival rate into the control section in a manner to allow greater volumes to enter from the ramps.

As in all good research projects, this study, in addition to answering many questions and providing tools and standards, also suggests additional areas to be examined. It seems that the system has reached a point where the operational mode could be installed in a number of cities for in-service testing.

DONALD G. CAPELLE, Traffic Research Engineer, Automotive Safety Foundation—Mr. McDermott has made a very convincing presentation of the benefits that can be obtained with freeway ramp control. Freeway congestion which results from peak traffic demands has become a widespread problem in all large metropolitan areas and the research presented by the author contributes significantly to the technology of freeway operations.

The use of more stringent controls to achieve satisfactory freeway operational requirements represents a valid approach to the improvement of freeway flow conditions. However, the effect that this control has on the adjacent street system must be reconciled. This is most evident from the author's comments on adverse control effects. His study reflects 10 vehicle-hours of ramp delay, which is very commendable when compared with the 186 vehicle-hours of delay saved on the freeway during the same time period. It would be interesting, although very difficult, to make a precise determination of increased vehicle-hours of delay on the adjacent major street system resulting from the ramp controls.

With the particular street system involved in the study, the delay problem on the streets adjacent to the freeway was apparently not aggravated to the extent that it was objectionable to the average motorist. However, this does not indicate that there was not a substantial overall increase in vehicle-hours of delay on the major street system.

The data resulting from this research show, unquestionably, that freeway operations can be vastly improved with a ramp metering control system. During all control studies, the severity of freeway congestion was significantly reduced with very little decrease in the number of through-put vehicles. This was accompanied by a reduction in the length of peak periods of flow and a 20 percent increase in the average speed of the traffic stream. This is a commendable increase in overall efficiency.

It is interesting to note, however, that the number of diverted ramp vehicles was not replaced by a similar number of freeway vehicles. As the author pointed out, maybe a more efficient ramp control system would permit a one-for-one exchange when ramp vehicles have to be diverted. This would provide an overall increase in the output of the system. It would be of interest to inquire further into why the present control system does not provide for this efficiency.

The accident data in Figure 17 show an overall reduction in accidents with the use of ramp controls. This is not surprising, since congestion and speed differential are two factors closely related to safety and a system which reduces the effect of these factors contributes to overall safer operation.

It appears from the plot of these accidents that the benefits of accident reduction are greatest in the freeway sections upstream from the control area. This can be attributed to the reduction in the upstream congestion. It is hard to comprehend, however, why there was not a similar decrease in accidents in the freeway sections within the control area.

Throughout the report, emphasis is placed on overall savings in vehicle-hours of travel time. This offers a good means for comparing different operations. It would appear, however, that an economic approach would provide a much more meaningful comparison. This type of data is very difficult to obtain, but when we as engineers become involved in projects which restrict the public from using facilities which were built with their tax money, we must be prepared to show the economic benefits that are accrued.

In closing, I would like to again commend Mr. McDermott for his development of a timely and helpful paper on the subject of ramp control. I am convinced that future operation of our freeways will require more and more control techniques similar to those developed in this paper.

JOSEPH M. McDERMOTT, Closure—The helpful suggestions and comments of the three discussions are genuinely appreciated, especially with regard to future research needs. While the research reported was concerned primarily with the operational effects produced through the automatic entrance ramp metering system, research has also been conducted and is continuing in such areas as economic analyses, system control theory, and demand-capacity, corridor relationships.

In this last research area, the Chicago Area Expressway Surveillance Project has been gaining metering experience in freeway corridors with network geometrics and entrance ramp demands quite different from conditions in the initial Eisenhower Expressway study section. In order to evaluate the effects of entrance ramp metering on traffic flows through various corridor configurations, the Project developed portable ramp metering equipment to expedite the conduct of short-term research studies.

The portable metering equipment features a pair of two-section, red-green, portable metering signals powered by standard 12-volt auto batteries. Auto headlight lamps serve as light sources in the signal heads; the interconnect between the two portable signals is accomplished by a radio-control circuit commonly used in model airplane applications. Ramp metering signal changes are triggered manually, by a technician

with a control button, at a pre-determined rate based on previous demand-capacity analyses of the freeway mainline. The technician can usually meter from a car parked on a frontage road adjacent to the ramp signals or from any other nearby inconspicuous vantage point.

Portable ramp metering equipment eliminates the commitment, expense, and installation time lag commonly associated with permanent operational signal systems. In two short-term research studies using four sets of portable metering equipment, the broad research objective of relating various corridor elements to the degree of ramp control was accomplished, while at the same time, the operational advantages of entrance ramp metering in a particular freeway corridor were demonstrated.

In November and December 1966, four entrance ramps on the inbound Dan Ryan Expressway were metered for research purposes with the portable equipment. The freeway corridor configuration was such that a significant reduction in expressway congestion was produced, while minimal adverse effects were transmitted to the surface streets. Inasmuch as these net operational benefits were even more obvious than those reported for the Eisenhower Expressway automatic control system, the Illinois Division of Highways requested continuation of the portable control on a daily basis when the short-term research study was scheduled for termination. Thus, daily peak-period entrance ramp control is now being maintained until a permanent, automatic, operational surveillance and control system is installed.

Although considerable freeway control system research undoubtedly remains to be undertaken, the Dan Ryan Expressway experience demonstrates that there are some freeway corridors suffering from inefficient distribution of traffic loads between the freeway and parallel alternates. There is no reason why ramp metering cannot be applied now as an operational measure in some locations. It can usually be expected that freeway operational improvements will result from entrance ramp controls whenever the entrance ramps cause the freeway overloading congestion problem. Careful preliminary corridor studies are needed, however, to establish that the net system benefits will indeed prove significantly positive.

A Study of the Feasibility of Using Roadside Communications for Traffic Control and Driver Information

Report No. 2

DONALD O. COVAULT, School of Civil Engineering, Georgia Institute of Technology;
TURGUT DERVISH, Traffic Planning Associates, Atlanta; and
ANDREW C. KANEN, Traffic Research Corporation, Toronto, Ontario

A method of driver-roadside communication was tested on the Atlanta Freeway System during daytime and nighttime driving activities in 1964 and 1965. The two related studies attempted to evaluate the effectiveness of roadside radio communication on behavior of the driver as related to his execution of a diverging maneuver from a freeway traffic system. The radio system, called Hy-Com, provides radio communications from the roadside to the driver and consists of a car-mounted receiver and a roadside transmitter.

Volunteer participants were randomly assigned to any one of various test conditions. Each test condition provided guidance information of varying degrees of advance and exit information by using highway signing, radio communication or a combination of both. While information was being given to participants in each test condition, data on traffic characteristics of the driver were collected at various positions along the freeway and the deceleration lane prior to an exit ramp selected for the study. Time-lapse motion photography, the BPR traffic analyzer, and manual recording were used.

Analysis of variance and multiple range test techniques were used to determine differences between driver performances under different levels of information provided during the running of each test condition. The results indicated that audio messages were as effective as visual messages and when given together the performance of test drivers was generally better than that of test drivers with only visual or audio messages. Indications were that a radio-signing system which will provide the necessary information where needed can be effective and at the same time avoid extensive over-signing. Additional research is required to determine the use of radio as a communication device on a system basis.

•A METHOD of driver-roadside communication was the subject of a research project conducted by the Engineering Experiment Station of the Georgia Institute of Technology in cooperation with the Bureau of Public Roads. The purpose was to investigate the feasibility of roadside induction radio communication for traffic control and driver information. Three related studies of communicating with the driver through a radio system were conducted during the years of 1963, 1964 and 1965.

The first year study was designed to measure the effectiveness of roadside radio communication as a traffic control and driver information device, to gage the driver's

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acceptance of this type of roadside radio communication, and to obtain enough information to determine a preliminary value of the price the driver is willing to pay for this communication service.

To accomplish these objectives, a section of a rural freeway (Kentucky Turnpike) was selected for study. At a point along the freeway vehicles from the general motoring public were randomly selected and equipped with radio receivers. The drivers were given audio information on accidents, typical highway maintenance activities and route information while traveling through the test section on the freeway. Time-lapse motion photography was used to collect data on traffic flow, and the test drivers were interviewed at the end of the test section.

As a result of the experiment conducted, it was shown that radio communication could be an effective device for controlling vehicle speed in hazardous areas. This conclusion was indicated by significant differences in speeds between test and control vehicles at locations of potential hazard along the test section.

Interview data showed that drivers generally considered radio communication a useful device for providing various types of information in a variety of situations. The amount of money that they were willing to pay for a receiver capable of receiving roadside broadcasts indicated driver acceptance of this mode of communication.

The studies of 1964 and 1965 were both concerned with the evaluation of the effectiveness of roadside radio communication on driver behavior as related to the execution of a diverging maneuver from a freeway traffic system. The 1964 study was conducted during the day, whereas the 1965 study was conducted at night.

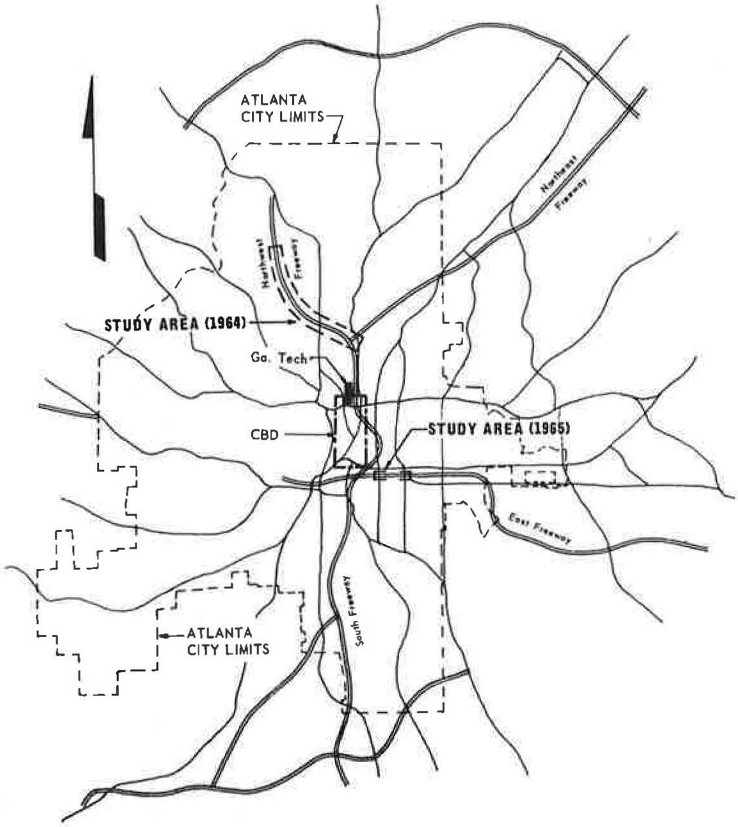


Figure 1. Location of study areas.

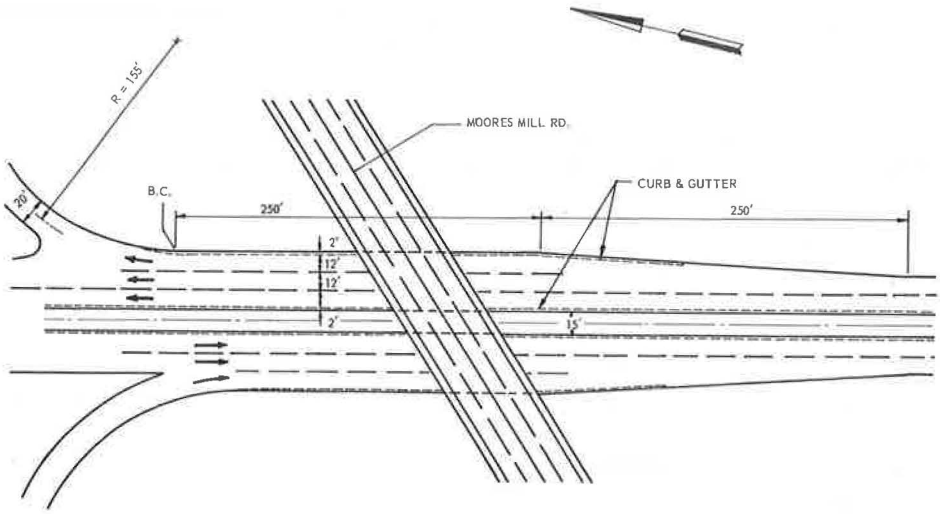


Figure 2. Configuration of deceleration lane and exit ramp, 1964 experiment.

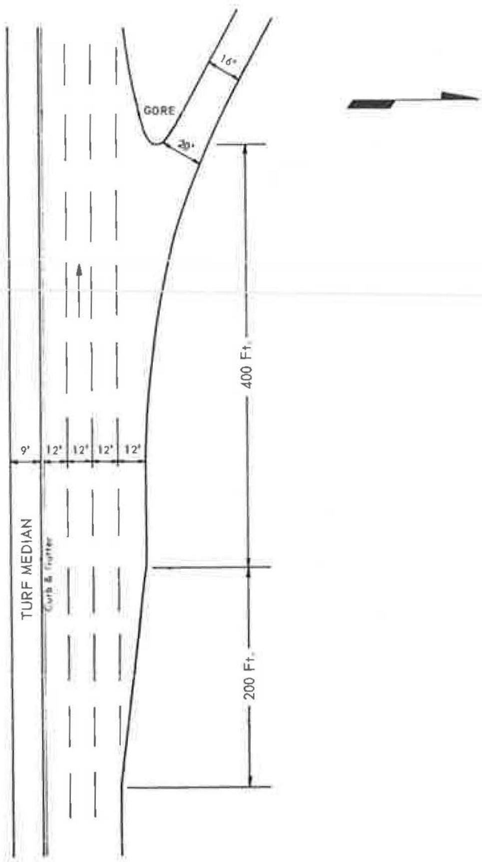


Figure 3. Configuration of the deceleration lane and exit ramp, 1965 experiment.

The experiments involved various test conditions with each test condition providing guidance information about a specific urban freeway exit ramp. The information provided in each test condition varied from very little to a maximum amount and was given through radio communication, highway signing and a combination of both. While information was being given to participants in each test condition, data on vehicle operating characteristics were collected at various positions along a section of the freeway selected as the test site. These data were then used to determine differences between driver performances along the test section prior to the exit ramp and in the execution of the diverging maneuver from the freeway under the different test conditions.

COLLECTION OF DATA

Study Areas

The 1964 experiment was performed on a 4-mi section of the Northwest Freeway in Atlanta. Three interchanges were located within the limits of the study area. Two were modifications of the standard cloverleaf type; the third was of diamond-type design. The exit terminal selected for study was a modification of the standard cloverleaf design.

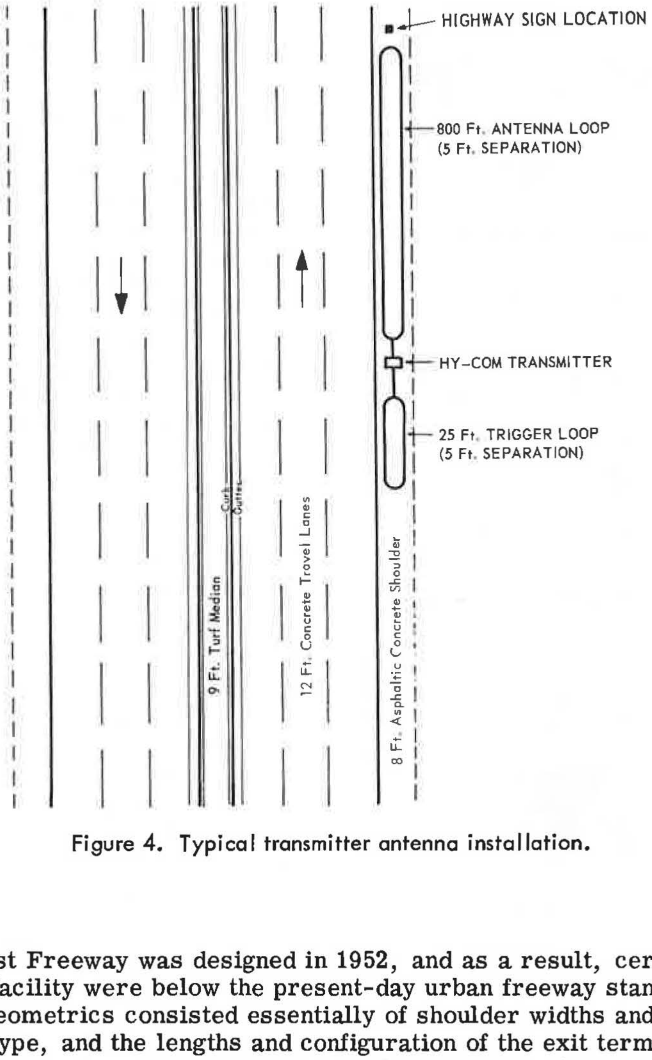


Figure 4. Typical transmitter antenna installation.

The Northwest Freeway was designed in 1952, and as a result, certain geometric features of the facility were below the present-day urban freeway standards. These differences in geometrics consisted essentially of shoulder widths and type, median width and curb type, and the lengths and configuration of the exit terminals.

The 1965 experiment was conducted on a 0.65-mi section of the East Freeway in Atlanta. This freeway was designed in 1957 and about the only geometric feature below present-day urban freeway design standards consisted of median width and curb type. The exit terminal selected for study was of diamond-type design.

Figure 1 shows the existing freeway systems in the Atlanta area and the location of the study areas. Details of the configuration of the deceleration lane and exit ramp are shown in Figures 2 and 3.

Equipment and Instrumentation

The special equipment and instrumentation used in the experiments were for the purpose of communicating with the participating drivers and for collecting information on vehicle operating characteristics at various locations along the test section. To communicate with the driver, Hy-Com and highway signs were used.

The radio communication system was developed by Delco Radio, a division of General Motors. It provides communications from the roadside and consists of a car-mounted receiver and speaker, and a roadside transmitter. As the receiver of an approaching automobile enters the induction field of the trigger-loop antenna (positioned on the highway shoulder just prior to the transmitter), a trigger circuit in the receiver is activated

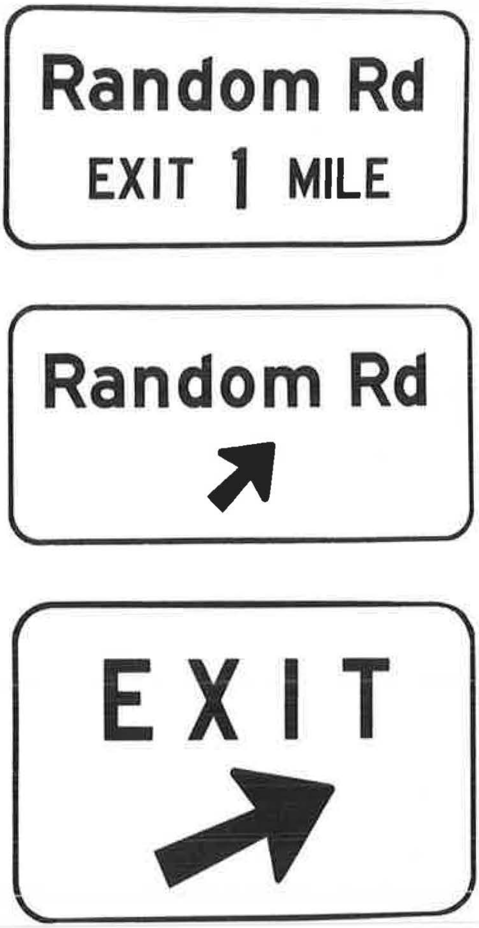


Figure 5. Highway information signs, 1964 experiment.

energizing the audio stages of the receiver. As the receiver enters the field of the information antenna it senses the information signal and provides an audible message to the driver.

The information antenna was an 800-ft antenna loop. It broadcast messages for about 10 seconds and each message was repeated at least 3 or 4 times. Highway signs were positioned at the end of this loop antenna where audio messages ceased to be given. A message given the first time at the beginning of the 800-ft antenna loop was not usually understood by most test drivers who were traveling about 50 mph. By the time the message was repeated, two or three seconds had elapsed corresponding to a distance of about 200 ft. Thus, the driver would start understanding the audio message at a distance of about 600 ft from the highway sign. A study has shown that the mean nighttime legibility distance of 10-in. white reflectorized letters on a dark non-reflectionized background, under simulated roadway conditions, is about 600 ft (14). In other words, audio messages were given as soon as highway signs became visible to drivers so that each method of communication was understood at about the same time. Figure 4 shows a transmitter and two loop antennae positioned along a roadside.

The highway information signs, approaching within practical limits freeway signing standards, were constructed of ¾-in. plywood. To avoid undue interference with and confusion to normal traffic, these signs consisted of a white legend on a black background as opposed to standard white legend on a green background. Upper-case letter height was 15 in. and lower-case letter height was 10 in. Figures 5, 6 and 7 show the highway information signs.

To collect data on traffic flow at various locations along the test section the BPR traffic analyzer, time-lapse motion photography, and manual recording were used. The traffic analyzer is a mobile unit containing an assembly of equipment which provides automatic digital recording of traffic data at several positions on the highway. As the front wheels of a vehicle pass over the first and second tubes of a speed detector and over the placement detector located between the speed detector tubes, speed, time and placement data are recorded on adding machine or punch tapes. The typical layout of vehicle speed and lateral placement detectors is shown in Figure 8.

The camera equipment for time-lapse motion photography consisted of Bolex 16-mm movie cameras driven by 110-volt AC synchronous motors at a rate of 100 frames per minute. At locations where no electric power was available, heavy-duty batteries and vibrator-type convertors were used.

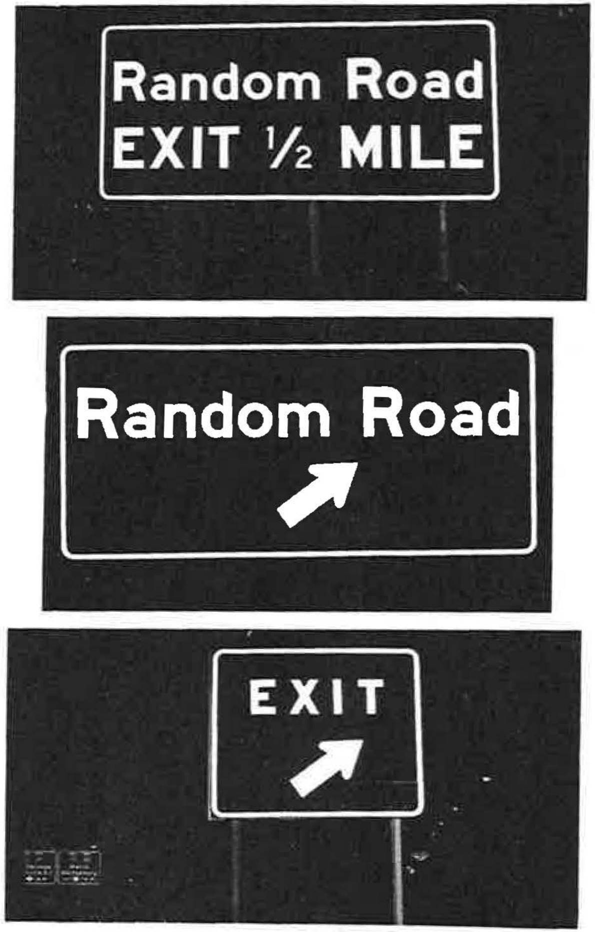


Figure 6. Highway information signs used in the actual experiment of 1965.

Figure 7. Highway information signs not used in the actual experiment of 1965.

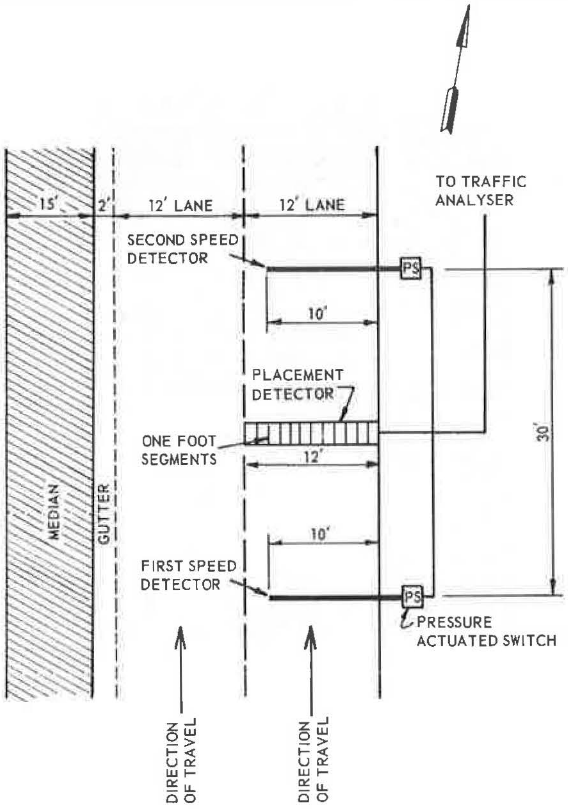


Figure 8. Typical layout of vehicle speed and lateral placement detectors.

Before photographing, a grid system was painted on the edges of the pavement perpendicular to the centerline of the freeway as it would appear superimposed on the projection screen. This grid system was to help in analyzing the film for traffic data. The grid system extended for a distance of 240 ft at 40-ft intervals at each camera location (Fig. 9).

To provide good coordination and efficiency in the performance of the experiments, the staging area, traffic analyzer and camera positions, and any mobile patrol units, were linked together with several citizen band and UHF radio units.

Design of Experiments

Test Subjects—Volunteer participants from local business offices, institutions, service club organizations, Georgia Tech students and staff were used as test subjects. A total of 127 volunteer participants were recruited for the 1964 experiment and 310 for the 1965 experiment.

To evaluate the effect of the radio communication on the learning process of motorists, test subjects were requested in the 1964 experiment to participate for three repeat trips over the test section, each trip taking place on a different day, under as nearly as possible identical conditions. The test condition on each of the three trips was the same, but the test subjects were not so informed before participation.

The 1965 participants were requested to travel over the test section only once. They were asked to travel in the right through lane because speed detector tubes of traffic analyzers covered only a one-lane width. If a slower vehicle was traveling ahead of them, they were instructed to pass the vehicle and return to the right through lane.

To familiarize the test subjects with the operation and characteristics of the radio equipment, a briefing session was held before the actual running of the experiments. In addition, specific directions were given regarding the test route, and each participant was randomly assigned to a specific test condition and provided with a schedule of days on which he was asked to travel over the test section. The chronological order of the test conditions was randomized. In the event of wet pavement or other difficulties, a test condition was rescheduled to take place after all other test conditions were conducted.

To negate, as much as possible, the test subjects' specific advance knowledge regarding the exact location of the selected freeway exit ramps, these ramps were renamed "Random Road." The participants were requested to exit at Random Road, the specific location not being revealed.

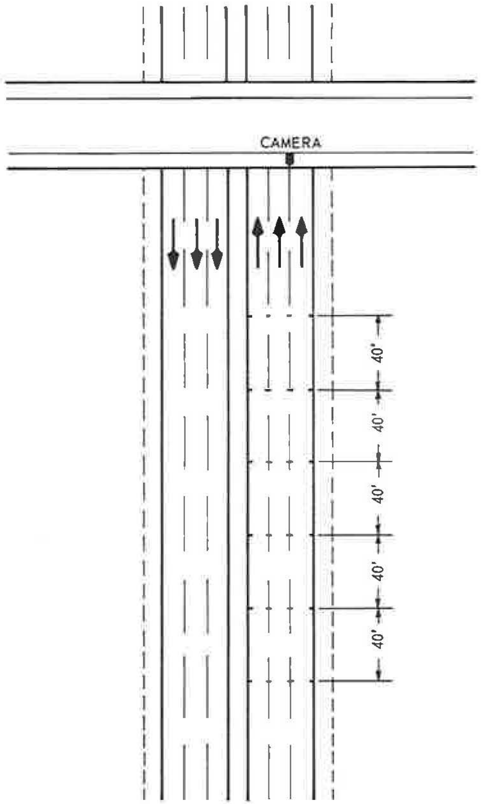


Figure 9. Typical camera and grid layout.

TABLE 1

TEST CONDITIONS, 1964

Test Conditions	Type of Information*	Audio Message	Visual Message
A_0V_1	Advance	No message	Random Road exit 1 mile
	Exit	No message	No message
A_1V_1	Advance	"Random Road one mile ahead"	Random Road exit 1 mile
	Exit	No message	No message
A_0V_2	Advance	No message	Random Road exit 1 mile
	Exit	No message	Random Road ↗; Exit ↗
A_2V_1	Advance	"Random Road one mile ahead"	Random Road exit 1 mile
	Exit	"Random Road exit"	No message
A_2V_2	Advance	"Random Road one mile ahead"	Random Road exit 1 mile
	Exit	"Random Road exit"	Random Road ↗; Exit ↗
. . .			
A_0V_0 (control)	No message	Data from a systematic sample of vehicles were collected for control. These drivers were unaware of the experiment but were considered to have maximum information since they exited at the same ramp as the test vehicles.	

*Advance information was given through transmitter No. 4; exit information, through transmitter No. 5.

TABLE 2

TEST CONDITIONS, 1965

Test Condition	Type of Information*	Audio Message	Visual Message
A_0V_0 (control group)	Advance	No message	No message
	Exit	No message	Exit ↗
A_0V_1	Advance	No message	Random Road exit ½ mile
	Exit	No message	Exit ↗
A_1V_0	Advance	"Random Road ½ mile"	No message
	Exit	No message	Exit ↗
A_1V_1	Advance	"Random Road ½ mile"	Random Road exit ½ mile
	Exit	No message	Exit ↗
A_0V_2	Advance	No message	Random Road exit ½ mile
	Exit	No message	Random Road ↗ / Exit ↗
A_2V_0	Advance	"Random Road ½ mile"	No message
	Exit	"Random Road exit"	Exit ↗
A_1V_2	Advance	"Random Road ½ mile"	Random Road exit ½ mile
	Exit	No message	Random Road ↗ / Exit ↗
A_2V_1	Advance	"Random Road ½ mile"	Random Road exit ½ mile
	Exit	"Random Road exit"	Exit ↗
A_2V_2	Advance	"Random Road ½ mile"	Random Road exit ½ mile
	Exit	"Random Road exit"	Random Road ↗ / Exit ↗

*Advance information was given through transmitter No. 4; exit information, through transmitter No. 5.

Test Conditions—The experiments were to evaluate radio communication as an effective aid to the motorist in performing a diverging maneuver from a freeway traffic stream.

There were various separate test conditions. The differences consisted of varying degrees of advance and exit information provided, and the use of different modes, or combinations of modes, of conveying this information. The test conditions are given in Tables 1 and 2. In these tables, visual information refers to highway signing and audio information refers to roadside radio communication. The advance information was provided approximately one mile in advance of the Random Road Exit for the 1964 experiment and approximately one-half mile in advance for the 1965 experiment. The exit information was provided near the beginning of the tapered section of the deceleration lane and at the gore of the exit ramp.

A secondary experiment, conducted concurrently with the 1964 experiment, consisted of providing audio messages regarding simulated roadside activities on the test

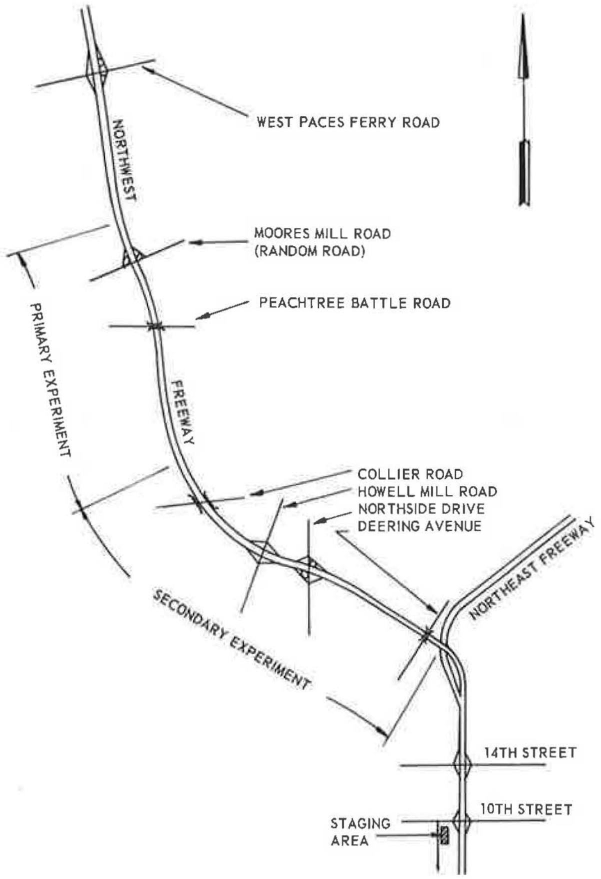


Figure 10. Test route and relative locations of staging area and test section, 1964 experiment.

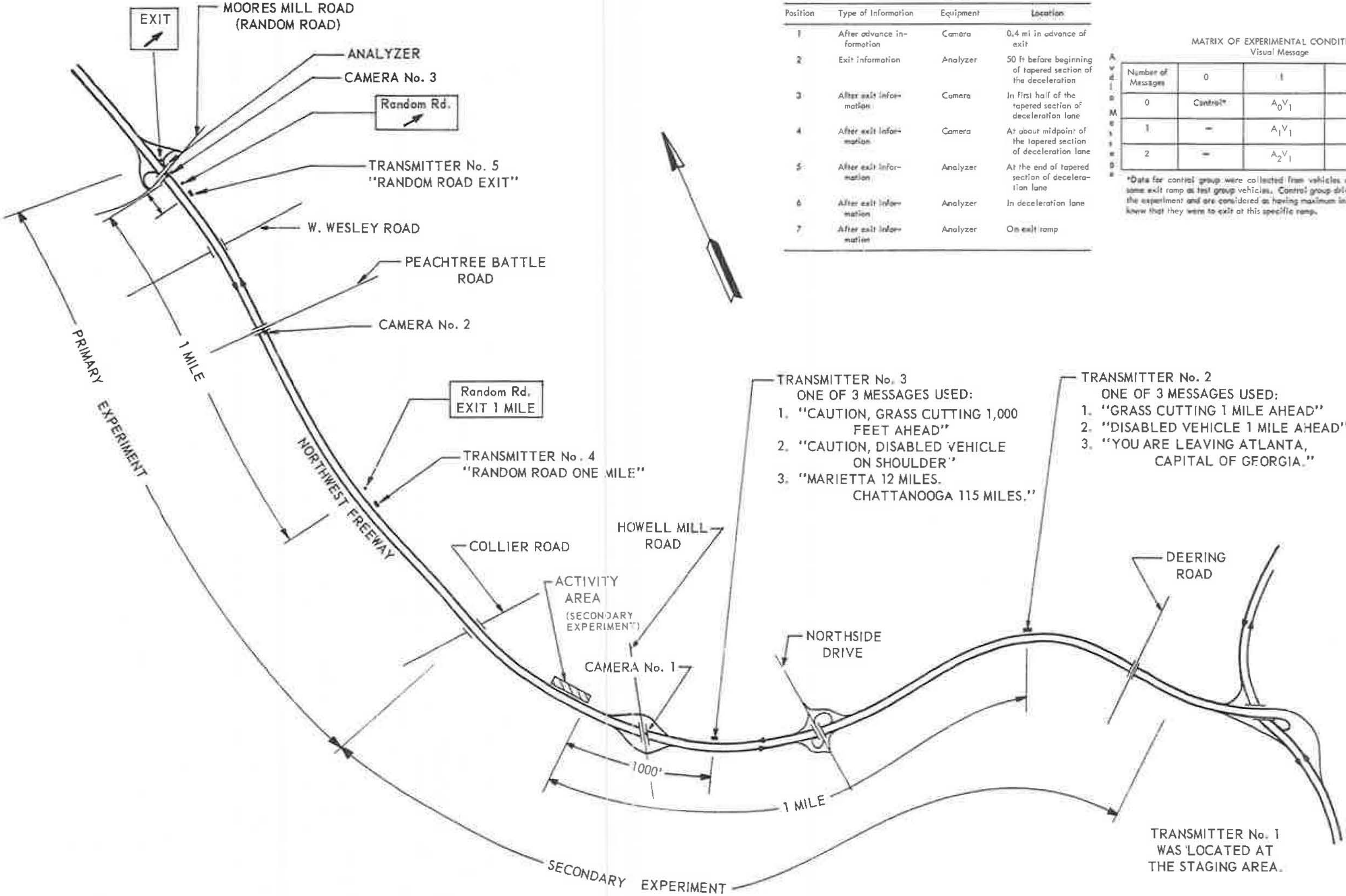
section. Three different test conditions were used: grass cutting on the highway shoulder, positioning of a disabled vehicle on the shoulder adjacent to the outside travel lane, and provision of general route information without any simulated physical roadside activity.

The purpose of the secondary experiment was to provide an evaluation of the effectiveness of roadside radio communication as a general informational device and to provide a "screen" by concealing from the participant the importance of the primary experiment.

For the grass cutting condition, the only warning motorists received other than radio messages was provided by a tractor operating with the headlights turned on. The radio messages were as follows: transmitter No. 2—"Grass cutting one mile ahead"; transmitter No. 3—"Caution, grass cutting 1,000 feet ahead."

For the disabled vehicle condition a ½-ton pickup truck with the hood raised was parked on the shoulder of the highway near the pavement edge. No other warning device other than the radio messages was employed. The messages were the following: transmitter No. 2—"Disabled vehicle one mile ahead"; transmitter No. 3—"Caution, disabled vehicle on shoulder."

The general route information condition was conducted without any simulated physical roadside activity and involved the provision of the following radio messages: transmitter No. 2—"You Are Leaving Atlanta, Capital of Georgia"; transmitter No. 3—"Marietta 12 miles; Chattanooga 115 miles."



Position	Type of Information	Equipment	Location
1	After advance information	Camera	0.4 mi in advance of exit
2	Exit information	Analyzer	50 ft before beginning of tapered section of the deceleration
3	After exit information	Camera	In first half of the tapered section of deceleration lane
4	After exit information	Camera	At about midpoint of the tapered section of deceleration lane
5	After exit information	Analyzer	At the end of tapered section of deceleration lane
6	After exit information	Analyzer	In deceleration lane
7	After exit information	Analyzer	On exit ramp

MATRIX OF EXPERIMENTAL CONDITIONS

Visual Message (columns); Audio Message (rows)

Number of Messages	0	1	2
0	Control*	A_0V_1	A_0V_2
1	–	A_1V_1	–
2	–	A_2V_1	A_2V_2

*Data for control group were collected from vehicles exiting freeway at same exit ramp as test group vehicles. Control group drivers were unaware of the experiment and are considered as having maximum information since they knew that they were to exit at this specific ramp.

Figure 11. Transmitter, analyzer, camera, and highway sign locations along the test section, 1964 experiment.

Performance of Experiments

Experiment of 1964—The experiment was conducted from July 21 to July 30, 1964. To avoid the influence of peak-hour traffic volumes on the experiment and the inconvenience to the test subjects resulting from peak-hour traffic conditions, the experiment was conducted between 9:30 a.m. and 3:30 p.m. The 6-hr period was divided into morning and afternoon periods, 9:30 a.m. to 12:30 p.m. and 12:30 p.m. to 3:30 p.m. Each 3-hr period constituted a test period during which one of the group of test subjects executed one trip over the test route.

On the designated day and within the limits of the stipulated 3-hr period, the participants in the appropriate test condition arrived at the staging area, where the Hy-Com receivers and speakers were installed on their vehicles by project personnel. Colored adhesive stickers were applied to the bumpers to permit easy identification during the film analysis and to provide ready identification for the traffic analyzer operators where manual coding was required.

Transmitter No. 1, located at the staging area, provided a final test for adequacy of performance of the receiver and speaker and helped the driver become familiar with the sound of the radio instructions. Driving instructions and travel directions which the test subjects had received during the briefing session were briefly reiterated for each driver before his departure from the staging area. The test driver then proceeded along the designated route to the test section on the Northwest Freeway where audio and

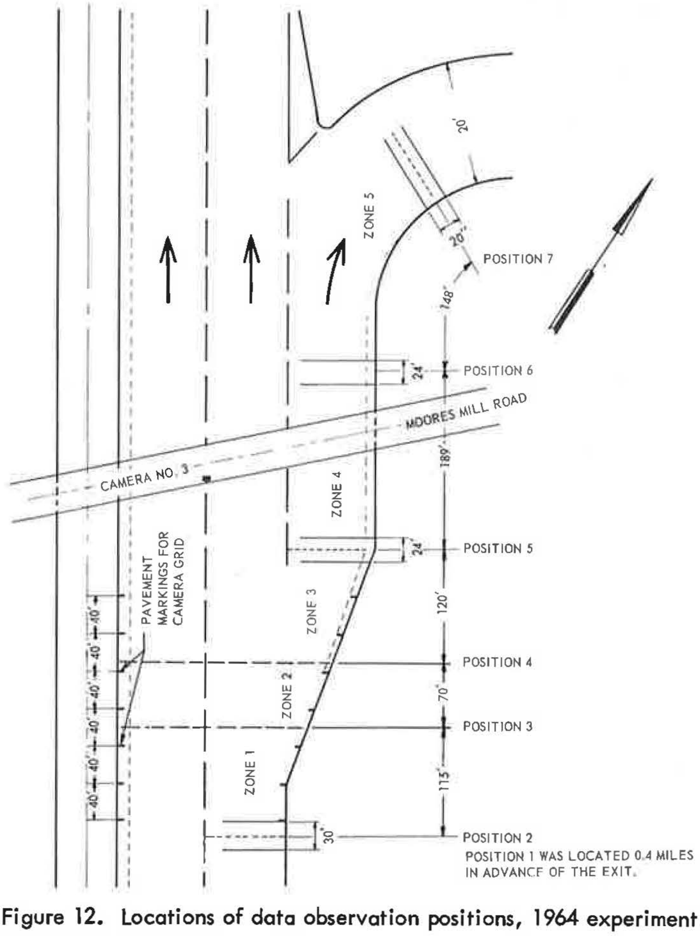


Figure 12. Locations of data observation positions, 1964 experiment.

visual messages were given in accordance with the test schedule employed for each period. Figure 10 shows the test route and relative locations of the staging area and test section; Figure 11 shows the test section with camera, transmitter and highway sign locations along with actual messages broadcast and highway information signs provided.

Approximately 1,000 ft beyond transmitter No. 3, camera No. 1 was positioned on the overhead structure of Howell Mill Road to record the passage of test vehicles. This camera position was the only data observation station for the secondary experiment. Approximately 0.4 miles before the Random Road exit, camera No. 2 was located on the overhead structure of Peachtree Battle Avenue. At this point, test subjects had already received the advance exit information, and data collection was based on the same rate and time schedule of film exposure as camera No. 1. The film was exposed continuously from 9:30 a.m. to 10:10 a.m. and from 10:30 a.m. to 11:10 a.m., and from 12:30 p.m. to 1:10 p.m. and from 1:30 p.m. to 2:10 p.m.

Camera No. 3 was located on the overhead structure of Moores Mill Road (Random Road) and recorded vehicle travel characteristics along the tapered section of the deceleration lane. In addition, four traffic analyzer data collection positions were located in the vicinity of the exit (Fig. 12). Data collection at these positions near the exit was on a continuous basis. The traffic analyzer traps together with camera No. 3 recorded data on all vehicles exiting the freeway. Time-lapse movie photography was interrupted only at the end of each film to allow reloading of the camera, which took approximately ten minutes.

Because the traffic analyzer speed and placement traps covered only one lane width at any one position, it was possible for a vehicle to bypass the first three traps and still exit via the Moores Mill Road off-ramp. Test vehicles missing some of the traps and the passage of vehicles through the camera observation areas during reloading periods produced unequal numbers of data observations at the various points along the test section. In addition, the percentage of test subjects actually participating differed between groups and between trips of any one group.

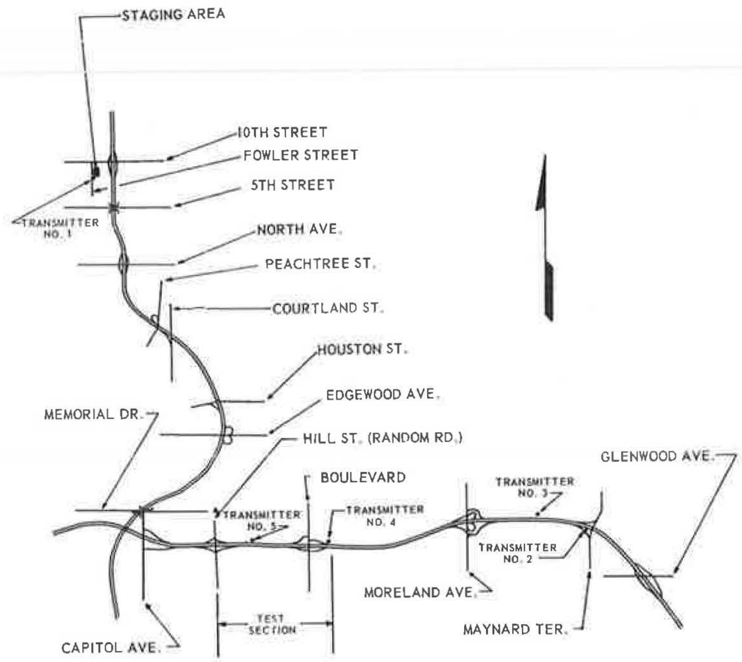


Figure 13. Location staging area, test route and test section, 1965 experiment (test drivers traveled easterly from the staging area to Glenwood Avenue interchange where they turned around and traveled westerly toward the test section).

TABLE 3

MESSAGES NOT INCLUDED IN THE ACTUAL EXPERIMENT OF 1965

Location	Audio Message	Visual Message
Staging area	Transmitter No. 1: "Information on actual roadway conditions will be given to you on East Freeway."	Radio experiment Atlanta Freeway
Near Maynard Terrace before Glenwood off-ramp	Transmitter No. 2: "Turn around at Glenwood."	Random Road –
East of Moreland Avenue	Transmitter No. 3: "Use right through lane."	No message

Experiment of 1965—The experiment was performed during a nighttime traffic situation from 8:30 p.m. to 11:00 p.m. from July 19 to August 3, 1965. One test condition was studied each night.

At the staging area, project personnel installed Hy-Com receivers and speakers on the vehicles of the participants. For identification during data recording and film analysis, a light bulb, connected with the car's battery, was attached to the grill at the lower right front of each car. Reflectionized bumper stickers were also placed on the rear bumper.

Transmitter No. 1 provided a final test for adequacy of performance of the receiver and speaker and helped the driver become familiar with the sound of radio instructions. Driving instructions and directions for travel over the test section were briefly repeated for each driver, and then he proceeded along a designated route to the test section and exited at Random Road.

Between the staging area and Random Road test drivers were given audio, visual, a combination of audio and visual, or no information according to the particular test schedule. Two of the four transmitters (Nos. 2 and 3) located at two positions between the staging area and test section were not included in the actual experiment, but broadcast information on test route and lane usage. Figure 13 shows the location of the staging area, test section and transmitters.

Transmitter Nos. 1, 2 and 3 broadcast their respective messages to all test subjects no matter what test condition was employed. No data were collected at these locations. Table 3 gives the location of transmitters and highway signs not included in the actual experiment and the type of message given at each location.

Data collection locations along the test section were at three critical positions: (a) in the vicinity of the advance information point for Random Road (before and after any advance information), (b) in the vicinity of the exit information point for Random Road (before and after any exit information), and (c) at the point of entry (beyond exit information at entrance to the exit ramp). Two traffic analyzers and three cameras were used to collect traffic data in the vicinity of the advance and the exit information points. At the point of entry the positions where the test cars entered the deceleration lane were recorded by determining the point of entry of both right and front wheels with respect to the beginning of the tapered section of the deceleration lane.

A traffic analyzer located about 1,000 ft west of the Boulevard Interchange structure was used to record data at four positions for vehicles prior to and during any advance information. Camera No. 1, which was positioned facing the traffic on the overhead structure of Boulevard, recorded the passage of vehicles while actual advance messages were being given according to the particular test schedule. Transmitter No. 4 and highway signing were used to give advance messages in accordance with the test schedule employed for the night at this portion of the test section.

Camera No. 2 was positioned on the same overhead structure as camera No. 1 and recorded the passage of vehicles going away after advance or no information had been given. Camera No. 3 was positioned on the overhead structure of Cherokee Avenue facing the traffic and recorded the passage of vehicles along a portion of the test section after any advance information and prior to any exit information.

Speed Position	Type of Information*	Equipment	Distance Between Positions (feet)
1	No information	Traffic analyzer	440
2	Advance information	Traffic analyzer	160
3	Advance information	Traffic analyzer	300
4	Advance information	Traffic analyzer	80
5	Advance information	Camera No. 1	420
6	No information	Camera No. 2	1170
7	No information	Camera No. 3	324
8	Exit Information	Traffic analyzer	100
9	Exit information	Traffic analyzer	328
10	Exit information	Traffic analyzer	100
11	No information	Traffic analyzer	

*According to test schedule employed, no exit information might have been given and in the case of the control groups, no information was given except an exit sign at the gore or the Random Road exit.

MATRIX OF EXPERIMENTAL CONDITIONS

Visual Message

Audio Message

Number of Messages	0	1	2
0	Control Groups A_0V_0	A_0V_1	A_0V_2
1	A_1V_0	A_1V_1	A_1V_2
2	A_2V_0	A_2V_1	A_2V_2

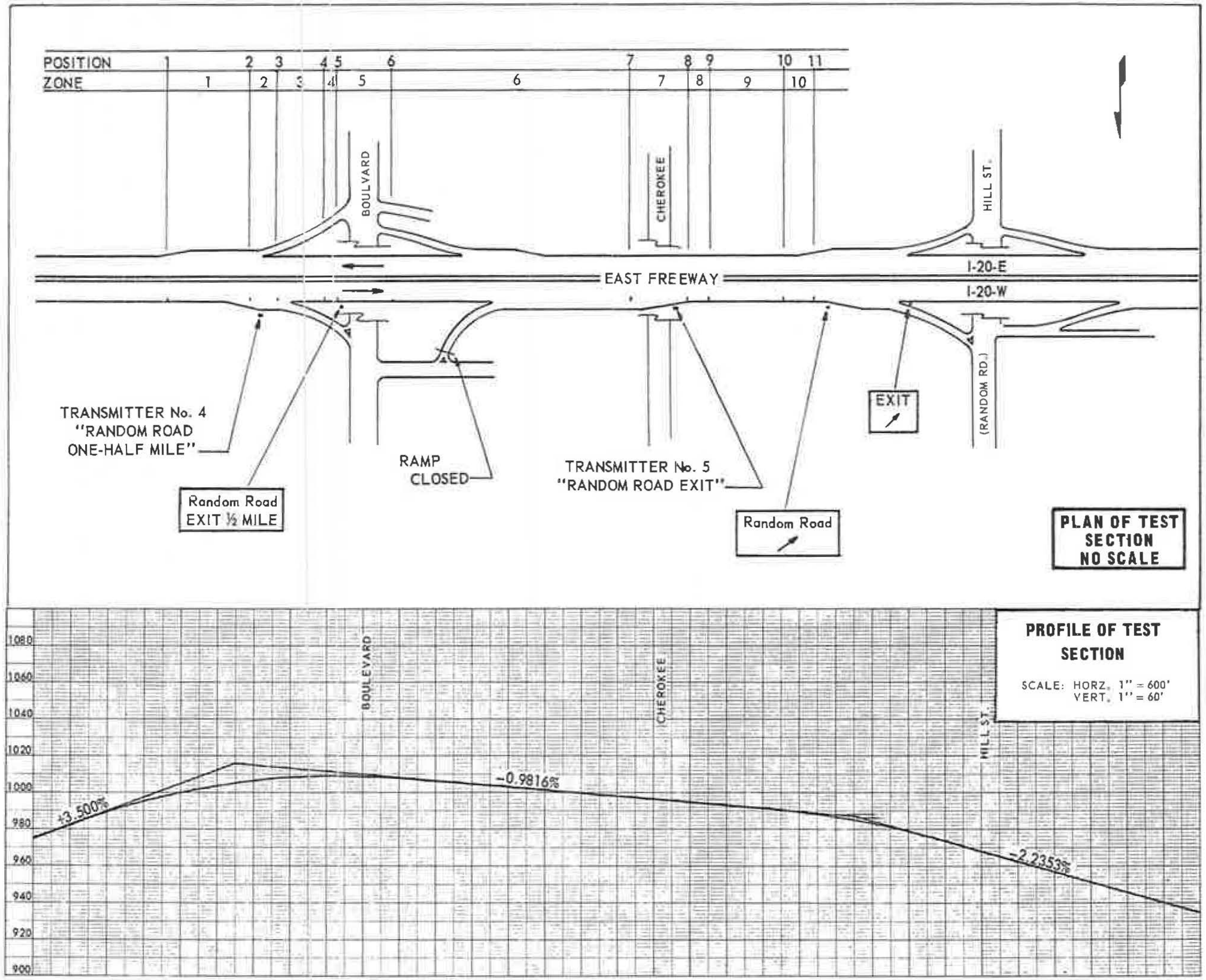


Figure 14. Analyzer, camera, transmitter, and highway sign locations along the test section, 1965 experiment.

A second traffic analyzer was used to collect traffic data during and after any exit information (or none) was given. Exit information was given through either transmitter No. 5 or highway signs, or both. Figure 14 shows the test section with transmitter, camera, analyzer, highway sign locations, and speed-measurement positions.

A total of 310 test cars participated in the experiment. However, due to camera or traffic analyzer failures, reloading of cameras, and the effect of cars leading or trailing the test car on the speed of the test car, some of the data collected were eliminated from the analysis. The data were analyzed with respect to the entire test section as a unit and also in two increments of the test section: (a) the vicinity of advance information point (positions 1 through 6), and (b) the vicinity of exit information point (positions 6 through 11). Of the 310 test cars which participated in the experiment, only 121 could be used for the entire test section, 141 for the vicinity of the advance information point, and 147 for the vicinity of the exit information point.

DATA ANALYSIS

The film exposed at the camera locations was analyzed by projecting it through a time and motion study projector which allowed a frame-by-frame analysis of each roll of film. The film was projected onto a screen on which a grid was superimposed to the same scale as the grid painted on the pavement. Using the grid technique it was possible to analyze the film for vehicle speed, travel time, and lateral placement.

The traffic analyzer data required the decoding of the printed or punch tape data and the correlation or tracing of each vehicle through each of four positions. Decoding the data yielded the time and speed of each vehicle at each position and the lateral distance between the center of each vehicle and the pavement edge. Lateral placement was measured only in the 1964 experiment.

After individual vehicle speeds were obtained at various positions along the test section, vehicle accelerations or decelerations were computed for the various zones which consisted of increments of the test section between speed measurement positions. These were computed on the basis of initial velocity, final velocity and distance traversed. The relationship used was

$$\text{Acceleration} = \frac{(v_2)^2 - (v_1)^2}{2S} \tag{1}$$

where

v_1 = initial velocity;

v_2 = final velocity; and

S = distance traversed.

The traffic parameter, acceleration noise, was computed from data obtained during the 1965 experiment. Continuous data on speed-time relationships, which permit the computation of acceleration noise, were collected at 11 speed-measurement positions along the test section. Acceleration noise is defined as a measure of the smoothness (or uniformity) of the driving given by the dispersion σ of the acceleration noise. Mathematically stated this quantity is

$$\sigma_a^2 = \frac{1}{T} \int_0^T (a_t - \bar{a}_t)^2 \, dt \tag{2}$$

where

σ_a = acceleration noise;

T = time duration of travel over the test section;

a_t = acceleration of a car at time t; and

$\bar{a}_t$ = average acceleration of the car over the test section.

If a car's final speed is the same as its initial speed, $\bar{a}_t = 0$. Also, for any prolonged journey $\bar{a}_t$ approaches zero and in most cases it is comparatively small and can be neglected. Thus, acceleration noise is normally calculated by the equation.

$$\sigma_a^2 = \frac{1}{T} \int_0^T a_t^2 \, dt \tag{3}$$

In this study acceleration is sampled at successive intervals, Δt, and therefore an estimate of acceleration noise was obtained by approximating Eq. 3 by

$$\sigma_a^2 \approx \frac{1}{T} \sum_{i=1}^{n} \left[a(t)_i\right]^2 \Delta t_i \tag{4}$$

where

$a(t)_i$ = acceleration at an increment of the test section;

Δt_i = travel time between consecutive speed measurement positions; and

n = number of increments of the test section.

In the study of 1965, the entry to the deceleration lane was divided into five equal increments of 100 ft in length starting from the beginning of the tapered section of the deceleration lane. An observer recorded the position of entry of both right and left front wheels of each test car as it entered the deceleration lane. Thus it was possible to determine the percentage of test cars entering the deceleration lane at various increments of the deceleration lane under different test conditions. These data were used to compare the point of entry of test subjects under different levels of information as they entered the deceleration lane.

Performance Criteria

To evaluate driver performance and behavior under various levels of information, several criteria were selected as "ideal standards." For the 1964 experiment continuous data were collected from a point located about 50 ft before the beginning of the tapered section of the deceleration lane to a point at the beginning of the off-ramp. The only other data observation point was located approximately 0.5 mi in advance of the exit. The criteria were

1. Test drivers should have constant speeds along the freeway and should exit the freeway without slowing down on it;
2. The drivers should decelerate along the deceleration lane in making the diverging maneuver; and
3. The vehicle placement on entry into the deceleration lane should be as close as possible to the outside edge of the pavement.

For the 1965 experiment continuous data were collected in the vicinity of both the advance and the exit information points. The criteria selected as "ideal standards" were as follows:

1. In the vicinity of the advance information point, deceleration should approach zero:

$$d_a = \frac{dv}{dt} \to 0 \tag{5}$$

2. In the vicinity of the exit information point, deceleration should approach zero:

$$d_b = \frac{dv}{dt} \rightarrow 0 \qquad (6)$$

3. At the point of entry, entry into the deceleration lane should be at earliest possible point:

$$\left(P_E - P_T\right) \rightarrow 0 \qquad (7)$$

where

P_E = point of entry, and

P_T = point of taper.

4. Acceleration noise along the test section should approach zero:

$$\sigma_a^2 = \frac{1}{T} \int_0^T (a_t - \bar{a}_t)^2 \, dt \rightarrow 0 \qquad (8)$$

In certain instances, the control group was used as a basis of comparison but not as an "ideal standard."

Statistical Analysis

After individual vehicle speed, acceleration and lateral placement data were obtained, they were analyzed using analysis of variance and multiple range techniques. Because of the unequal number of test subjects in each test condition, the standard analysis of variance technique was not applicable. The analysis of variance for the nonorthogonal data is described in detail elsewhere (9). Subsequent to the analysis of variance, Scheffe's method was used in order to judge all contrasts in the analysis where significant differences were indicated. It was decided to determine contrasts at the 10 percent significance level instead of the usual 5 percent and 1 percent because of the large variabilities involved in traffic data.

Statistical analysis made for data computed for acceleration noise was through the usual techniques of analysis of variance for a one-way classification experimental design.

Primary Experiment (1964)

Analysis of Variance, Vehicle Speeds—After the analysis of variance for the dependent variable of speed, comparison tests were made to determine which of the test conditions and which of the trips were significantly different. The results are given in Table 4, which shows the rank order and significant differences of vehicle speeds at the seven positions.

The results at position 1 indicate that drivers in the various test conditions operated their vehicles in a manner which did not produce significantly different speeds among groups. All other positions showed significant differences in vehicle speeds between the various test conditions. Test condition A_2V_2, which was given the greatest information, was found to have significantly higher speeds throughout the length of the deceleration lane, except at the beginning of the tapered section, than test condition A_0V_1, with least information. The control group, similar to test condition A_2V_2, exhibited significantly higher speeds than test condition A_0V_1 throughout the deceleration lane, except at the end of the lane and on the off-ramp (positions 6 and 7 respectively). The control group data were collected for vehicles which exited the same ramp as test vehicles but were unaware of the experiment. The control group drivers were considered

TABLE 4

RANK ORDER AND SIGNIFICANT DIFFERENCES BETWEEN TEST CONDITIONS FOR THE DEPENDENT VARIABLE SPEED
(Significance Level 10 Percent)

Position	Test Condition and Mean Speed (mph)					
	Lowest					Highest
1	A_0V_1 48.15	A_0V_2 49.15	A_2V_1 50.45	A_1V_1 50.49	A_2V_2 50.54	Control 51.99
2	A_0V_1 41.41	A_1V_1 43.06	A_2V_1 43.37	A_0V_2 43.52	A_2V_2 44.44	Control 45.9
3*	A_0V_1 39.46	A_1V_1 40.18	A_0V_2 40.88	A_2V_2 41.39	A_2V_1 41.89	Control 42.03
4	A_0V_1 37.88	A_1V_1 39.73	A_2V_1 40.13	A_2V_2 40.34	A_0V_2 40.38	Control 41.43
5	A_0V_1 35.33	A_0V_2 38.18	A_1V_1 38.61	A_2V_2 39.19	A_2V_1 39.49	Control 39.62
6	A_0V_1 29.02	A_1V_1 31.01	Control 31.55	A_2V_1 31.91	A_0V_2 32.02	A_2V_2 32.14
7*	Control 21.50	A_0V_2 21.55	A_0V_1 21.71	A_1V_1 21.96	A_2V_1 22.48	A_2V_2 22.69

There are no significant differences between those conditions underlined.
*The significant ranges for positions 3 and 7 were not detectable with Scheffe's procedure, and were estimated using Duncan's procedure.

as having maximum information since they knew they would be exiting the freeway at this specific ramp. The nonsignificant differences in vehicle speeds between the control group and any of the other test conditions at the end of the deceleration lane and on the off-ramp, and the significantly lower speeds recorded for the control group (maximum information) on the off-ramp than test condition A_2V_2 (supplied with the most

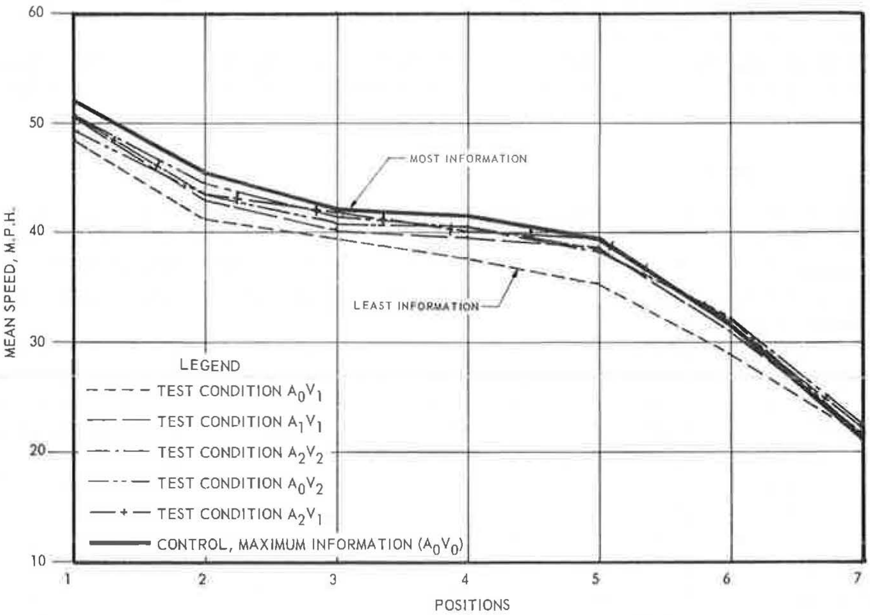


Figure 15. Mean vehicle speeds of the test conditions at seven positions near the exit.

TABLE 5

RANK ORDER AND SIGNIFICANT DIFFERENCES BETWEEN TRIPS FOR THE DEPENDENT VARIABLE SPEED
(Significance Level 10 Percent)

Position	Lowest		Highest
1	Trip 1	Trip 2	Trip 3
	49.13	49.47	51.75
2	Trip 1	Trip 3	Trip 2
	43.00	43.65	43.90
3	Trip 3	Trip 1	Trip 2
	40.61	40.91	41.40
4	Trip 1	Trip 2	Trip 3
	39.64	40.04	40.31
5	Trip 1	Trip 3	Trip 2
	37.58	38.85	38.95
6	Trip 1	Trip 3	Trip 2
	30.45	31.45	31.92
7	Trip 1	Trip 3	Trip 2
	21.32	22.12	22.49

There are no significant differences between those trips underlined.

information), indicate that the control group test subjects were aware of the geometry of the exit ramp and their speed was controlled by prior knowledge of its configuration.

Test condition A_0V_2 with visual advance and exit information was found to have significantly higher speeds than test condition A_0V_1 at the midpoint along the taper and at the end of the deceleration lane. Similarly, test condition A_2V_1 with two audio and one visual message showed significantly higher speeds than test condition A_0V_1 at two locations along the deceleration lane.

These results indicate that as the amount of information given to test subjects was increased, they traveled generally at higher speeds during the execution of the diverging maneuver. Although there were no significant differences in the behavior of all the test subjects in the vicinity of the advance information, there were significant differences in the vicinity of the exit information. The significantly lower speeds of test condition A_0V_1 from some of the other test conditions provided with more information at positions before the beginning and along the tapered section of the deceleration lane, indicate that test subjects with least information showed a tendency to slow down on the freeway and in the early stages of the diverging maneuver.

Mean vehicle speeds for each test condition at the seven positions near the exit ramp are shown in Figure 15. The results of the comparison tests between the three trips for the seven positions are summarized in Table 5.

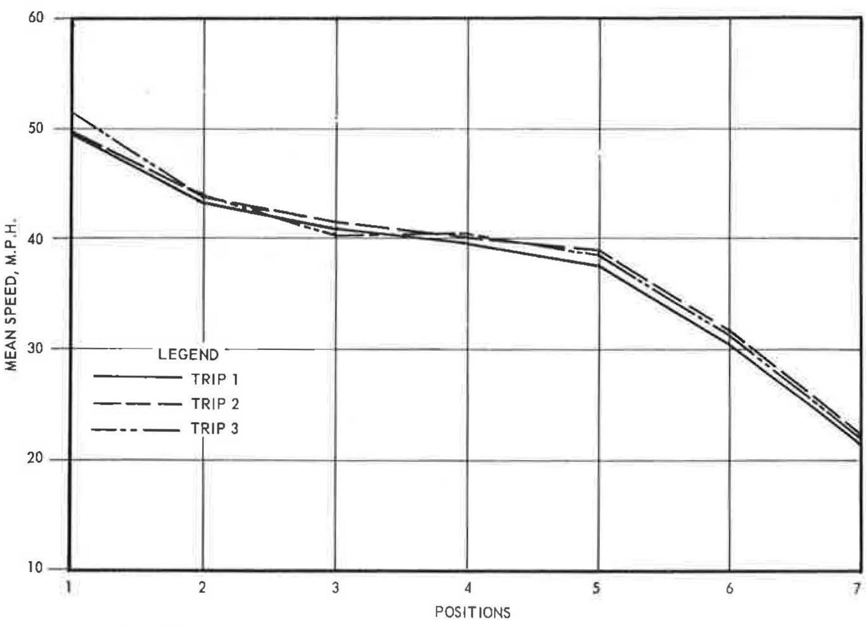


Figure 16. Mean vehicle speeds for each trip at seven positions near the exit.

TABLE 6

RANK ORDER AND SIGNIFICANT DIFFERENCES BETWEEN TEST CONDITIONS FOR THE DEPENDENT VARIABLE DECELERATION
(Significance Level 10 Percent)

Zone	Test Condition and Mean Deceleration (ft/sec²)					
	Lowest					Highest
1	A_0V_1	A_2V_1	A_1V_1	A_0V_2	A_2V_2	Control
	2.02	2.09	2.17	2.65	2.79	2.91
2	Control	A_0V_2	A_2V_2	A_1V_1	A_0V_1	A_2V_1
	0.88	1.13	1.51	4.52	1.57	1.80
3*	A_1V_1	A_2V_2	A_2V_1	A_0V_2	Control	A_0V_1
	0.32	0.81	1.18	1.36	1.34	1.49
4	A_0V_1	A_2V_1	A_0V_2	A_2V_2	A_1V_1	Control
	2.29	2.74	2.79	2.85	3.02	3.21
5	A_0V_1	A_1V_1	A_2V_2	A_2V_1	Control	A_0V_2
	2.88	3.63	3.79	3.93	3.97	4.33

There are no significant differences between those conditions underlined.
*The significant ranges for position 3 were not detectable with Scheffe's procedure, and were estimated using Duncan's procedure.

The vehicle speeds during the three trips were found to be not significantly different at approach to and along the tapered section of the deceleration lane (positions one through four). However, from the beginning of the full-width deceleration lane through to the beginning of the exit ramp, the vehicle speeds during trip 1 were significantly lower than those during trip 2. During trip 2, test subjects were familiar with the configuration of the deceleration lane and the the exit ramp and the higher speeds recorded during trip 2 show the effects of learning. Mean vehicle speeds for each trip at the seven positions are shown in Figure 16.

Analysis of Variance, Deceleration—After analysis of variance for the dependent variable of deceleration, comparison tests were made to determine which of the test

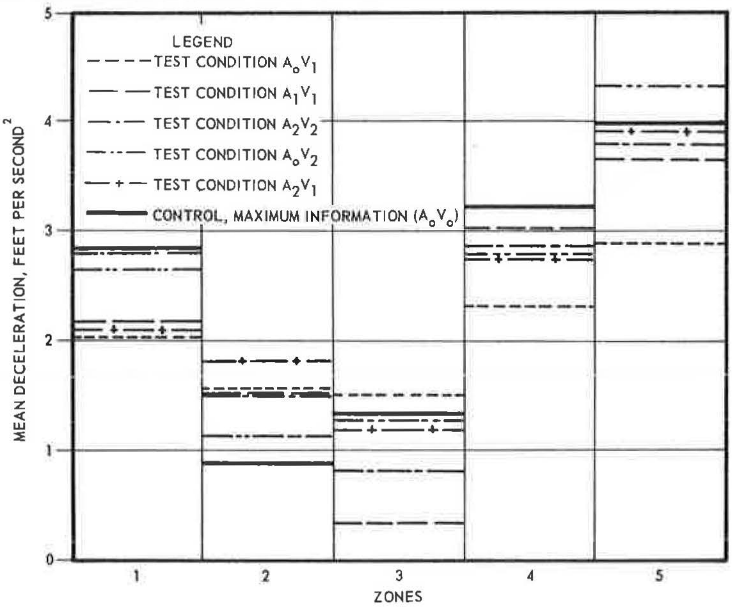


Figure 17. Mean vehicle deceleration of the test conditions in five zones near the exit.

TABLE 7

RANK ORDER AND SIGNIFICANT DIFFERENCES BETWEEN TRIPS FOR THE DEPENDENT VARIABLE DECELERATION
(Significance Level 10 Percent)

Zone	Trip and Mean Deceleration Rate (ft/sec^2)		
	Lowest		Highest
1	Trip 1	Trip 3	Trip 2
	1.82	2.55	2.96
2	Trip 3	Trip 2	Trip 1
	1.11	1.23	1.87
3	Trip 2	Trip 3	Trip 1
	0.83	1.04	1.32
4	Trip 1	Trip 2	Trip 3
	2.74	2.82	2.89
5	Trip 1	Trip 3	Trip 2
	3.57	3.79	3.91

There are no significant differences between those trips underlined.

conditions and which of the trips were significantly different. The results are shown in Table 6, which shows the rank order and significant differences of vehicle decelerations at the various zones along the deceleration lane.

Decelerations were found to be not significantly different among the test conditions in the first two zones or along the first half of the tapered section of the deceleration lane.

In zone 3, in the second half of the tapered deceleration section, the control group (maximum information) and condition A_0V_1 (least information) decelerated at significantly higher values than test condition A_1V_1 with both visual and audio advance information.

In zones 4 and 5, which are located at the full-width section of the deceleration lane and on the exit ramp, the control group decelerated at a significantly higher value than test condition A_0V_1. Also in zone 5, test condition A_0V_2 with visual advance and exit information decelerated at a significantly greater value than test condition A_0V_1.

The control group had high decelerations all through the deceleration lane except in zone 2, located near the midpoint of the tapered deceleration section. Test condition A_0V_1, with only an advance visual message, had high decelerations in zones 2 and 3 along the tapered section of the deceleration lane, and the lowest decelerations in zones 1, 4 and 5, at the beginning of the tapered section of the deceleration lane and along the full width of the deceleration lane.

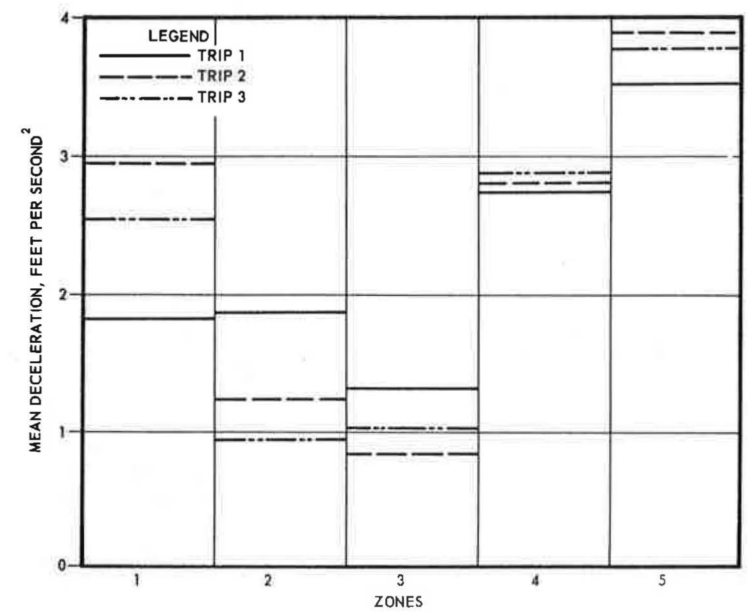


Figure 18. Mean vehicle deceleration for each trip in five zones near the exit.

TABLE 8

RANK ORDER AND SIGNIFICANT DIFFERENCES BETWEEN THE TEST CONDITIONS FOR THE DEPENDENT VARIABLE LATERAL PLACEMENT
(Significance Level 10 Percent)

Position	Test Condition and Mean Lateral Placement (ft)					
	Lowest					Highest
1			(Not analyzed)			
2	A_1V_1	A_0V_1	A_2V_2	A_2V_1	A_0V_2	Control
	4.37	5.51	5.55	5.55	5.57	5.65
3	A_2V_2	A_1V_1	A_0V_1	A_2V_1	Control	A_0V_2
	5.81	5.86	5.99	6.28	6.49	6.77
4	A_2V_2	A_1V_1	A_0V_1	A_2V_1	A_0V_2	Control
	5.80	6.03	6.28	6.90	7.13	7.39
5	A_0V_2	A_0V_1	A_2V_1	A_1V_1	A_2V_2	Control
	7.30	7.38	7.52	7.68	7.93	8.20
6			(No data collected at this position)			
7	A_0V_1	A_2V_2	A_2V_1	Control	A_1V_1	A_0V_2
	5.90	5.98	6.20	6.28	6.59	6.77

There are no significant differences between those conditions underlined.

The high decelerations for test condition A_0V_1 with least information in the early stages of the diverging maneuver may be an indication of the uncertainty of the drivers as to the location of "Random Road."

Mean vehicle decelerations for each of the groups in each of the five zones are shown in Figure 17. The results of the comparison tests between the three trips are given in Table 7.

A significant difference between decelerations during the three trips was found in zones 1 and 2 in the vicinity of the beginning of the tapered deceleration section. The mean deceleration was significantly lower during the first trip than during the second trip in zone 1, and in zone 2 the rate during the first trip was significantly higher than during trip 3. However, these data do not provide evidence to indicate conclusively the effects of learning on deceleration. Mean vehicle deceleration rates for each trip in the five zones are shown in Figure 18.

Analysis of Variance, Lateral Placement Data—The term lateral placement, as used in this report, refers to distance between the pavement edge and center of test cars at various positions along the test section. In view of the limitations of the lateral placement data produced by the bypassing of the placement traps at positions 2 and 5, the analysis of variance results for these data must be interpreted with caution. No analysis of variance was performed on the placement data at position 1 because test vehicles traveled in both the median lane and the outside travel lane of the freeway at this position.

The analysis of variance indicated significant differences between vehicle placements of the test conditions at positions 3 and 4 (along the tapered deceleration section), and position 5 (beginning of the full 12-ft width of deceleration lane), and position 7 (on the exit ramp). There were no significant differences found between

TABLE 9

RANK ORDER AND SIGNIFICANT DIFFERENCES BETWEEN TRIPS FOR THE DEPENDENT VARIABLE LATERAL PLACEMENT
(Significance Level 10 Percent)

Position	Trip and Mean Lateral Placement (ft)		
	Lowest		Highest
1		(Not analyzed)	
2	Trip 2	Trip 3	Trip 1
3	Trip 3	Trip 2	Trip 1
4*	Trip 3	Trip 2	Trip 1
5	Trip 3	Trip 1	Trip 2
6		(No data collected at this position)	
7	Trip 2	Trip 3	Trip 1

There are no significant differences between those trips underlined.

*The significant ranges at position 4 were not detectable with Scheffe's procedure and were estimated using Duncan's procedure.

vehicle placements of the test conditions at position 2 (on the freeway just prior to the beginning of the tapered section of the deceleration lane).

The vehicle placements during the three trips were significantly different at all positions except position 5. The results of the comparison tests between vehicle lateral placements of the test conditions are shown in Table 8. The vehicle placements of test cars in test condition A_2V_2, provided with most information, were found to be generally lower in value than those of other test conditions and not significantly different from those of the control group except at position 4, located at the beginning of the full-width deceleration lane. This indicates that vehicles in test condition A_2V_2 were traveling nearer to the outside or shoulder edge of the pavement than other test conditions. In other words, the entry of test condition A_2V_2 vehicles into the deceleration lane was earlier than other test groups. Vehicle placements in the control group (maximum information), however, were generally higher in distance from the pavement edge than those of other test conditions. Drivers in the control group may have exited at this ramp more casually than the test drivers because these drivers were more familiar with the general configuration and layout of the exit ramp than any of the test condition drivers.

The results of comparison tests indicated that the vehicles in test condition A_2V_2 were traveling significantly nearer to the outside or shoulder edge of the pavement than test condition A_0V_2 (visual advance and exit information) at positions 3 and 7, and significantly nearer the edge than the control group at position 4. At position 5, the beginning of the full width of deceleration lane, vehicle placements in test condition A_0V_2 were significantly nearer to the outside edge of the lane than the control group vehicles. It was at this position that the highest frequency of vehicles bypassing the traps occurred.

The results of the comparison tests of vehicle lateral placements for the three trips are shown in Table 9. The vehicle placements during the first trip were significantly farther from the outside edge of the pavement at positions 2 and 3 than during the second and third trips. These data indicate that drivers during the second and third trips were familiar with the deceleration lane and exit ramp, and made more efficient use of the deceleration lane by traveling closer to the shoulder edge of the pavement. Vehicle placements were also significantly farther from the edge during trip 1 than during trip 3 at position 4, and significantly farther from the edge than during trip 2 at positon 7. There were no significant differences in lateral placements between the three trips at position 5, the beginning of the full width of deceleration lane.

Secondary Experiment (1964)

The analysis of data for the secondary experiment consisted of the determination of significant differences in speed between the test and control vehicles for the three experimental conditons. The results of the analysis are shown in Table 10.

The vehicle speeds of the test and control vehicles were found to be not significantly different during the route information and the disabled vehicle experiments. During the grass cutting activity, however, the test vehicle speeds were found to be significantly lower than the control vehicle speeds. This may indicate the effectiveness of radio communication in warning motorists of potential hazards along the highway. The nonsignificant difference in speeds between the test and control vehicles during the disabled vehicle experiment indicates the possibility that, because of the frequency with which urban motorists encounter disabled vehicles, this situation is not considered potentially hazardous or unusual by the drivers.

TABLE 10

RANK ORDER AND SIGNIFICANT DIFFERENCES BETWEEN THE TEST CONDITIONS FOR THE DEPENDENT VARIABLE SPEED UNDER DIFFERENT EXPERIMENTAL CONDITIONS (Significance Level 10 Percent)

Experimental Condition	Driver Speed	
	Lowest	Highest
Disabled vehicle	TEST	CONTROL
Grass cutting	TEST	CONTROL
Route information	TEST	CONTROL

The Experiment of 1965

The results of the analysis of variance and multiple range tests employed for various comparisons of speed and acceleration are shown for (a) the entire test

section, (b) the vicinity of advance information point, and (c) the vicinity of exit information point.

Analysis of Variance, Vehicles Speeds--The following comparisons show the rank order and significant differences between the various test conditions for mean speed in miles per hour. There are no significant differences between the mean speed of those test conditions underlined. Level of significance used is 10 percent.

A. Test Conditions

(a) Entire test section

Test Condition	A_2V_0	A_1V_2	A_2V_2	A_0V_0	A_0V_2	A_1V_1	A_1V_0	A_2V_1	A_0V_1
Mean Speed	49.44	49.44	48.90	48.74	48.35	47.78	47.51	45.55	45.41

(b) Vicinity of advance information point (positions 1 through 6)

Test Condition	$A_1V^*_{(2)}$	$A_{(2)}V_0$	$A_0V_{(2)}$	$A_{(2)}V_{(2)}$	A_0V_0	A_1V_0	A_1V_1	$A_{(2)}V_1$	A_0V_1
Mean Speed	50.08	49.23	48.49	48.38	48.33	48.31	47.63	46.67	46.33

(c) Vicinity of exit information point (positions 6 through 11)

Test Condition	A_0V_0	A_2V_2	A_2V_0	A_1V_1	A_1V_2	A_1V_0	A_0V_2	A_0V_1	A_2V_1
Mean Speed	51.54	49.75	49.68	48.92	48.88	48.06	47.42	45.65	44.60

In the above analysis it can be seen that test condition A_2V_1 had a relatively low average speed as compared to test conditions with about the same amount of information. This was an unexpected result because it consisted of almost maximum information and a better performance was expected. However, volume counts made during the nights of the experiment showed a relatively high volume (1,116 vehicles/hr) for test condition A_2V_1 compared to the average volume of 861 vehicles/hr for other test conditions. The film analysis also showed that cars approached the test section in platoons and traveled through in the same manner. The low speeds of test condition A_2V_1 may be due to the fact that, for this particular night, test subjects might have been under the influence of the general traffic.

In the comparison of mean vehicle speeds of the nine test conditions employed in the experiment for the entire test section (see A-a), the results of the analysis of variance and multiple range tests indicated that the performance of test subjects in all test conditions except A_2V_1 and A_0V_1 did not significantly differ from each other. Test vehicles in test conditions A_2V_1 and A_0V_1 had different performances by being significantly different from those in test conditions A_2V_0, A_1V_2, A_2V_2 and A_0V_2, which had higher mean speeds closer to that of the control group.

*The subscript 2 in parentheses indicates that although two messages were given for that particular test condition, measurements were taken in the vicinity of advance information point and therefore no exit information had been given yet (e.g., $A_{(2)}V_1 = A_1V_1$).

In the vicinity of the advance information point the performance of all test subjects was not significantly different from the control group no matter what guidance information was given (see A-b). However, test conditions $A_{(2)}V_1$ and A_0V_1 with lower mean speeds were significantly different from test condition $A_1V_{(2)}$.

In the vicinity of the exit information point test vehicles in test conditions A_1V_0 and A_0V_1 with only one level of advance information had lower mean speeds and differed significantly from the control group, while test conditions with two or more messages, except A_2V_1 and A_0V_2, did not differ significantly (see A-c). The different behavior of A_2V_1 has already been explained but in the case of A_0V_2, this may well have been an indication of the reaction of drivers to conventional highway signing. In other words, visual advance and exit information caused a greater reduction in the speed of test subjects than audio advance and exit information. This also indicates the higher and more uniform speeds of test cars receiving audio messages than those receiving visual messages. Test conditions A_0V_1 and A_2V_1 were also found to have significantly lower speeds than test conditions A_2V_2 and A_2V_0 for this area of the test section.

These comparisons of mean speeds of all test conditions with respect to the entire test section and portions of the test section indicated that, except for test conditions A_2V_1 and A_0V_2, test subjects in test conditions with both advance and exit information had higher mean speeds than test subjects in test conditions with less information. This indicated that well-informed test drivers did not have to travel at low speeds in search of the exit ramp.

Similarly, vehicles in the control group had speeds comparable to those in test conditions with substantial information. Vehicles in the control group were not given any information at all along the test section. These vehicles drove normally along the test section and no uncertainty was involved as to the location of the exit ramp at the end of the test section.

B. Speed Measurement Positions

(a) Entire test section

Position	6	7	1	8	4	9	5	10	3	11	2
Mean Speed	50.37	50.16	48.52	48.18	48.12	47.96	47.83	47.60	46.64	46.07	45.57

(b) Vicinity of advance information point

Position	6	1	4	5	3	2
Mean Speed	50.65	48.86	48.35	47.87	47.01	45.91

(c) Vicinity of exit information point

Position	7	6	8	9	10	11
Mean Speed	50.01	49.88	48.07	47.90	47.64	46.09

The analysis of variance and multiple range tests for average speeds at the eleven speed measurement positions showed that there were significant differences between critical positions (positions where some kind of information was being given) and various other positions.

TABLE 11

RANK ORDER AND SIGNIFICANT DIFFERENCES BETWEEN MEAN SPEED OF TEST CONDITIONS AT THE VARIOUS SPEED POSITIONS

Position	Test Condition								
1*	A_2V_0 50.69	A_1V_2 50.15	A_0V_2 49.33	A_2V_2 49.14	A_1V_1 48.00	A_1V_0 47.82	A_0V_0 47.67	A_2V_1 47.56	A_0V_1 45.73
2	$A_0V_{(2)}$ 47.83	$A_{(2)}V_0$ 47.31	$A_1V_{(2)}$ 47.08	$A_{(2)}V_{(2)}$ 46.36	A_0V_0 45.75	A_1V_1 44.77	$A_{(2)}V_1$ 44.00	A_1V_0 43.82	A_0V_1 43.45
3	$A_1V_{(2)}$ 49.75	$A_{(2)}V_{(2)}$ 47.64	$A_{(2)}V_0$ 47.50	A_0V_0 47.42	$A_0V_{(2)}$ 47.17	A_1V_1 46.29	A_1V_0 45.55	$A_{(2)}V_1$ 45.00	A_0V_1 43.36
4	$A_1V_{(2)}$ 50.83	$A_0V_{(2)}$ 49.08	A_0V_0 48.33	A_1V_0 48.27	A_1V_1 48.24	$A_{(2)}V_0$ 48.19	$A_{(2)}V_{(2)}$ 47.29	$A_{(2)}V_1$ 46.88	A_0V_1 46.27
5	$A_1V_{(2)}$ 51.92	$A_{(2)}V_{(2)}$ 49.00	$A_{(2)}V_0$ 48.75	A_1V_0 48.73	A_0V_0 47.42	$A_0V_{(2)}$ 47.17	A_1V_1 46.88	$A_{(2)}V_1$ 45.75	A_0V_1 45.36
6	$A_1(V_2)$ 52.25	$A_{(2)}V_0$ 52.12	A_0V_0 51.92	A_1V_0 51.36	$A_{(2)}V_{(2)}$ 50.86	$A_0V_{(2)}$ 49.92	A_0V_1 49.64	A_1V_1 49.59	$A_{(2)}V_1$ 46.63
7	$A_{(2)}V_0$ 52.75	A_0V_0 52.42	$A_{(2)}V_{(2)}$ 51.57	$A_1V_{(2)}$ 51.50	A_1V_1 50.35	A_1V_0 49.73	$A_0(V_2)$ 49.58	A_0V_1 47.00	$A_{(2)}V_1$ 46.69
8	A_0V_2 50.75	A_2V_0 50.25	A_1V_2 49.25	A_2V_2 49.14	A_0V_0 48.67	A_1V_1 48.18	A_1V_0 47.55	A_0V_1 45.18	A_2V_1 44.69
9	A_2V_0 50.25	A_2V_2 49.36	A_0V_0 49.00	A_1V_1 48.41	A_0V_2 48.33	A_1V_2 48.25	A_1V_0 47.27	A_2V_1 45.44	A_0V_1 44.64
10	A_0V_0 49.92	A_2V_2 49.43	A_2V_0 48.88	A_1V_1 48.47	A_1V_2 47.17	A_0V_2 47.17	A_1V_0 47.09	A_2V_1 44.94	A_0V_1 44.91
11	A_2V_2 48.14	A_0V_0 47.67	A_2V_0 47.12	A_1V_1 46.71	A_1V_2 46.17	A_0V_2 45.50	A_1V_0 45.45	A_0V_1 44.00	A_2V_1 43.50

*At position 1 no information was given about test condition employed in the experiment.

The comparison of mean speeds at the eleven speed measurement positions along the test section indicated that mean speeds at position 6 and 7 were significantly different from those at positions 2, 3 and 11 (see B-a). Positions 6 and 7 were located after advance information had been given and had higher speeds than positions 2 and 3, located during advance information, and 11, located after exit information. It was also found that the approaching speed at position 1 was not significantly different from positions 6 and 7, thus indicating that test subjects showed a tendency to slow down after they received advance and exit information (visual or audio).

In the vicinity of the advance information point, which included positions 1 through 6, and in the vicinity of the exit information point, which included positions 6 through 11, the slowing down of test vehicles was evidenced as indicated by the significant differences between positions after advance and before any exit information and positions during advance information and after exit information (see B-b and B-c).

The analysis of speeds at the speed measurement positions indicated that the test subjects approached the test section at a certain speed, then a reduction in speed was noticed after receiving advance information. The subjects then increased their speeds toward the middle of the test section after which they again slowed down after exit information was given in order to enter the deceleration lane.

Other analysis of variance tests made for each position with respect to all test conditions did not show any significant differences between any of the test conditions. Table 11 shows the rank order for speeds at the various positions for each test.

Analysis of Variance, Accelerations—Rank order and significant differences for acceleration in ft/sec² are given below for various test conditions. A 10 percent significance level is used in all cases. There are no significant differences between those conditions underlined.

A. Test Conditions

(a) Entire test section

Test Condition								
A_2V_2	A_1V_2	A_1V_0	A_2V_0	A_2V_1	A_0V_1	A_0V_0	A_1V_1	A_0V_2
Mean Acceleration								
0.093	0.051	-0.045	-0.183	-0.201	-0.210	-0.250	-0.255	-0.659

(b) Vicinity of advance information point (positions 1 through 6)

Test Condition								
$A_1V_{(2)}$	$A_{(2)}V_{(2)}$	A_1V_0	$A_{(2)}V_0$	A_0V_1	A_0V_0	A_1V_1	$A_{(2)}V_1$	$A_0V_{(2)}$
Mean Acceleration								
0.642	0.558	0.449	0.209	0.105	0.058	-0.113	-0.302	-0.404

(c) Vicinity of exit information point (positions 6 through 11)

Test Condition								
A_2V_1	A_2V_2	A_0V_1	A_1V_1	A_1V_2	A_1V_0	A_2V_0	A_0V_0	A_0V_2
Mean Acceleration								
-0.224	-0.372	-0.416	-0.459	-0.509	-0.545	-0.584	-0.659	-0.744

The data analysis for mean accelerations of test subjects indicated results similar to the speed data. In the analysis of variance and multiple range tests it was found that average accelerations of test subjects in test conditions did not differ significantly from those of the control group (see A-a, A-b and A-c). However, in the entire test section, A_0V_2, with the highest deceleration, was significantly different from test conditions A_2V_2 and A_1V_2 (see A-a). Test conditions A_2V_2, A_1V_2, A_1V_0, A_2V_0 and A_2V_1 had accelerations or decelerations closer to zero than other test conditions, thus indicating a better performance than the other test conditions. It is interesting to note that these test conditions include either audio messages only or visual messages supplemented by audio messages.

In the vicinity of the advance information point (see A-b) mean acceleration of test condition $A_0V_{(2)}$ was significantly different from those of test conditions $A_1V_{(2)}$ and $A_{(2)}V_{(2)}$, but none were significantly different from that of the control group. Test conditions which gave accelerations closer to zero than others were $A_{(2)}V_0$, A_0V_1, A_0V_0 and A_1V_1. This condition indicated that advance information could be given with any mode of communication and the results would not be significantly different from each other.

In the vicinity of the exit information point no test condition was significantly different from another or from the control group and all were decelerations (see A-c). These data indicate that test subjects reduced speed on the freeway before entering the deceleration lane.

B. Acceleration Zones

Acceleration zone indicates the distance between consecutive speed measurement positions. Rank order and significant differences for acceleration between these zones are as follows:

(a) Entire test section

Zone									
5	2	3	6	9	8	4	7	1	10
Mean Acceleration (all test conditions)									
0.653	0.646	0.494	-0.008	-0.048	-0.301	-0.404	-0.627	-0.656	-0.594

(b) Vicinity of advance information point (zones 1 through 5)

Zone

2	5	3	1	4

Mean Acceleration (all test conditions)

0.665	0.629	0.449	-0.660	-0.685

(c) Vicinity of exit information point (zones 6 through 10)

Zone

6	9	8	7	10

Mean Acceleration (all test conditions)

-0.005	-0.034	-0.245	-0.617	-1.606

The significant differences between accelerations for various increments of the test section are as given above. No decisions could be made to indicate which of the test conditions caused the various significant differences between these zones as they are the results of all test conditions combined together. As it was with the speeds at the various positions, the slowing down of test subjects as a result of some kind of information given to them was evidenced. Zone 1 always had a deceleration and generally from zone 2 to about 6 there would be an acceleration. After zone 6 the test cars would decelerate reaching maximum deceleration at zone 10 just before exiting the freeway.

Other statistical analysis of accelerations in each zone with respect to test conditions indicated that there were no significant differences between the accelerations in zones 1, 2, 3, 6, 7, 9, and 10. The significant differences found were at zones 4, 5, and 8. Rank order and significant differences between test conditions at the various zones for acceleration are shown in Table 12. Zones 4 and 5 fall within that portion of the test section along which advance information was being given. Zone 8 is the beginning of the latter portion of the test section along which exit information was being given.

TABLE 12

RANK ORDER AND SIGNIFICANT DIFFERENCES BETWEEN ACCELERATIONS OF TEST CONDITIONS AT THE VARIOUS ACCELERATION ZONES

Zone	Test Condition and Mean Acceleration in Zone								
1	$A_0V_{(2)}$ -0.409	A_0V_0 -0.420	A_0V_1 -0.501	$A_{(2)}V_{(2)}$ -0.616	$A_{(2)}V_0$ -0.697	A_1V_1 -0.749	$A_1V_{(2)}$ -0.749	A_1V_0 -0.792	$A_{(2)}V_1$ -0.809
2	$A_1V_{(2)}$ 1.578	A_0V_0 0.995	A_1V_0 0.991	A_1V_1 0.960	$A_{(2)}V_{(2)}$ 0.874	$A_{(2)}V_1$ 0.585	$A_{(2)}V_0$ 0.157	A_0V_1 0.064	$A_0V_{(2)}$ -0.373
3	A_0V_1 0.955	A_1V_0 0.896	$A_0V_{(2)}$ 0.720	A_1V_1 0.676	$A_{(2)}V_1$ 0.556	$A_1V_{(2)}$ 0.462	A_0V_0 0.326	$A_{(2)}V_0$ 0.223	$A_{(2)}V_{(2)}$ -0.190
4	$A_{(2)}V_{(2)}$ 2.270	$A_1V_{(2)}$ 1.583	$A_{(2)}V_0$ 0.759	A_1V_0 0.551	$A_{(2)}V_1$ -1.397	A_0V_1 -1.445	A_0V_0 -1.606	A_1V_1 -1.923	$A_0V_{(2)}$ -2.309
5	A_0V_0 1.213	A_0V_1 1.091	$A_{(2)}V_0$ 0.873	A_1V_1 0.692	$A_0V_{(2)}$ 0.658	A_1V_0 0.656	$A_{(2)}V_{(2)}$ 0.451	$A_{(2)}V_1$ 0.228	$A_1V_{(2)}$ 0.141
6	$A_{(2)}V_1$ 0.086	A_1V_1 0.070	$A_{(2)}V_0$ 0.067	$A_{(2)}V_{(2)}$ 0.058	A_0V_0 0.042	A_1V_0 -0.084	$A_0V_{(2)}$ -0.096	$A_1V_{(2)}$ -0.125	A_0V_1 -0.219
7	$A_0V_{(2)}$ 0.409	$A_{(2)}V_1$ -0.523	A_0V_1 -0.534	$A_1V_{(2)}$ -0.657	A_1V_0 -0.671	A_1V_1 -0.702	$A_{(2)}V_{(2)}$ -0.801	$A_{(2)}V_0$ -0.826	A_0V_0 -1.236
8	A_2V_1 0.749	A_0V_0 0.364	A_1V_1 0.310	A_2V_2 0.242	A_2V_0 -0.032	A_1V_0 -0.289	A_0V_1 -0.663	A_1V_2 -1.059	A_0V_2 -3.134
9	A_1V_2 0.354	A_0V_0 0.299	A_2V_2 0.028	A_0V_1 0.027	A_1V_1 0.018	A_1V_0 -0.043	A_2V_1 -0.141	A_0V_2 -0.375	A_2V_0 -0.463
10	A_0V_1 -0.878	A_1V_2 -1.018	A_2V_1 -1.288	A_2V_2 -1.387	A_0V_2 -1.666	A_1V_0 -1.668	A_2V_0 -1.885	A_1V_1 -1.922	A_0V_0 -2.480

TABLE 13

ACCELERATION NOISE IN ORDER OF REDUNDANCY OF INFORMATION

Test	Redundancy of Information	No. of Test Cars	Travel Time (sec)	Mean Acceleration Noise ($\bar{\sigma}_a$ = ft/sec²)
A_0V_0	0	12	47.65	1.30
A_0V_1	1	11	51.50	0.89
A_1V_0	1	11	48.50	0.98
A_1V_1	2	17	49.21	1.20
A_0V_2	2	12	48.49	1.70
A_2V_0	2	16	46.79	0.88
A_1V_2	3	12	47.42	1.09
A_2V_1	3	16	51.51	0.89
A_2V_2	4	14	47.67	0.92

In zone 4, test condition $A_0V_{(2)}$ was found to be significantly different from test condition $A_{(2)}V_{(2)}$, the former having a high value of deceleration and the latter a high value of acceleration as compared to other test conditions. Test conditions $A_{(2)}V_0$ and A_1V_0 had accelerations which were closer to zero than other test conditions, thus having rather more constant speeds than others.

In zone 5, test conditions $A_{(2)}V_1$ and $A_1V_{(2)}$ were significantly different from test conditions A_0V_0 and had accelerations closer to zero, thus indicating a better performance than other test conditions.

In zone 8, test conditions A_2V_1 and A_0V_0 were significantly different from test condition A_0V_2, which had a high deceleration and indicated a poor performance. For this zone, which is the beginning of the vicinity of the exit information point, test conditions A_2V_2, A_2V_0 and A_1V_0 gave accelerations or decelerations lower than all other test conditions and closer to zero. These test conditions indicated better performance within this zone than other test conditions which had greater values for acceleration or deceleration.

Acceleration Noise—The values of acceleration noise as given in Table 13 have been computed for each test condition by using accelerations and travel times of each zone of the test section between speed measurement positions. Individual values computed for zones of each test condition when added gave a value for acceleration noise.

An analysis of variance and multiple range test for acceleration noise showed significant differences between some of the test conditions employed. The rank order and significant differences at the 10 percent level between test conditions and control group are shown below.

Test Condition

A_0V_2	A_0V_0	A_1V_1	A_1V_2	A_1V_0	A_2V_2	A_0V_1	A_2V_1	A_2V_0
Mean Acceleration Noise ($\bar{\sigma}_a$ = ft/sec²)								
1.70	1.30	1.20	1.09	0.98	0.92	0.89	0.89	0.88

Test conditions with two audio messages generally had more favorable values for acceleration noise than test conditions with two visual messages. Test condition A_0V_2 had the largest value of acceleration noise and was found to be significantly different from test conditions A_2V_2, A_0V_1, A_2V_1 and A_2V_0. Test condition A_0V_0 had a lower value of acceleration noise than A_0V_2 because most of the test subjects had missed the ramp and therefore may have not changed speeds in the vicinity of the exit ramp. However, test condition A_0V_0 had the highest deceleration at zone 10 which is the zone prior to entry into the deceleration lane. For this particular zone test condition A_0V_0 had the highest value of acceleration noise for those test subjects which exited the freeway at the end of the test section.

TABLE 14
PERCENT OF TEST CARS ENTERING DECELERATION LANE AT VARIOUS INCREMENTS

Test Condition and Level of Information	Distance From Point of Taper: 0-100 ft, % Entry of Front Wheel: Right	0-100 ft: Left	100-200 ft: Right	100-200 ft: Left	200-300 ft: Right	200-300 ft: Left	300-400 ft: Right	300-400 ft: Left	400-500 ft: Right	400-500 ft: Left	% of Test Cars Failing to Exit at "Random Road"
A_0V_0	0	0	6.25	0	6.25	6.25	18.75	6.25	0	18.75	68.75
A_0V_1	13.33	0	33.33	13.33	33.34	13.34	6.67	40.00	0	20.00	13.33
A_1V_0	0	0	14.29	0	28.57	7.15	21.43	28.57	0	28.57	35.71
A_1V_1	5.56	0	22.22	0	22.22	33.33	44.44	38.89	0	22.22	5.56
A_0V_2	23.53	0	47.06	5.88	23.53	41.18	5.88	41.18	0	11.76	0
A_2V_0	30.00	0	50.00	15.00	15.00	45.00	5.00	30.00	0	10.00	0
A_1V_2	41.67	0	41.67	16.67	16.66	58.33	0	25.00	0	0	0
A_2V_1	40.00	0	50.00	5.00	10.00	75.00	0	20.00	0	0	0
A_2V_2	57.14	0	35.71	28.57	7.15	50.00	0	21.43	0	0	0

Point of Entry Data—Although the position of entry of both front wheels into the deceleration lane was recorded, the right front wheel was used as the entry indicator. Therefore, the position of the right front wheel shows the distance from the point of taper just after a decision to enter the deceleration lane has been made. The position of the left front wheel indicates the complete entry of test cars into the deceleration lane.

Table 14 gives the percent of cars for each test condition entering the deceleration lane at various increments of the deceleration lane. Values in Table 14 indicate that test subjects behaved differently on entry into the deceleration lane according to the amount of information given to them. About 69 percent of the test subjects participating in test condition A_0V_0 missed the "Random Road" exit merely because the only message given to them was the sign at the gore of the exit ramp. Sixty percent of those in test condition A_0V_0 who exited, entered the deceleration lane at a distance between 300 and 400 ft from the beginning of the taper.

As the amount of information being given was increased it was found that the entry of test subjects into the deceleration lane was made closer to the point of taper. In test condition A_2V_2, 57 percent of the test subjects entered the deceleration lane within the first 100 ft from the point of taper. The values were 40, 42, 30 and 23 percent for test conditions A_2V_1, A_1V_2, A_2V_0 and A_0V_2 respectively. It can also be seen that the entry of the left front wheel, i.e., complete entry of cars into the deceleration lane, followed the same pattern as the right front wheel. In every case where only advance information was given it was found that there were test cars missing the exit ramp. No cars missed the ramp when any one of the messages was exit information.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary of Results

1. The results of the analysis of variance for vehicle speeds indicated the following:

(a) Vehicle speeds over the entire test section in test conditions with substantial information were higher than those in test conditions with less information.

(b) In the vicinity of the advance information point (approximately 0.5 mi in advance of the exit ramp), there was not enough evidence to determine differences in the behavior of test subjects in either vehicle speeds or rates of acceleration no matter what test condition was employed.

(c) In the vicinity of the exit, vehicle speeds in test conditions with both advance and exit information and in the control group were generally higher than those in test conditions with only advance guidance information.

(d) The results of the analysis of variance for vehicle speeds over the entire test section and portions of the test section seemed to indicate that as the amount of information given was increased, vehicles traveled through the entire test section at generally higher speeds. The low vehicle speeds in test conditions with less information may be regarded as an indication of the uncertainty of the driver with regard to the location of the exit ramp.

(e) Vehicle speeds in test conditions A_1V_0 and A_2V_0 (all radio) were not significantly different from those in test conditions A_0V_1 and A_0V_2 (all signs) respectively. This fact may indicate that audio messages were as effective as visual messages.

(f) Comparisons of mean speeds of test conditions at speed measurement positions showed that there were significant differences between some of the positions where information was being given and various other positions where no information was being given. These significant differences were due to the fact that some sort of information was given. Also, the low speeds of test conditions with less information in the vicinity of the exit contributed to this fact.

(g) Vehicle speeds during trip 1 were significantly lower than the speeds during trip 2 along the full-width deceleration lane. During trip 2, test vehicles were familiar with the configuration of the deceleration lane and exit ramp and the higher speeds recorded during trip 2 show the effects of learning.

2. The analysis of accelerations or decelerations indicated the following:

(a) The mean accelerations of vehicles in the various test conditions along the freeway before the exit ramp did not differ significantly from those of the control group. However, vehicles in test condition A_0V_2 showed a poor performance by having high rates of deceleration along the freeway as compared to those in test conditions A_2V_2 and A_1V_2.

(b) Vehicles in test conditions with audio messages and with visual messages supplemented by audio messages had accelerations or decelerations closer to zero along the freeway prior to the exit ramp than those in test conditions with only visual messages.

(c) In the vicinity of the advance information point there were no indications as to which test condition produced better driving performance.

(d) In the vicinity of the exit information point, all test conditions had different decelerations but no significant differences could be found.

(e) Various comparisons of accelerations in acceleration zones along the freeway prior to the exit ramp indicated that test vehicles decelerated after some advance or exit information was given.

(f) No significant differences existed between the decelerations of the test condition vehicles along the first two zones in the tapered section of the deceleration lane. However, significant differences were found in zone 3, just prior to the full-width deceleration lane, and in zones 4 and 5 along the deceleration lane.

(g) The control group vehicles (maximum information, 1964 study) had high values of deceleration just prior to the full-width deceleration lane and along the deceleration lane. This fact indicates that vehicles in the control group which had the highest speeds along the deceleration lane except toward the end of it, decelerated in the full-width deceleration lane because they were familiar with the configuration of the deceleration lane and exit ramp.

(h) Vehicle decelerations in test condition A_0V_1 (least information, 1964 study), were greater just prior to the beginning of the full-width deceleration lane and lower along the deceleration lane than all other test conditions and the control group. The high mean deceleration just prior to the full-width deceleration lane as compared to other test conditions may indicate the amount of uncertainty of the test vehicles in the execution of the diverging maneuver.

(i) The analysis indicated that significant differences between the three trips took place in the early phases of the diverging maneuver.

3. The results of the analysis of the point of entry data are as follows:

(a) Exit information given only in the vicinity of the advance information point was found to be an inadequate amount of information. Even when both audio and visual advance information were given there were test cars missing the exit ramp.

(b) As the amount of information given about an exit ramp was increased, the entry of test cars into the deceleration lane was closer to the beginning of the tapered section of the deceleration lane.

4. The analysis of the lateral placement data indicated the following:

(a) Vehicles in test condition A_2V_2, supplied with most information, traveled generally nearer to the outside or shoulder edge of the pavement and therefore entered the deceleration lane earlier than other test conditions except the control group.

(b) Vehicles in the control group (maximum information, 1964 study) traveled generally farther from the outer or shoulder edge of the pavement than other test conditions. This fact may indicate that drivers in the control group may have exited at this ramp more casually than the other test condition drivers because they were more familiar with the general configuration and layout of the deceleration lane and exit ramp than any of the test condition drivers.

(c) There were no significant differences between vehicle placement of any of the test conditions along the freeway just prior to the beginning of the tapered section of the deceleration lane.

5. The analysis of data for the three repeat trips indicated that the vehicle placements during the first trip were significantly farther from the outside edge of the pavement on the freeway just before the point of taper and at the midpoint of the first half of the tapered section of the deceleration lane than during the second and third trips. This indicates that drivers during the second and third trips were familiar with the deceleration lane and exit ramp, and made more efficient use of the deceleration lane by traveling closer to the shoulder edge of the pavement. Vehicle placements were also significantly farther from the edge during trip 1 than during trip 3 at the midpoint of the tapered section of the deceleration lane, and significantly farther from the edge during trip 2 at the beginning of the off-ramp.

Conclusions

1. The results of data analysis indicated that the provision of roadside radio communication in addition to standard highway signing will be useful to drivers as evidenced in the better performance of test subjects in the test conditions where the amount of information given was maximum or close to maximum.
2. Audio messages were found to be as effective as visual messages, as indicated by the nonsignificant difference in speeds and accelerations along the freeway prior to the exit ramp between test conditions A_1V_0 and A_2V_0 and A_0V_1 and A_0V_2, respectively.
3. The provision of radio roadside communication in addition to standard advance highway signing did not significantly affect the speed and accelerations of motorists traveling at a point one-half mile in advance of the exit terminal.
4. Giving advance guidance information only about the location of an exit ramp was found to be inadequate in the performance of a diverging maneuver from the freeway. This inadequacy was evidenced by the number of test subjects missing the ramp when only advance information was given.
5. When the information given about an exit ramp consisted of both advance and exit information, no test subject missed the ramp and as the amount of information was increased the entry into the deceleration lane was made earlier than otherwise.
6. The significant differences between trip 1 and trip 2 along the full-width deceleration lane and the higher speeds recorded during trip 2 may be an indication of the effects of learning.
7. The analysis of lateral placement data indicated that vehicles traveled generally closer to the outer or shoulder edge under test conditions supplied with most information. This indicates that well-informed vehicles entered the deceleration lane earlier than the less informed. This is evidenced also in vehicle placements during trips 2

and 3 which were significantly closer to the pavement edge than those during trip 1 just before the point of taper and at the midpoint of the first half of the tapered section of the deceleration lane.

8. The control group (maximum information, 1964 study) vehicles traveled along the deceleration lane generally farther from the pavement edge, indicating that motorists in this group were very familiar with the configuration of the deceleration lane and exit ramp and therefore diverged from the freeway more casually than other test condition drivers, and in less uncertain maneuvers.

9. The results of the secondary experiment indicated significantly different vehicle speeds between the test and control vehicles during the grass-cutting experiment. This fact indicates that radio communication could be an effective device in warning motorists of potential hazards along a freeway.

Recommendations

The results of these experiments gave a general idea about the effectiveness of roadside radio communication as a traffic control device. Audio messages are shown to be as effective as visual messages and when given together the performance of test drivers has proven to be generally better than the performance of test drivers with only visual or audio messages.

A radio-signing system which will provide necessary information where needed can be effective and at the same time avoid extensive over-signing. Further research should be conducted on a system basis to investigate the use of radio as a communication system to provide traffic control and driver information and to determine conclusively specific areas of feasibility of roadside radio communication.

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REFERENCES

1. Covault, Donald O., and Bowes, Robert W. A Study of the Feasibility of Using Roadside Radio Communications for Traffic Control and Driver Information. Annual Report No. 1, Engineering Experiment Station, Georgia Institute of Technology, Feb. 1964; also, Highway Research Record 49, pp. 89-106, 1964.
2. Covault, Donald O. Time-Lapse Movie Photography. Traffic Engineering, Vol. 3, No. 6, March 1960.
3. Duncan, D. B. Multiple Range and Multiple F Tests. Biometrics, The Biometric Society, Vol. 11, No. 1-4, 1955.
4. Greenshields, Bruce D. Driving Behavior and Related Problems. Highway Research Record 25, pp. 14-32, 1963.
5. Guenther, William C. Analysis of Variance. Prentice-Hall, Englewood Cliffs, N. J., 1964.
6. Hoppe, Donald A., and Lauer, A. R. Factors Affecting the Perception of Relative Motion and Distance Between Vehicles at Night. HRB Bull. 43, pp. 1-16, 1951.

7. Jones, Trevor R., and Potts, Renfrey B. The Measurement of Acceleration Noise—A Traffic Parameter. Operations Research, Vol. 10, 1962.
8. Jouzy, Neddy C., and Michael, Harold L. Use and Design of Acceleration and Deceleration Lanes in Indiana. Highway Research Record 9, pp. 25-51, 1963.
9. Kendall, Maurice George. The Advanced Theory of Statistics, Vol. II, 3rd Ed. Hafner Publishing Co., New York, 1951.
10. Malo, A. F., Mika, H. S., and Walbridge, V. P. Traffic Behavior on an Urban Expressway. HRB Bull. 235, pp. 19-37, 1960.
11. Powers, Lawrence D. Advance Route Turn Markers on City Streets. HRB Proc., Vol. 41, pp. 483-494, 1962.
12. Schwar, Johannes F., and Rockwell, Thomas H. Sensing and Communications Between Vehicles. Annual Report, Engineering Experiment Station, Ohio State University, 1964.
13. Schoppert, David W., Moskowitz, Karl, Hulbert, Slade F., and Burg, Albert. Some Principles of Freeway Directional Signing Based on Motorists' Experiences. HRB Bull. 244, pp. 30-87, 1960.
14. Solomon, David. The Effect of Letter Width and Spacing on Night Legibility of Highway Signs. HRB Proc., Vol. 35, pp. 600-617, 1956.
15. Taragin, A., and Hopkins, R. C. A Traffic Analyzer: Its Development and Application. Public Roads, Vol. 31, No. 5, pp. 120-124, Dec. 1960.

An Evaluation of the Northway Emergency Telephone System

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•THE need to provide for the safety and convenience of disabled motorists on limited-access highways is receiving considerable attention from state and federal highway agencies. This emphasis is certainly warranted when one considers that 85 percent of the planned 41,000-mile Interstate Highway System lies in rural areas, many of which are remote, sparsely populated, and completely lacking in motorist services. In event of accident, illness, or vehicle failure, the motorist is almost entirely dependent upon assistance from another passing motorist. Quite often the delay while waiting for service is unreasonably long and may, in the event of a severe injury, result in a loss of life.

Emergency communication systems are not new; however, they have generally been limited to urban areas. The purpose of this paper is threefold: (a) to describe the physical and operational characteristics of an emergency telephone system on a rural interstate highway, (b) to report and analyze data obtained from the system, and (c) to outline and recommend continuing research in several related areas.

DESCRIPTION OF SYSTEM

New York State's Adirondack Northway (I-87), when completed, will form an important 178-mile link in the Interstate System between the eastern United States and Canada. From its southern connection with the New York Thruway at Albany, the Northway carries traffic from the New York metropolitan area to the Adirondack Mountains and other upstate recreational areas. It is the primary route between New York City and major Canadian cities such as Montreal and Quebec City (Fig. 1).

The Northway Emergency Telephone System (NETS) has been in operation on the southerly 55 miles of the highway since January 7, 1966, and consists of 222 free telephones spaced at ½-mile intervals on both sides of the facility (Fig. 2).

The call box (Fig. 3) is a standard telephone housing mounted on a treated cedar pole. The box has been coated with reflectorized yellow paint to improve nighttime visibility. The vertical number on the right identifies the location of the box. Immediately above the number are the instructions for using the phone, and on the left is the familiar icebreaker handle which opens the box. The phones are located just off the right shoulder of the highway approximately 12 feet from the edge of the pavement. This gives the motorist ample space to pull his vehicle completely off the pavement while using the phone (Fig. 4). Phones on the northbound side are located directly opposite those on the southbound side to discourage any motorist on foot from crossing the highway.

Each phone is connected by underground cable to the nearest of three State Police substations, permitting the motorist to explain the exact nature of his call to a police officer. The officer is immediately able to supply information or dispatch appropriate assistance. The system is composed of 12 individual circuits and includes a continuous test of circuit continuity for rapid detection of line failure. The phones are distributed among the three substations as follows: Loudonville, 36; Malta, 98; South Glens Falls, 88.

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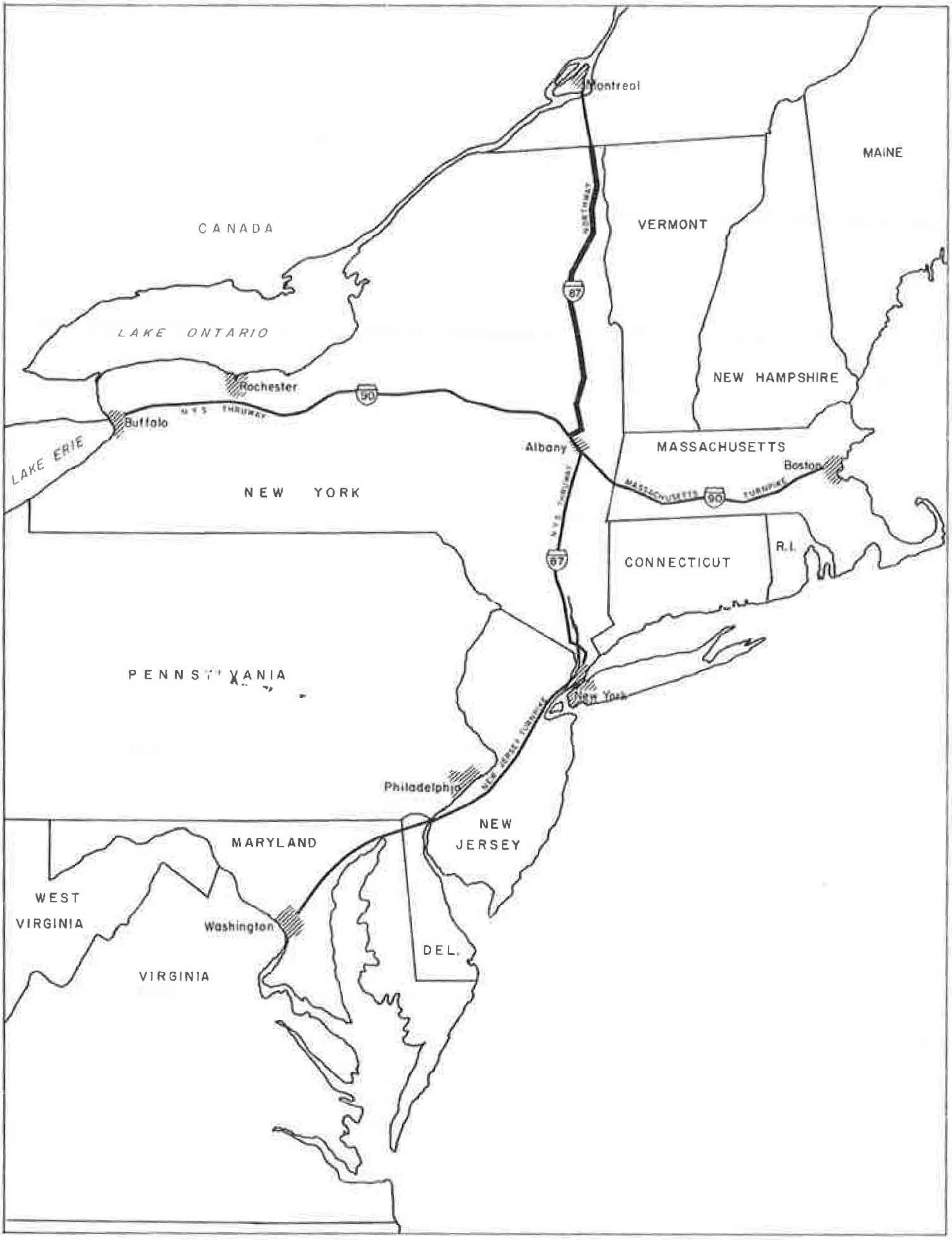

Figure 1. Regional setting of Northway.

Figure 2. Sign on Northway indicating spacing of telephones.

Figure 3. Call box of the emergency telephone system.

Figure 4. Location of call box in relation to roadway.

The telephone equipment is owned and maintained by a private utility company. The initial nonrecurring installation charge was approximately $950 per instrument. The monthly rental rate is about $15 per phone and includes repairs or replacement of material and equipment. Experience during the first year of operation should determine the future maintenance cost.

All incoming calls are recorded and coded by the State Police on the NETS log sheet which was developed by the Department of Public Works to facilitate electronic data processing. At the end of each month, these forms are transmitted to the Department of Public Works for additional coding and keypunching. A computer program has been developed which produces monthly and cumulative summaries of call information as well as a complete listing of data pertaining to each call.

PRELIMINARY RESULTS AND ANALYSIS

An analysis of the number and type of calls received during the first 10 months of operation indicates that the telephone system is providing a needed service to motorists; nearly 4000 calls had been received since January 7, 1966. Moreover, the system was immediately accepted by the motorists as a means for obtaining aid in that they made over 300 calls during the first month of operation. The monthly variation in the frequency of calls is shown in Figure 5. (January is from the 7th through the 31st.)

The sharpest increase in call activity occurred during June, July, and August when more than 40 percent of the calls were received. The number of calls ranged from a high of 660 in July to a low of 228 in October.

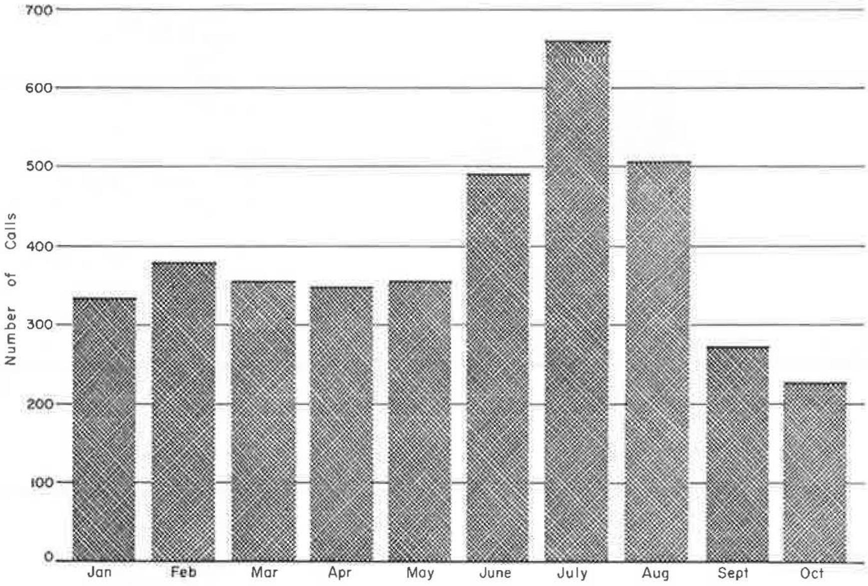

Figure 5. Monthly call frequency.

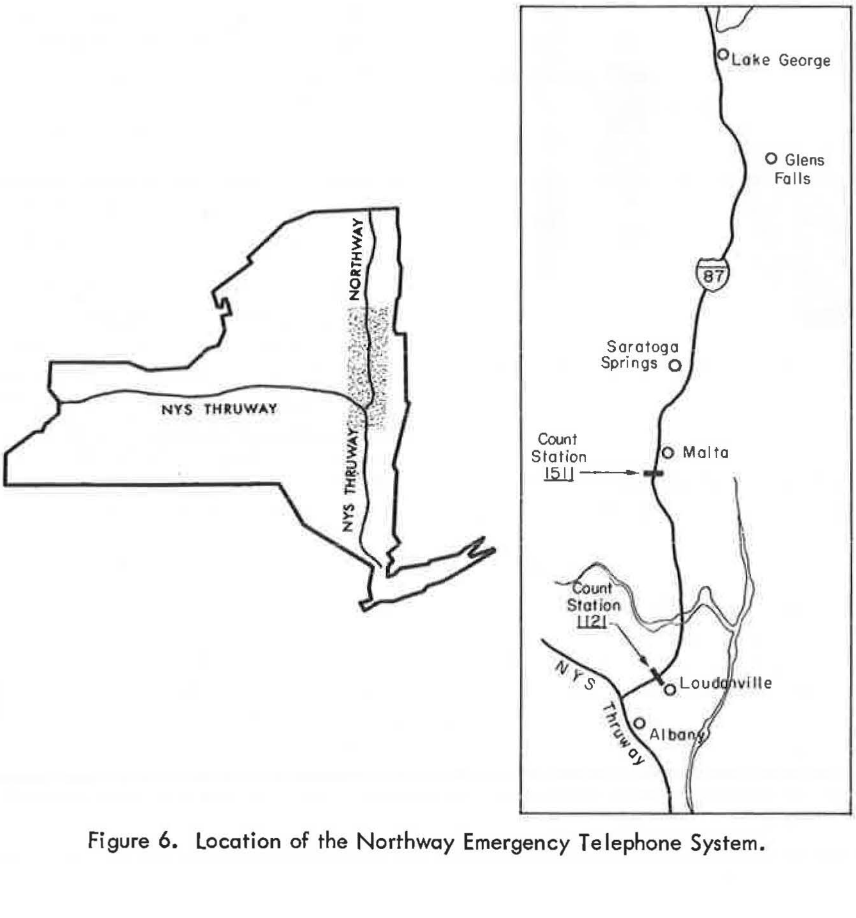

Figure 6. Location of the Northway Emergency Telephone System.

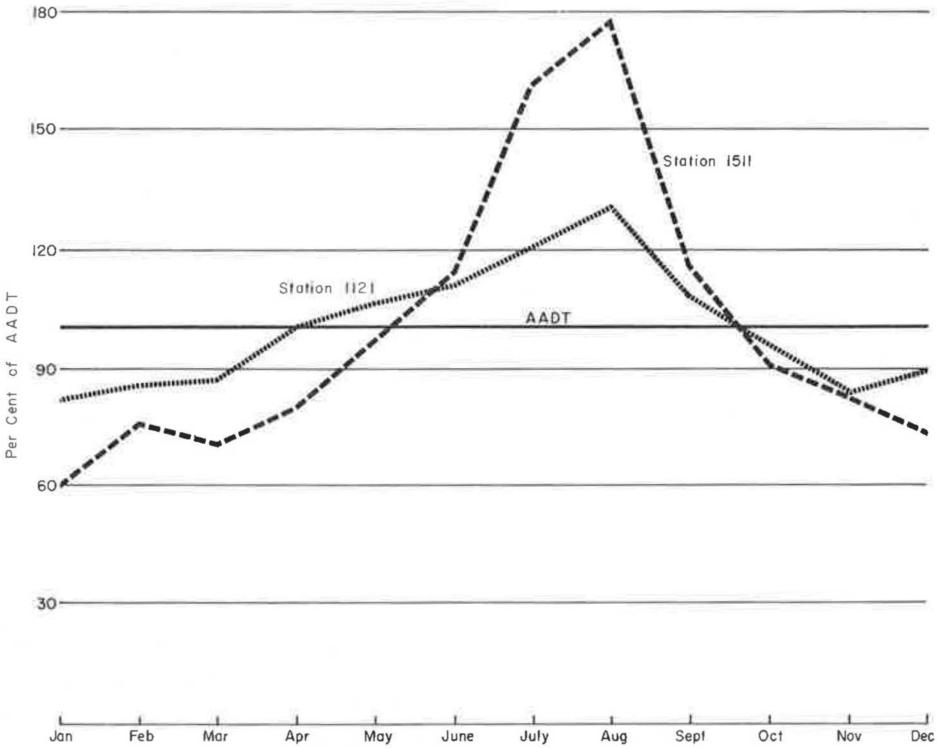

Figure 7. Annual traffic pattern.

The preliminary analysis of NETS usage has dealt primarily with the relationship between call activity and traffic volume. The data sources for traffic volume are two continuous count stations located on the Northway as shown in Figure 6. The lower station (1121) is located in the Loudonville section just north of the interchange with the New York State Thruway. This station is classified in the New York State traffic counting program as representing an urban route with some recreational characteristics. The section of the Northway represented by this station carries heavy commuter traffic between Albany and suburbs to the north. Beyond the suburbs, the area served by the Northway is characterized by many recreational and resort areas such as the famed Saratoga Race Track and the Adirondack State Park. The upper continuous count station (1511) is located within the Malta section and is classified as representing a recreation facility.

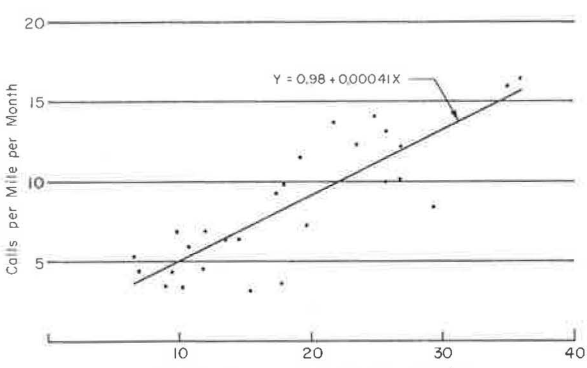

Figure 8. Relation between calls and monthly traffic volume.

Figure 7 illustrates the monthly variation in traffic volume as a percentage of AADT for each station. Station 1511 has the familiar summer peak associated with a recreational route. Station 1121 also has a summer peak, but it is much less pronounced.

The relationship between call activity and traffic volume was investigated through regression analysis.

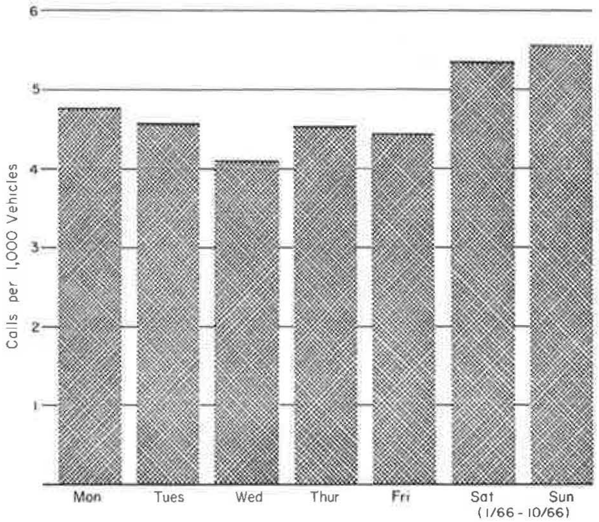

Figure 9. Calls per thousand vehicles by day (Loudonville count station 1121).

The number of calls per roadway mile each month was compared with the monthly average daily traffic; the results are shown in Figure 8. The correlation coefficient of 0.84 indicates the importance of the relationship between traffic volume and the number of calls. As volume increases, the number of calls can also be expected to increase.

The calling rate is expressed as the average number of calls per thousand vehicles. It is based on the actual count of the number of vehicles passing a continuous count station and not on vehicle-miles of travel. While vehicle-miles of travel can be determined for the entire length of the Northway, it is difficult to assign specific amounts to segments of the facility from which calls are received. On the other hand, the calls received from the telephones in the vicinity of the continuous count station can be meaningfully and more accurately related to the number of vehicles passing that station. The following discussion of the calling rate, therefore, pertains to calls received from the Loudonville reporting section and to traffic volumes recorded at the continuous count station located in that section.

The daily calling rate (Fig. 9) varies little during weekdays; however, the weekend rate is considerably higher. The weekend average of 5.46 calls per thousand vehicles is 22 percent greater than the weekday average of 4.48.

Figure 10 shows the hourly variation in the calling rate. The mean hourly calling rate for the Loudonville section of the Northway is 4.66 calls per thousand vehicles passing count station 1121. As shown in the figure, the rate varies from a high of 10.7 between 4:00 and 5:00 a.m. to a low of 2.7 between 9:00 and 10:00 p.m. Of particular interest is the fact that the calling rate between midnight and 7:00 a.m. consistently

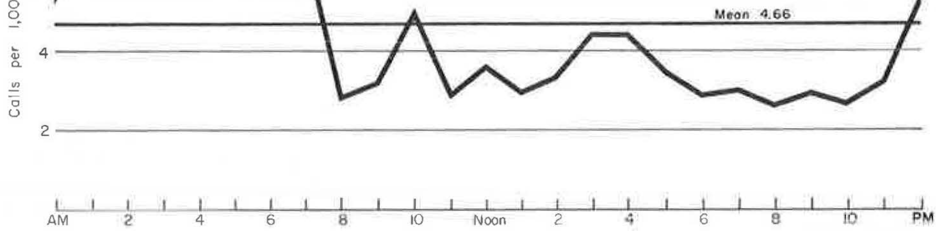

Figure 10. Calls per thousand vehicles by hour (Loudonville count station 1121).

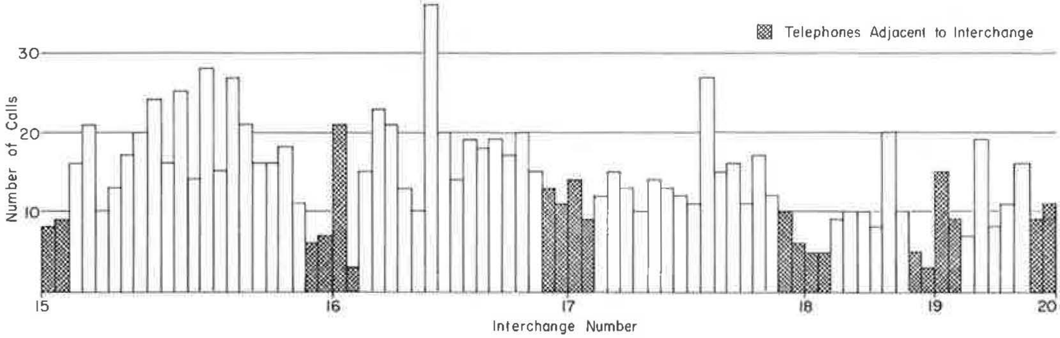

Figure 11. Number of calls by box number sequence between interchanges.

exceeds the mean, while it is generally less than the mean during the remainder of the day. Perhaps this reflects the motorist's reluctance to leave a rural facility at night in search of fuel, repairs, or other assistance.

Individual telephone use has varied considerably, ranging from a low of one to a high of 62 calls during the first 10 months of operation. A pattern was observed (Fig. 11) showing that telephones between interchange areas experienced much more frequent use than those located within or near interchanges. (The telephones within the limits of each interchange were used an average of only 12.5 times as compared to the overall average use of 18.) This suggests that a motorist experiencing difficulty in the immediate area of an interchange will most often attempt to exit and locate a service station himself.

The number of calls per million vehicles-miles of travel for each of the three reporting stations did not vary substantially from the overall average of 15.1. Values of 16.2, 13.5 and 15.6 were observed for Loudonville, Malta, and South Glens Falls, respectively.

Passenger car drivers were responsible for 77 percent of the calls, while the 714 calls from truckers represent an additional 21 percent of telephone use. Motorcyclists and bus drivers accounted for the remaining 2 percent. It is particularly interesting that while trucks represent 11 percent of the traffic volume, they account for 21 percent of the calls. Practically all truck calls requested vehicular service. This fact underscores the value of an emergency communication system to the efficient transportation of goods.

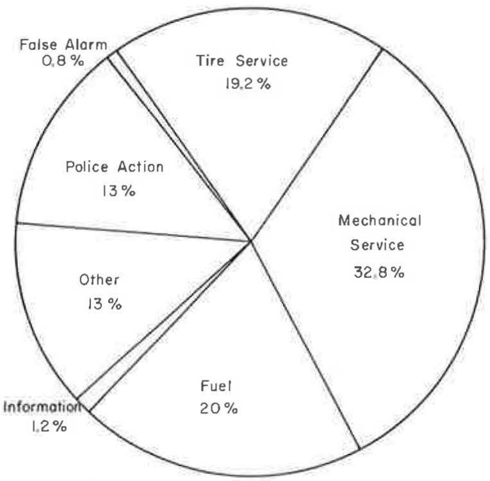

Figure 12. Percent of calls by type of assistance needed.

All incoming calls were classified by type of assistance requested (Fig. 12). Vehicle services, for which more than three-fourths of the calls were made, include requests for gas and tire and mechanical repairs. Police action calls were made primarily to report motor vehicle accidents and traffic violators, the latter including several drivers traveling the wrong way. False alarms amounted to less than 1 percent of the total calls.

AREAS OF FURTHER STUDY

Prior to the collection and preliminary analysis of the NETS data, little, if any, information was available concerning the use of an emergency telephone system on

a limited-access rural highway. While a few basic relationships have become evident from this study, several factors remain to be investigated before sound policies or warrants can be established for expanding similar installations to other rural highways on the Interstate System. Several areas where additional research is needed are described in the following. Some of these will be investigated as part of a continuation of the present study.

Factors Affecting Use

To provide the best service available for any customer in a competitive market, a service agency must know its customers. This fact is doubly important on the Northway because we are dealing with emergency services. A close analysis of the type of service requested reveals a great deal about the disabled motorist. For example, 20 percent of the time he needs only fuel in order to resume his journey.

As was shown in Figure 9, the calling rate on the average weekend is more than 20 percent higher than it is on the average weekday. This suggests that the Monday-to-Friday commuter has a different impact on phone use than the social-recreational traveler on Saturday or Sunday.

There are several factors which may contribute to the use of an emergency telephone. These include the purpose of the trip, the length of the trip on the Northway, the drivers' familiarity with the area, the vehicle type, the day of the week, and the hour of the day. To obtain data on trip characteristics, a screenline inventory is planned. The data from this inventory will be compared with data obtained from the motorists using the telephones. These comparisons will reveal the differences, if any, between the trip characteristics of travelers who use the telephones and those of all the travelers on the Northway. The Northway is an excellent laboratory for this research not only because of the heavy influence of the tourist during the summer but also because of the variation in the degree of development adjacent to the facility.

Physical Features

As might be expected, a few operational difficulties have emerged since the system was installed. A motorist stopping along the shoulder is sometimes unable to see a call box in either direction and often starts for the more distant phone. This problem is intensified at night, and at least one motorist walked nearly a half mile, unaware that another call box was located only a few feet away in the opposite direction. Further, it is often difficult for the nighttime user to identify the number of the box from which he is calling. Some modification of the present system is contemplated to eliminate these undesirable features.

The decision to space the Northway telephones at one-half mile was subjective and may not represent the ideal distance between instruments on any emergency communication system. The spacing between adjacent phones, not necessarily constant, might be based on the ability of the potential user to locate and identify a telephone from any point along the roadway. Identification may thus be a function of horizontal and/or vertical curvature, as well as a function of the level of signing effort. Many areas of the country experience extreme climatic conditions and, while this criterion suggests a maximum spacing, it might be modified by the ability of the user to get to the instrument under adverse weather conditions.

Benefits

Today the highway administrator is more than ever aware of the increasing competition for the dollar invested in transportation systems. To advocate an emergency communication system as extensive as NETS, he must have a firm evaluation of the benefits attainable. The benefits of an emergency communication system may be expressed in terms of time savings, service, safety, and security (1). It may be difficult to attach a dollar value to each benefit; however, the utility of each item can be examined. Time savings credited to an emergency communication system in an urban area might be considerable, since a disabled vehicle often creates congestion and delays

to other motorists using the facility. Motorists on rural highways, however, seldom experience the congestion normally associated with urban routes; thus a rural communication system results in time savings only for the disabled motorist. It is unlikely that savings in driver time alone will completely offset the cost of a communication system. Time savings do, however, constitute an important part of the total benefit to the motorist.

Time savings can also be evaluated in a slightly different context. Prior to the installation of the Northway Emergency Telephone System, a portion of the New York State Troopers' day was spent assisting disabled motorists. While this service can never be completely eliminated, a reduction in stopped time of the patrol vehicle has been observed. This saving results in increased surveillance time, thus offering better service to the motorist.

Attention has been given nationally to establishment of emergency aid stations so as to reduce the time lag between a motor vehicle accident injury and the arrival of competent medical assistance. It is realistic to assume that an emergency communication system may be instrumental in reducing the discomfort and suffering which result from accident injuries on the facility. Installation of an emergency communication system nationally may eliminate or reduce the need for aid stations.

An emergency communication network can also be utilized as an early warning system to alert the police of the existence of a hazardous condition. This can take the form of a traffic violator such as a wrong-way driver, or an actual roadway hazard such as pavement icing or a foreign object in the road. The element of security is perhaps the most difficult to evaluate. It may be expressed in terms of the value of knowing that prompt assistance is readily available in event of an emergency. The importance of this knowledge to the motorist in his decision to select one route over another is a subject for further investigation.

SUMMARY

The Adirondack Northway presents a unique opportunity to study the operation of an emergency telephone system under a variety of conditions. Experience thus far indicates that NETS has been well accepted by the traveling public and is providing a much-needed service. Of course, there remain many unanswered questions concerning emergency communication systems. However, New York State has taken the initiative in establishing a complete system on an Interstate highway. It is anticipated that the evaluation of this system over the next few years will be a significant contribution to the field of emergency communications.

REFERENCE

1. Pogust, F., Kuprijanow, A., and Forster, H. Means of Locating and Communicating With Disabled Vehicles—Interim Report. Highway Research Board, NCHRP Rept. 6, 1964.

A Nationwide Study of Freeway Merging Operations

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This paper introduces the research project entitled "Gap Acceptance and Traffic Interaction in the Freeway Merging Process" which forms a part of a four-year program on freeway merging undertaken by the Bureau of Public Roads. Field studies for the collection of data were performed on a nationwide basis at a number of selected entrance ramps, utilizing an aerial photographic technique. This technique, the data reduction methods and the study sites selected are described in detail.

Data editing routines and the analysis of the data for basic traffic parameters are discussed and some of these parameters are used to illustrate the merging operation at each study site. The qualitative effect of various geometric elements on the operation, as mirrored by the traffic parameters of volume, density, speed and acceleration noise, is noted. The paper further serves to demonstrate not only the nature of the data available, but also the vast quantity of data involved.

•THE engineer's problem in freeway design is basically one of estimating the parameters defining the traffic demand, capacity, and level of service of the facility. However, as a chain is only as strong as its weakest link, so the overall level of service on a freeway is highly subject to the operation in critical sections, manifest by either sudden increases in traffic demand or the creation of intervehicular conflicts within the traffic stream, or a combination of both. Such regions of more restrictive vehicular operation often occur in the vicinity of entrance ramps. It is therefore imperative that these regions be considered when determining the design volume and capacity of a facility. A knowledge of the operational characteristics and traffic requirements at such locations is essential for proper planning, design and control.

To this end the Texas Transportation Institute was awarded a research contract entitled "Gap Acceptance and Traffic Interaction in the Freeway Merging Process" by the U. S. Bureau of Public Roads. This research problem forms part of the program of Traffic Systems research undertaken by the Bureau.

OBJECTIVES

Generally, the objective of this project is the development of detailed criteria on traffic stream interaction and geometric features pertaining to the freeway merging process, so as to develop methods for increasing capacity and safety through effective control and functional design. While primary emphasis is on gap acceptance characteristics, the system under study includes the freeway from a point upstream to a point downstream of the entrance ramp proper and takes due regard of other traffic characteristics, geometric features, and environmental conditions. The ultimate purpose of the research is the application of this information in traffic design and operation and in computer simulation.

Field studies for the gathering of data used in the testing and evaluation of existing and developed criteria were carried out at a large number of selected entrance ramps

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between freeway and entering vehicles. These relationships were derived by means of a one-variable probit analysis. This analysis is of a more specialized nature and is further explored in another paper. On some of these figures presented later in this report, only one of the relative speed lines is shown. This is due to either extremely low sample sizes in one of the relative speed groups causing the results to be unreliable or otherwise the probit analysis did not converge.

A number of other computer programs have been written for the more specialized analysis of the merging operation. These specialized programs will not be discussed in this report apart from listing them in Table 8, which shows the library of computer programs written, all using the simple data format obtained by the time-oriented analysis of the films.

TYPICAL RESULTS

The following is a discussion of the traffic operation at some of the study locations with emphasis on the effects of geometrics as mirrored by the contour diagrams.

No Geometric Effect

Figures 42 and 43 illustrate the operation at two ramps in California. These two ramps are quite similar in the geometric features of acceleration lane length and shape, angles of convergence, grades, curvature, number of lanes and others. Furthermore, these two ramps are located in the same geographic area and both studies were made at about the same time of day. It is therefore reasonable to expect that the difference in operation at these two locations was caused by the traffic demand.

By comparing the two volume contours, it can be seen that the demand on the freeway was about the same at both locations but that the ramp volume was higher at the Ashby Avenue ramp. This may be the cause for the concentration of higher acceleration noise in the area near the nose of the Ashby ramp. This feature was also evidenced at other high-volume ramps. Although the range in magnitude of acceleration noise is about the same at both ramps, the noise is more evenly spread out at the Broadway ramp except for a high acceleration noise spot that coincides with a sudden peak in average speed and decrease in density. Another such peak in acceleration noise is unexplained but occurred at the end of the study period, perhaps before it was reflected by the other parameters. A region of low acceleration noise also coincides with a high-density, low-speed area. None of these peaks and dips in acceleration noise are reflected in the volume contours. As can be expected, the patterns of the contours showing the probability of a gap greater than 3 seconds resemble those of the volume contours. The probability seems to be lower generally at Broadway than at Ashby, although the volume levels in the Broadway merging area are generally lower. This may be caused by the bunching of vehicles which could cause the number of gaps greater than 3 seconds to remain essentially the same as volume increases slightly, while the total number of gaps increase directly with the number of vehicles.

Another situation in which traffic demand completely overshadows the effect of geometric variables, and which the reader of the contour diagrams should be watchful for, is when forced flow exists. Such a condition is illustrated in Figure 44. From an inspection of the volume and acceleration noise contours one could easily conclude, in view of the low acceleration noise, that this ramp is of an excellent design. That may well be so, but the speed and density contours reveal that conditions of forced flow exist, overshadowing any effect that geometrics might have. Note that again the regions of higher acceleration noise correspond to the regions of lower density and higher speed. This observation bears a distinct implication regarding the use of acceleration noise as a measure of the level of service on a facility. Another such example of congested flow is demonstrated by Figure 45. It is clear that in view of the low speeds, no observations on gap acceptance at high relative speeds were made and therefore only the low relative speed lines can be exhibited on the diagrams of gap acceptance characteristics.

These effects of traffic demand on operations in the merging area, as displayed by the contour diagrams, should be kept in mind when reading the figures for the effects of geometrics.

and the rows showing the distribution of speeds for different levels of gap sizes (volume levels). Similarly, Table 6 displays speed and space headway (density) distributions of volumes against densities. It should be noted that volumes are calculated by inverting gaps and densities by inverting space headways. Note the emergence of the volume-density curve in Table 7.

Further output of this program consists of one-minute and five-minute averages of volume counts, one- and five-minute space and time mean speeds and one- and five-minute average densities calculated first as the inverse of the mean space headway, second as the mean of the inverse space headways and third as defined by Lighthill and Whitham: the total travel time over the section divided by the product of the length of the study section and the time intervals used in averaging. As stated earlier, the program can compute this information for any or all of the sections in the study area, giving rise to output far too bulky for inclusion in this report.

Other programs used to extract basic traffic parameters describing the merging operation include programs to investigate ramp vehicle speeds and accelerations, freeway gap distributions over shorter time intervals, and ramp arrival distributions. Another program investigates the averages of a number of traffic parameters measured over short time intervals at each consecutive space interval. These parameters include: volume, density, speed, energy, shock wave speeds, speed noise, acclerations, acceleration noise, expected length of queue, and probability of a gap less than the critical gap. The program further proceeds to plot contour maps of each of these variables using a digital incremental plotter, thus effectively demonstrating their variation in both time and space. For the purpose of illustrating the merging operation in this report, only five of these contour maps were selected and drawn up on a figure. One such figure was prepared for each film taken, illustrating the merging operation during that period. Figure 41 is such a figure displaying the parameters of volume, speed, density, acceleration noise and probability of a gap of greater than 3 seconds on a continuous basis in both time and space. This figure was derived from one of the films taken at the Dempster Street northbound entrance ramp on Edens Expressway, illustrated earlier in Figure 35. The figures are read in exactly the same manner as a conventional contour level map with which civil engineers are more familiar. For example, a horizontal section through the volume contours gives a profile of volumes along the length of the study area at the instant in time at which the section is taken. Similarly, a vertical section through this figure gives a profile of volume levels over the time length of the study period at a point in space on the study area. The contour figure as a whole is an excellent way of demonstrating the variation of such a parameter in both dimensions and therefore effectively illustrates the overall operation in the study area.

All these parameters were computed by averaging over the time interval indicated on the ordinates of the figures (2 minutes in this case) and over the distance intervals indicated on the abscissae. The volume is computed by simply counting the number of arrivals at each station during the averaging interval and expanding it to hourly volumes. The speed is calculated by averaging the travel times over each interval for each averaging period and then dividing the distance by this average travel time. It is therefore the time mean speed. The density is not calculated from the volume and the speed, but is computed by summing the travel times of vehicles traversing the 200-ft section during the averaging time interval and then dividing this sum by the product of the distance interval over which the travel times were computed (200 ft) and the time interval (2 minutes). The acceleration noise is computed as the standard deviation of the accelerations of consecutive vehicles over two adjacent distance intervals and is therefore based on a 100 percent sample rather than on a single vehicle as has been conventionally done. The probability of a gap of greater than 3 seconds is computed by taking the ratio of the number of gaps greater than 3 seconds to the total number of gaps. The values thus calculated were considered to be the value of the parameter at the midpoint of the time and distance interval over which it was computed, and contour lines drawn through it in the conventional manner.

The sixth diagram on the figure illustrates the gap acceptance characteristics observed at the entrance ramp during the study period. It shows the cumulative probability of a certain size gap being accepted under conditions of high and low relative speeds

TABLE 8

COMPUTER PROGRAMS FOR THE ANALYSIS OF ENTRANCE RAMP OPERATIONS

Program Number	Descriptive Title
1	Check for obvious errors
2	Plot time-space diagram
3	Analysis of speed-density relations
4	Speeds and accelerations of ramp vehicles
5	Analysis of gap availability
6	Analysis of gap stability
7	Analysis of gap acceptance characteristics
8	Probit analysis of gap acceptance characteristics
9	Effect of following gap on gap acceptance
10	Effect of time waiting on gap acceptance
11	Deviation of ramp speed distributions
12	Deviation of arrival distributions
13	Distributions of relative speeds
14	Distributions of point of entry
15	Distributions of number of gaps rejected
16	Analysis of vehicle travel times

makes use of a digital incremental plotter to plot a time-space diagram of the operation on the study section during the study period. A sample from such a time-space plot is shown on Figure 40.

The solid lines on the diagram represent vehicle paths in time and space on the freeway and the dashed lines represent vehicles on the ramp. The dashed line of a ramp vehicle changes to a solid line at the first marked station after it enters the freeway.

The time-space traces begin where vehicles enter the study section, either at the beginning of the section or when a vehicle weaves into the outside freeway lane from the second lane. Similarly, traces terminate when a vehicle leaves the study area either at the end or by weaving into the second lane from the outside lane. A dashed line that terminates without changing to a solid line indicates that a ramp vehicle entered the freeway and then changed lanes before it reached the next reference mark.

The uses of the time-space diagram are many and varied. In the first place, it was used for detecting errors in the data that could not be found by the first error routine. Such errors are usually evidenced by sudden increases or decreases in speed (sharp changes in the slope of the time-space line) or by two lines coming very close together at certain points without actually crossing. (If it did cross, it would have been found by the first error program.) These probable errors were revealed by the time-space diagram and could then be checked by referring back to the film.

A second use of the time-space diagram was in simulating missing data points. These missing data points were usually at the ends of the study section, as explained earlier. As the computer cannot recognize the difference between a missing data point and a weaving vehicle, it was essential that these points be filled in. This was easily achieved by extrapolating the time-space path, taking due regard of the preceding and following vehicle. It was felt that this technique introduced only negligible error and was to be preferred to either shortening the study section by at least 400 ft or to breaking the continuous study period up into a number of shorter intervals.

The digital plotter plots to an accuracy of 0.01 inches, that is, an accuracy of $\frac{1}{10}$ second on the time scale used. Although this accuracy is adequate for taking most measurements directly off the time-space plot, it was not used in this capacity. It did, however, further prove its use in revealing the general operating conditions on the study section at a glance and was also found to be an almost indispensable aid in the writing and debugging of further programs.

Basic Traffic Parameters

Several programs were written to extract a number of basic traffic parameters from the data. One of these programs computes the speed-volume-density relationships of the traffic stream in three different ways, based on individual vehicles, based on one-minute averages, and based on five-minute averages. These relationships can be computed over any or all 100-ft or 200-ft intervals in the study section. Some typical examples of the computer output on the microscopic relationships, that is, based on individual vehicles, are shown in Tables 5, 6, and 7 for the 200-ft section located between 400 ft and 200 ft upstream of the nose. These tables not only display the basic traffic parameters measured during the study period, but also present frequency tables of each parameter for different levels of any of the other parameters in a fashion appropriate for the further analysis of the stochastic properties of the interrelationships between speed, volume and density. For example, Table 5 is a two-way frequency table, the columns showing the distribution of gaps (volumes) for different speed levels

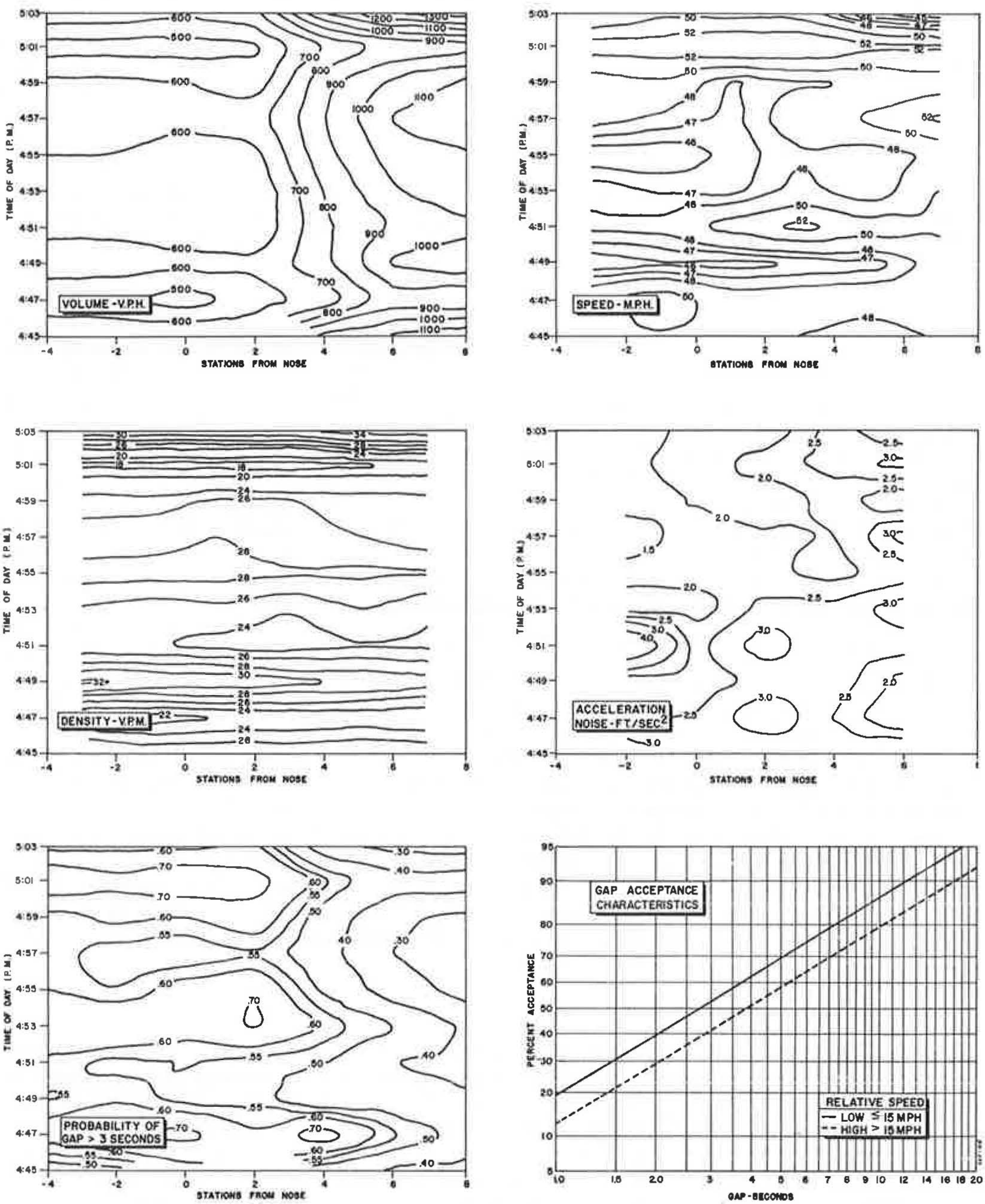


Figure 41. Traffic characteristics of Dempster Street northbound entrance ramp to Edens Expressway, Chicago.

TABLE 5

FREQUENCY TABLE OF GAPS AND SPEEDS, STATION -4 TO -2, MONUMENT JUNCTION SOUTHBOUND ENTRANCE RAMP ON I680, PLEASANT HILL, SAN FRANCISCO

Gaps	Speeds										
	0-25	25-35	35-40	40-45	45-50	50-55	55-60	60-65	65-75	75-99	Totals
0-1	0	2	1	8	3	2	4	1	0	0	21
1-2	0	0	13	18	41	17	6	4	2	0	101
2-3	0	5	7	20	25	14	5	2	1	1	80
3-4	0	0	5	8	23	10	4	3	2	3	58
4-5	0	1	3	5	8	5	6	5	1	0	34
5-6	0	0	2	4	12	6	10	5	3	0	42
6-7	0	0	1	5	10	3	4	4	0	2	29
7-8	0	0	0	4	13	5	4	2	0	0	28
8-9	0	0	1	2	7	1	5	1	0	1	18
9-10	0	0	1	0	5	1	1	3	1	0	12
10-11	0	0	1	3	0	2	2	1	1	0	10
11-12	0	0	0	1	4	1	2	1	0	0	9
12-13	0	0	0	0	2	1	0	4	1	0	8
13-14	0	0	1	0	1	1	1	2	0	0	6
14-15	0	0	0	1	1	1	1	0	0	0	4
15-99	0	0	0	1	7	0	1	3	0	0	12
Totals	0	8	36	80	162	70	56	41	12	7	472

TABLE 6

FREQUENCY TABLE OF SPACE HEADWAYS AND SPEEDS, STATION -4 TO -2, MONUMENT JUNCTION SOUTHBOUND ENTRANCE RAMP ON I680, PLEASANT HILL, SAN FRANCISCO

Space Headway	Speeds										
	0-25	25-35	35-40	40-45	45-50	50-55	55-60	60-65	65-75	75-99	Totals
0-40	0	0	0	0	0	0	0	0	0	0	0
40-60	0	2	2	6	2	0	0	0	0	0	12
60-80	0	0	3	4	7	2	2	0	0	0	18
80-100	0	0	6	12	7	3	3	4	0	0	35
100-120	0	2	3	4	12	1	2	0	0	0	24
120-140	0	2	3	13	13	3	0	1	1	0	36
140-160	0	1	3	3	10	10	3	0	1	0	31
160-180	0	0	2	4	9	2	0	0	0	0	17
180-200	0	0	2	3	5	7	1	1	0	0	19
200-240	0	1	4	3	16	7	4	0	1	0	36
240-280	0	0	1	4	10	6	0	1	0	0	22
280-320	0	0	2	4	5	2	3	1	0	1	18
320-360	0	0	1	2	6	2	2	2	1	0	16
360-400	0	0	0	3	6	3	4	1	1	0	18
400-440	0	0	0	4	6	4	5	4	0	2	25
440-500	0	0	1	3	11	3	5	2	1	1	27
500-560	0	0	1	2	13	3	3	4	3	0	29
560-620	0	0	1	1	2	4	3	3	0	0	14
620-999	0	0	1	5	22	8	16	17	3	3	75
Totals	0	8	36	80	162	70	56	41	12	7	472

TABLE 7

FREQUENCY TABLE ON VOLUMES AND DENSITIES, STATION -4 TO -2, MONUMENT JUNCTION SOUTHBOUND ENTRANCE RAMP ON I680, PLEASANT HILL, SAN FRANCISCO

Density	Volume											
	0-10	10-20	20-25	25-30	30-35	35-40	40-45	45-50	50-55	55-60	60-99	Totals
0-10	101	4	0	0	0	0	0	0	0	0	0	105
10-20	35	88	1	0	0	0	0	0	0	0	0	124
20-30	0	41	26	5	0	0	0	0	0	0	0	72
30-40	0	1	15	23	17	3	2	0	0	0	0	61
40-50	0	0	1	9	13	12	5	1	0	0	0	41
50-60	0	0	0	0	6	8	6	1	3	4	1	29
60-70	0	0	0	0	0	0	6	0	4	3	2	15
70-80	0	0	0	0	0	0	0	0	4	2	4	10
80-90	0	0	0	0	0	0	0	0	0	0	3	3
90-100	0	0	0	0	0	0	0	0	0	1	6	7
100-110	0	0	0	0	0	0	0	0	0	0	0	0
110-120	0	0	0	0	0	0	0	0	0	0	3	3
120-130	0	0	0	0	0	0	0	0	0	0	2	2
130-140	0	0	0	0	0	0	0	0	0	0	0	0
140-999	0	0	0	0	0	0	0	0	0	0	0	0
Totals	136	134	43	37	36	23	19	2	11	10	21	472

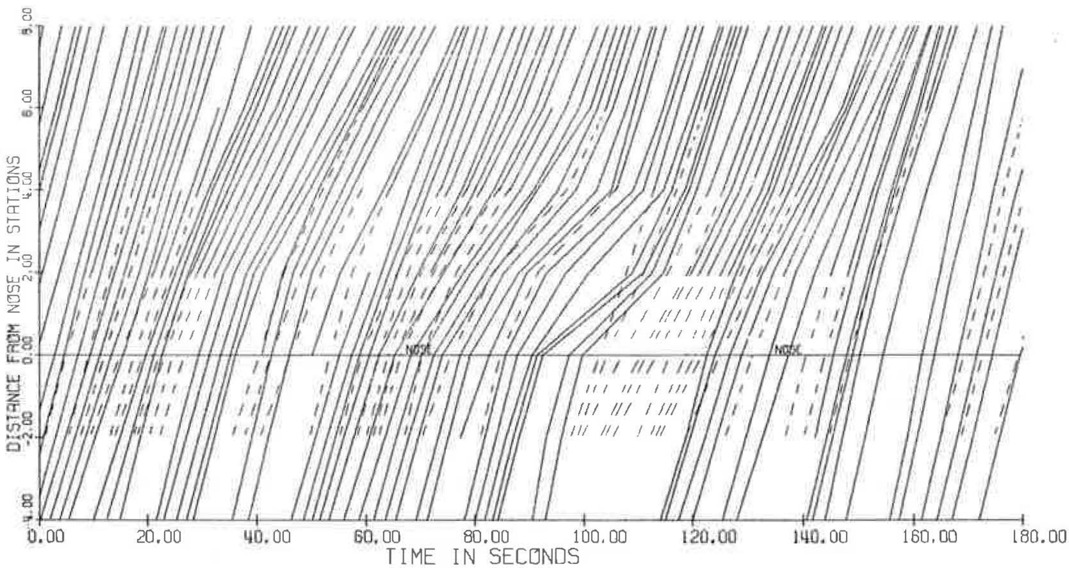


Figure 40. Time-space diagram: Broadway northbound entrance ramp, Bayshore Freeway, San Francisco. Dashed lines represent vehicles on ramp, solid lines represent vehicles on freeway.

on the rate of filming or camera speed. The vehicles were also classified into trucks and passenger cars. At the peak of the data reduction, for a period of about two months, nine analysis crews were working three 8-hr shifts per day. These data were then punched into computer cards for further processing by an electronic computer. This produced a library of traffic data on entrance ramp operation of such magnitude and scope as has been heretofore unavailable.

DATA ANALYSIS

The vast amount of data collected on this project could only be processed by electronic computer and to this end, use was made of the facilities of the Texas A and M University Data Processing Center, of which the principal computer is an IBM 7094. In order to extract the most possible information with the highest possible sample sizes from the data, extreme care had to be taken with data editing routines for removing all possible errors made during the film analysis and punching of the data cards.

Error Routines

One such error-detecting program simply scans the data for such obvious errors as negative speeds, vehicle paths crossing or two vehicles at the same point at the same time. The latter seems to have been the most frequent mistake, caused by improper identification of vehicles from one picture to the next. This could easily happen under conditions of high density on the study section where, at the same time, the film image definition was less than ideal. The first editing program was written to detect these errors which could then easily be rectified by going back to the original film.

Due to the circling technique of filming, certain problems were experienced in the data analysis that must be immediately evident to a practical person—the problem of missing data points. This could occur in several ways. For example, the reference mark at either end of the study section could be momentarily out of the viewfinder of the camera because of a sudden movement of the airplane. Sometimes even the whole study section could momentarily disappear from view. Normally this would cause a drastic reduction in the sample size of continuous study and in the length of the study section. These problems were overcome by the second editing program. This program

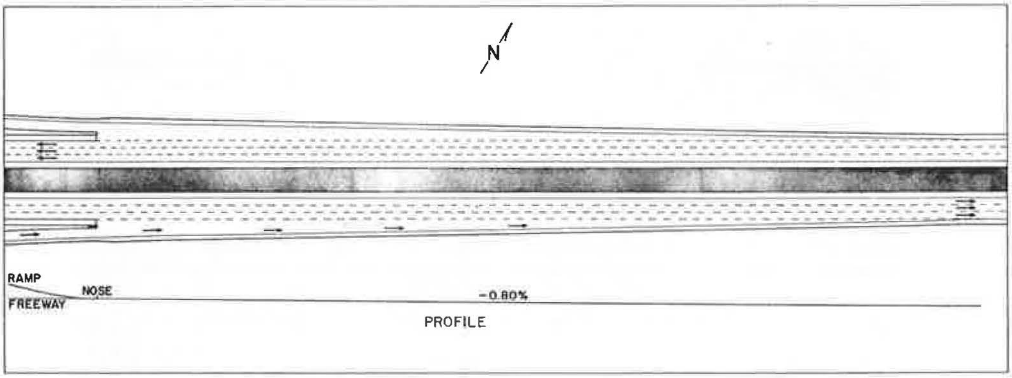
Figure 39. Film analysis in progress.

filmed on these trips are listed in Table 4. Drawings of most of these ramps, with tables listing the major geometric characteristics, appear as Figures 10, 12, and 26 through 38.

DATA REDUCTION

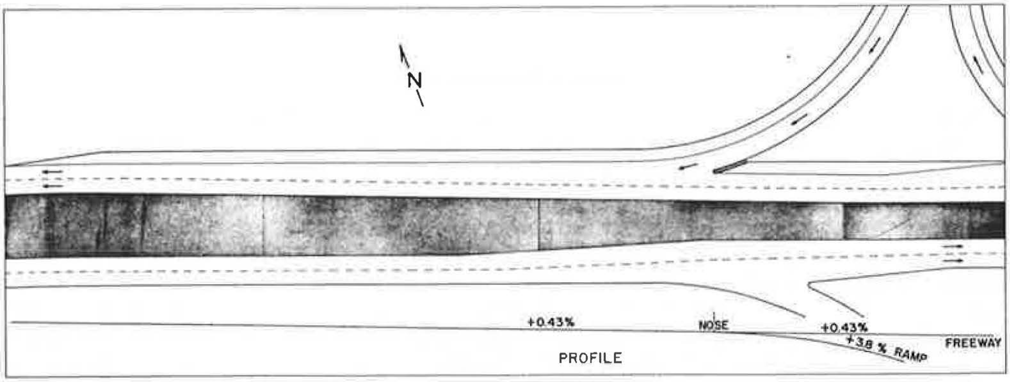
Before the data collection could be started, serious consideration had to be given the film analysis techniques and equipment to be used. The analyst of time-lapse photographs has the choice as to time orientation or space orientation, i. e., either measuring the time required for a certain fixed displacement or measuring the displacement over a certain fixed time interval. Either method has certain advantages and disadvantages over the other. Time-oriented analysis requires a higher rate of filming than space-oriented analysis for the same accuracy, but is usually considerably faster to perform. Specialized projectors with automatic X-Y coordinate readout equipment are available for space-oriented data reduction and these were considered for use on this project. However, because of the continuously changing space scale inherent in the technique of data collection used, space-oriented analysis was rejected in favor of time orientation. To this end, marks were made along the freeway and the ramp at fixed intervals both upstream and downstream of the ramp nose as described earlier. Time orientation, as used on this project, has the further advantage of eliminating the need for maintaining a strict altitude or flight pattern during filming. The data reduction was performed on three 35-mm projectors, two of which were Richardson Model 300 projectors (Fig. 39) and the third was a Vanguard projector, consisting of a Model M-35CD projection head and a Model M-13 projection case.

Data reduction consisted of following each vehicle on the outside freeway lane and each vehicle on the ramp through the study area, recording the frame number at which the vehicle crossed each reference mark. In this manner data were collected on the time-space trajectories of approximately 60,000 vehicles. Further information taken off the films included the frame number at which each ramp vehicle entered the freeway and frame numbers at fixed time intervals so as to determine and keep a running check



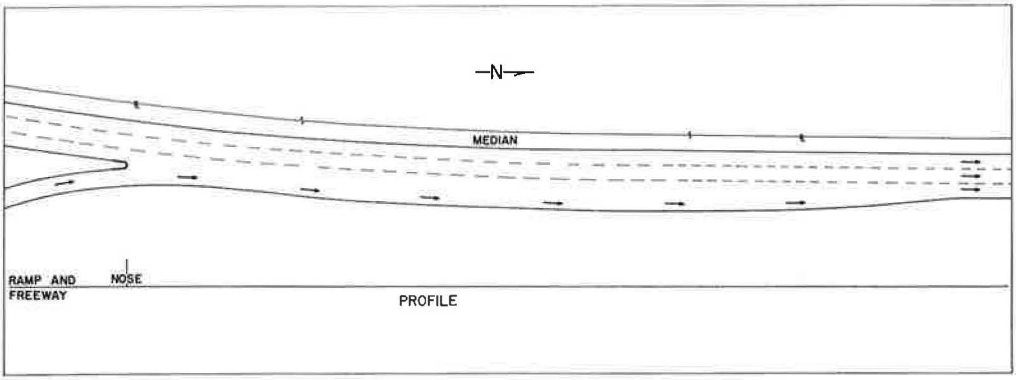
GEOMETRIC CHARACTERISTICS	DATA
LENGTH OF ACCELERATION LANE	1500 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	2°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	1½°
RAMP GRADE AT NOSE	-0.80 %
FREEWAY GRADE AT NOSE	-0.80 %
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	10 FEET
LENGTH OF RAMP	900 FEET

Figure 37. Geometric characteristics of Pulaski Road eastbound entrance ramp, Southwest Expressway, Chicago.



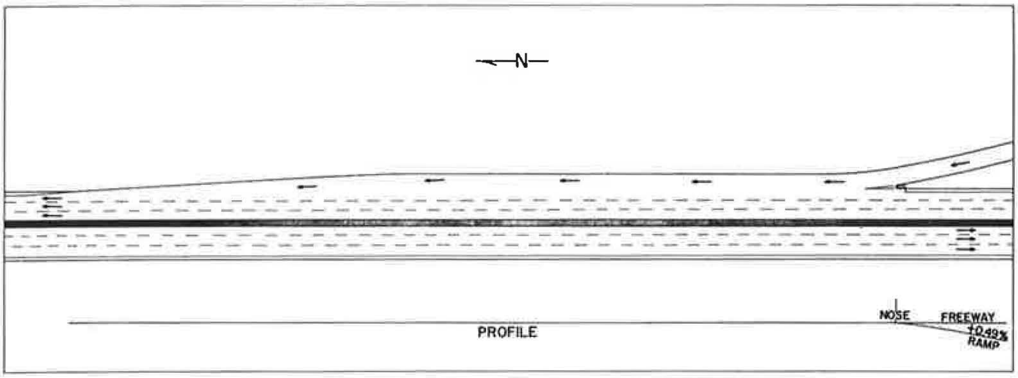
GEOMETRIC CHARACTERISTICS	DATA
LENGTH OF ACCELERATION LANE	593 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	14½°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	14½°
RAMP GRADE AT NOSE	+0.43%
FREEWAY GRADE AT NOSE	+0.43%
NUMBER OF FREEWAY LANES	2
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	SLIGHT RIGHT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	600 FEET

Figure 38. Geometric characteristics of Brentwood Avenue westbound entrance ramp, Daniel Boone Expressway, St. Louis.



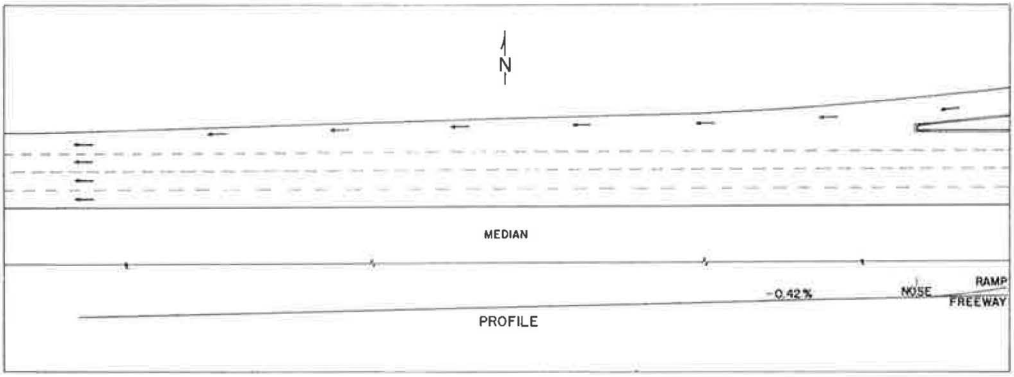
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	785 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	11°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	11°
RAMP GRADE AT NOSE	LEVEL
FREEWAY GRADE AT NOSE	LEVEL
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	SLIGHT RIGHT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	925 FEET

Figure 35. Geometric characteristics of Dempster Avenue northbound entrance ramp, Edens Expressway, Chicago.



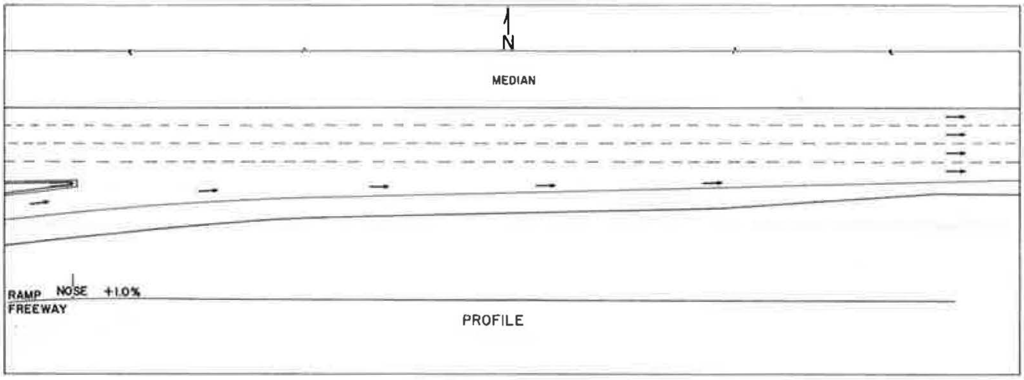
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	468 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	11°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	11°
RAMP GRADE AT NOSE	+0.49%
FREEWAY GRADE AT NOSE	LEVEL
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	SLIGHT RIGHT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	1020 FEET

Figure 36. Geometric characteristics of Peterson Avenue northbound entrance ramp, Edens Expressway, Chicago.



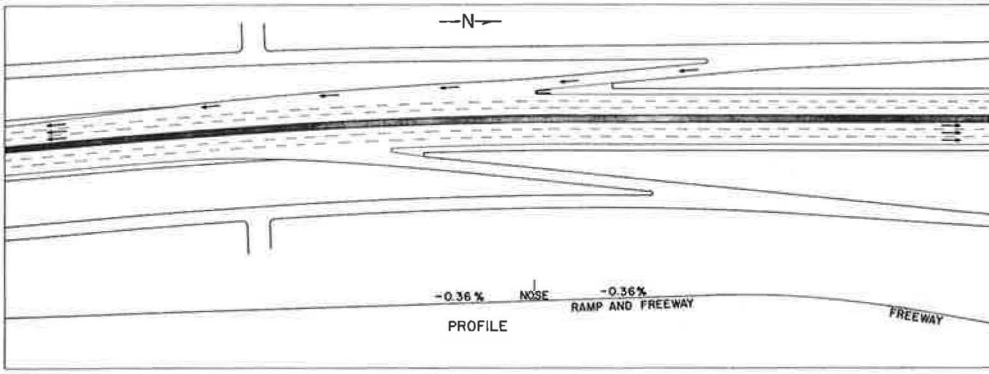
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	738 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	6°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	6°
RAMP GRADE AT NOSE	-0.42%
FREEWAY GRADE AT NOSE	-0.42%
NUMBER OF FREEWAY LANES	4
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	432 FEET

Figure 33. Geometric characteristics of Independence Avenue westbound entrance ramp, Eisenhower Expressway, Chicago.



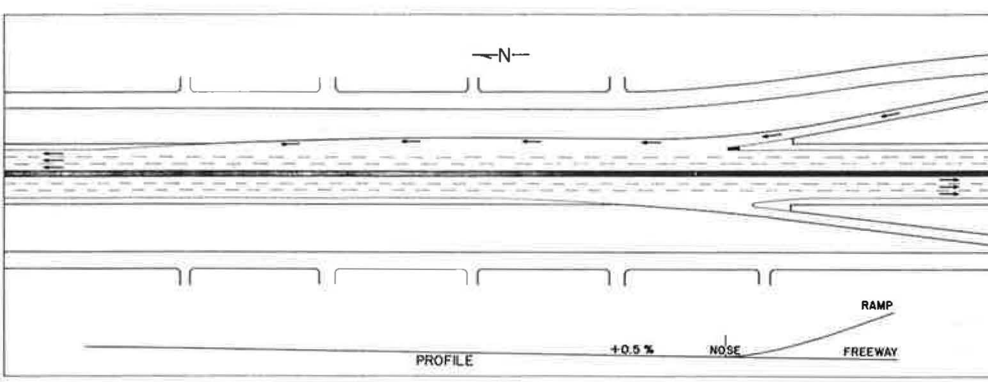
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	712 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	6°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	6°
RAMP GRADE AT NOSE	+1.0%
FREEWAY GRADE AT NOSE	+1.0%
NUMBER OF FREEWAY LANES	4
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	SLIGHT RIGHT
RAMP CURVATURE	SLIGHT RIGHT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	350 FEET

Figure 34. Geometric characteristics of Kostner Avenue eastbound entrance ramp, Eisenhower Expressway, Chicago.



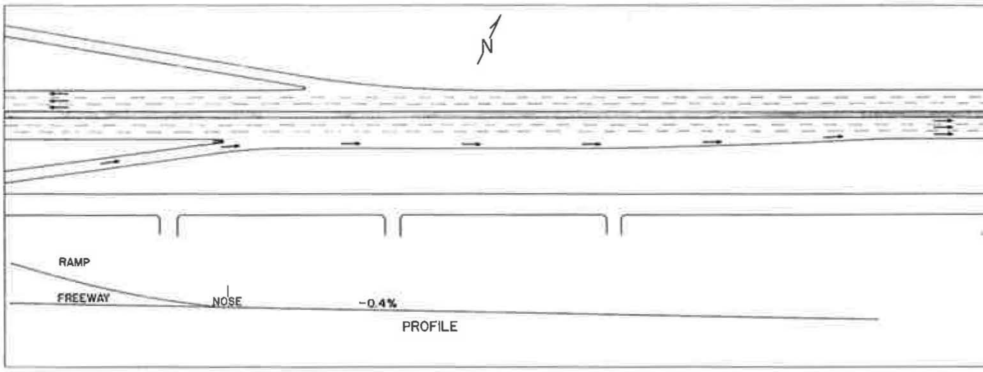
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	660 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	3° 30'
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	3° 30'
RAMP GRADE	-0.36%
FREEWAY GRADE	-0.36%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	50 FEET

Figure 31. Geometric characteristics of Warren eastbound entrance ramp, Southfield Expressway, Detroit.



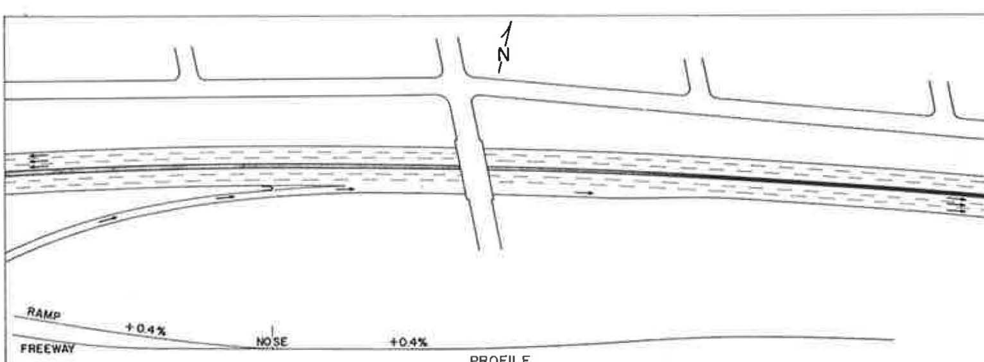
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	515 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	4° 45'
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	4° 45'
RAMP GRADE AT NOSE	+0.5 %
FREEWAY GRADE AT NOSE	+0.5%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	550 FEET

Figure 32. Geometric characteristics of Linwood northbound entrance ramp, John Lodge Expressway, Detroit.



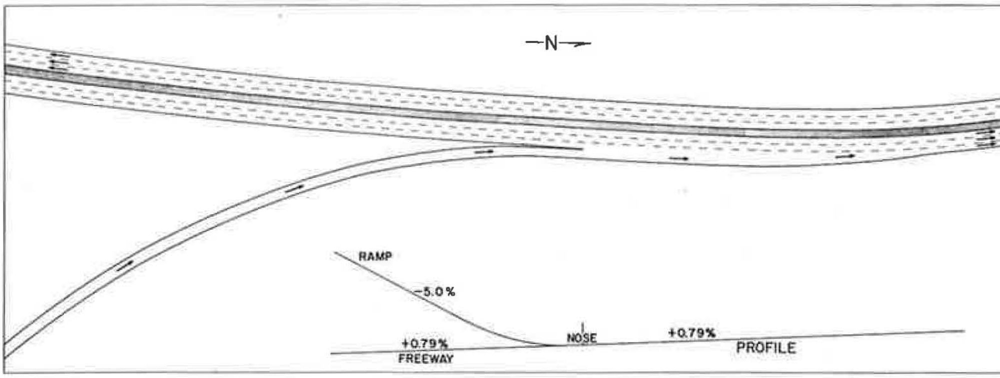
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	693 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	6° 15'
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	6° 15'
RAMP GRADE	-0.3%
FREEWAY GRADE	-0.4%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	450 FEET

Figure 29. Geometric characteristics of Chene eastbound entrance ramp, Edsel Ford Expressway, Detroit.



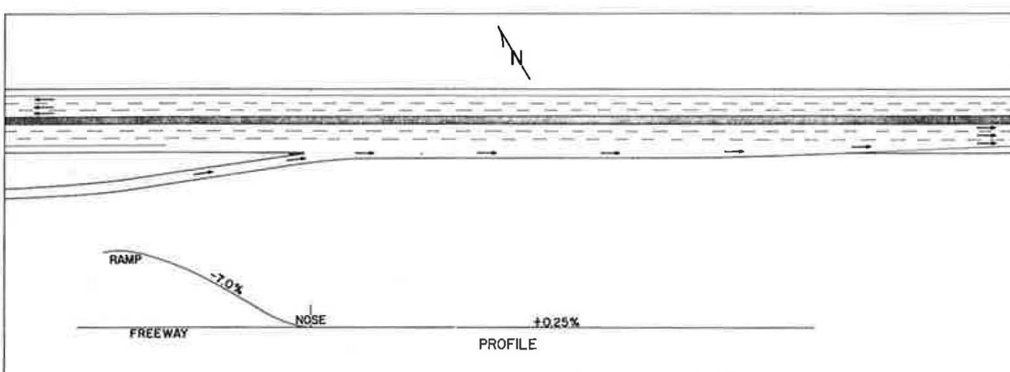
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	500 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	7°15'
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	7°15'
RAMP GRADE AT NOSE	+0.4 %
FREEWAY GRADE AT NOSE	+0.4 %
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	600 FEET

Figure 30. Geometric characteristics of Gratiot eastbound entrance ramp, Edsel Ford Expressway, Detriot.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	1200 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	1°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	1°
RAMP GRADE AT NOSE	+0.79 %
FREEWAY GRADE AT NOSE	+0.79 %
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	SLIGHT LEFT
RAMP CURVATURE	SLIGHT LEFT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	1200 FEET

Figure 27. Geometric characteristics of Long Island Expressway westbound entrance ramp to Cross Island Expressway, New York.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	500 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	5°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	5°
RAMP GRADE AT NOSE	+0.25%
FREEWAY GRADE AT NOSE	+0.25%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	12 FEET
LENGTH OF RAMP	520 FEET

Figure 28. Geometric characteristics of 69th Road southbound entrance ramp, Grand Central Expressway, New York.

TABLE 4

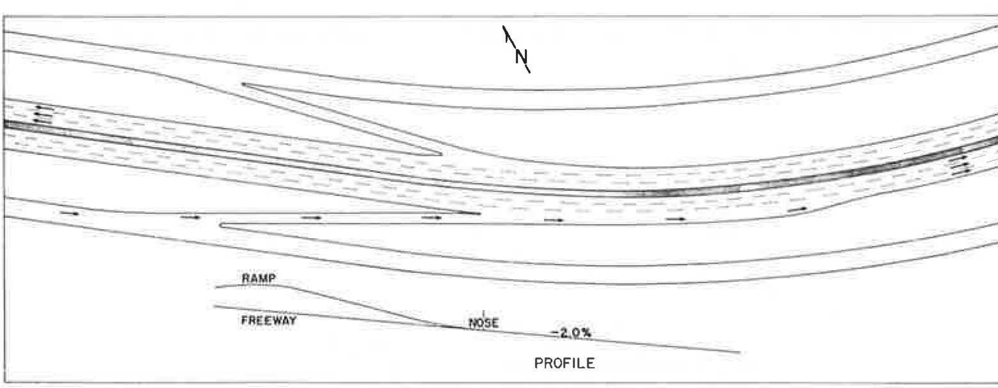
STUDY LOCATIONS IN NEW YORK, DETROIT, CHICAGO AND ST. LOUIS

Location of Ramp	Number of Films
New York Area	
Jericho, eastbound on Long Island Expressway	2
Community Drive, eastbound on Long Island Expressway	3
Long Island Expressway, northbound on Cross Island Expressway	3
Broadway, eastbound on Northern State Parkway	2
Brush Hollow Road, eastbound on Northern State Parkway	2
69th Road, southbound on Grand Central Parkway	2
Rockaway, northbound on Van Wyck Expressway	2
Detroit	
Chene, eastbound on Edsel Ford Expressway	2
Gratiot, eastbound on Edsel Ford Expressway	2
Warren Avenue, southbound on Southfield Expressway	2
Linwood, northbound on John Lodge Expressway	2
Chicago	
Independence Avenue, westbound on Eisenhower Expressway	2
Kostner, westbound on Eisenhower Expressway	2
Dempster Street, northbound on Edens Expressway	3
Peterson, northbound on Edens Expressway	3
Pulaski, eastbound on Southwest Expressway	
St. Louis	
Brentwood, westbound to Daniel Boone Expressway	3

Circling at a quarter of a mile radius provided a better view of the study area, facilitating the analysis, and yet was wide enough that the pilot and plane could maintain the circle for long periods of time.

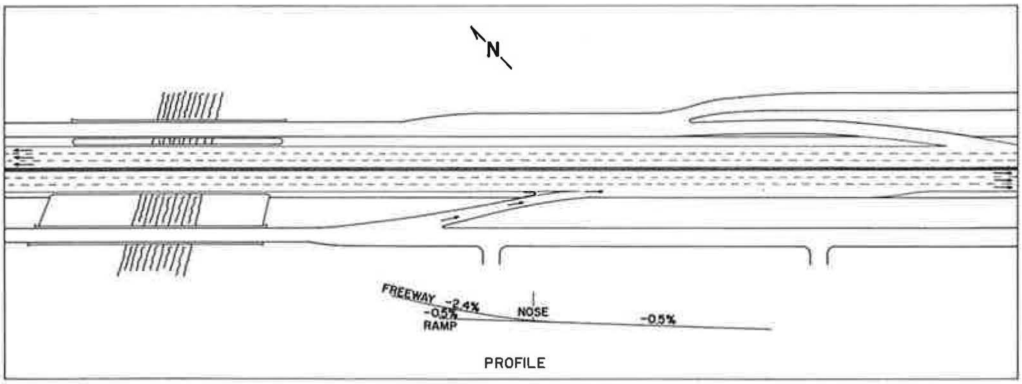
During this time, studies were carried out in Houston and the techniques and procedures perfected. The Houston studies were carried out at the eight ramps listed in Table 3. These ramps are illustrated in Figure 13 and Figures 20 through 25.

In May of 1966, the second out-of-state study was made in New York, Detroit and Chicago. This trip was a great success except for being marred by unfavorable weather, which made necessary a third trip, this time to Chicago and St. Louis. The ramps



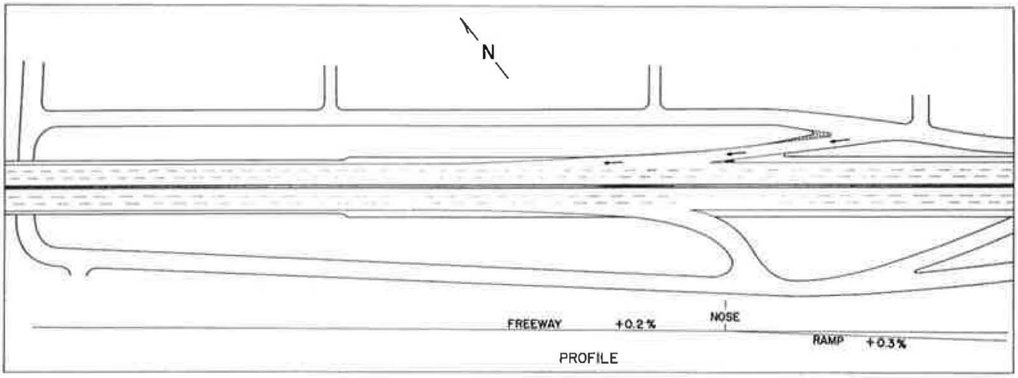
GEOMETRIC CHARACTERISTICS	DATA
LENGTH OF ACCELERATION LANE	700 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	3°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	3°
RAMP GRADE AT NOSE	-2.0%
FREEWAY GRADE AT NOSE	-2.0%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	SLIGHT LEFT
RAMP CURVATURE	SLIGHT LEFT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	660 FEET

Figure 26. Geometric characteristics of Community Drive eastbound entrance ramp, Long Island Expressway, New York.



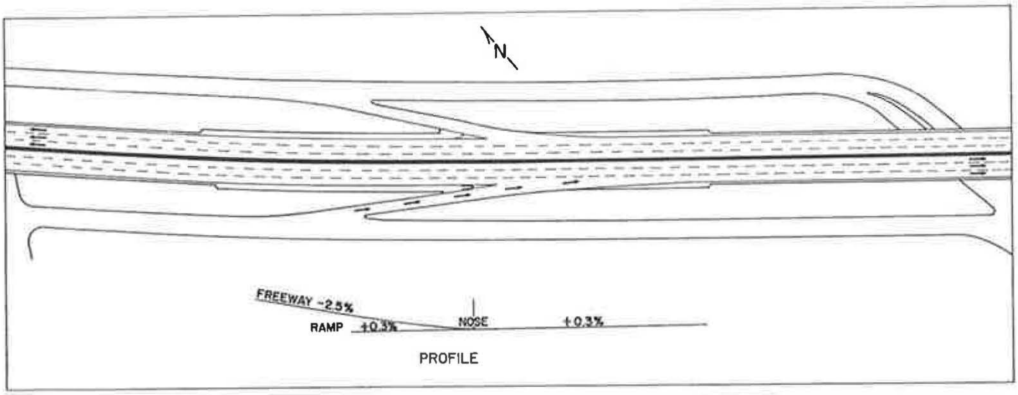
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	680 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	10°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	10°
RAMP GRADE AT NOSE	-0.5%
FREEWAY GRADE AT NOSE	-0.5%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	380 FEET

Figure 24. Geometric characteristics of Wayside eastbound entrance ramp, Gulf Freeway, Houston.



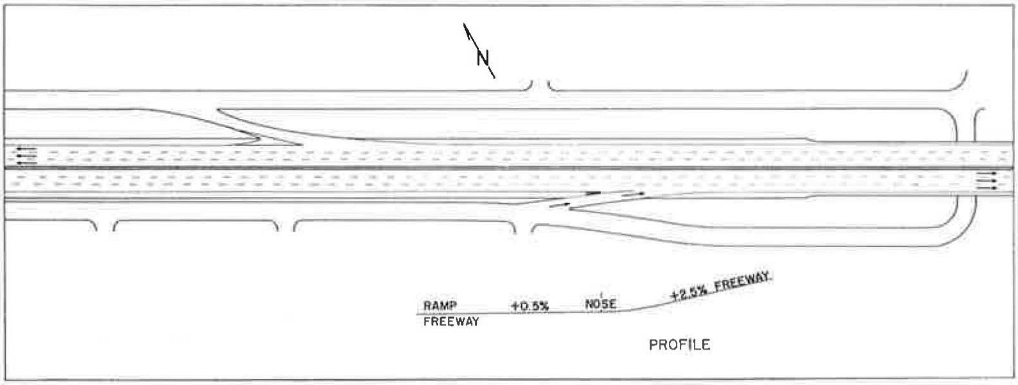
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	310 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	10°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	10°
RAMP GRADE	+0.3%
FREEWAY GRADE	+0.2%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	150 FEET

Figure 25. Geometric characteristics of Mossrose northbound entrance ramp, Gulf Freeway, Houston.



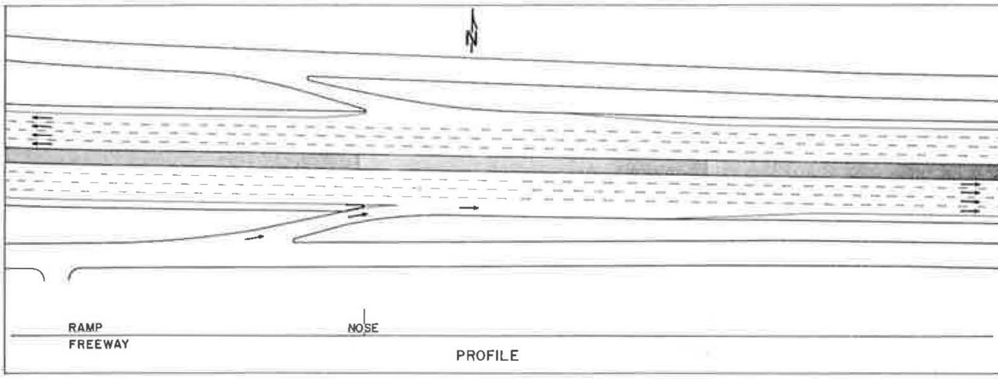
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	340 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	10°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	10°
RAMP GRADE AT NOSE	+0.3%
FREEWAY GRADE AT NOSE	-0.3%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	180FEET

Figure 22. Geometric characteristics of Cullen eastbound entrance ramp, Gulf Freeway, Houston.



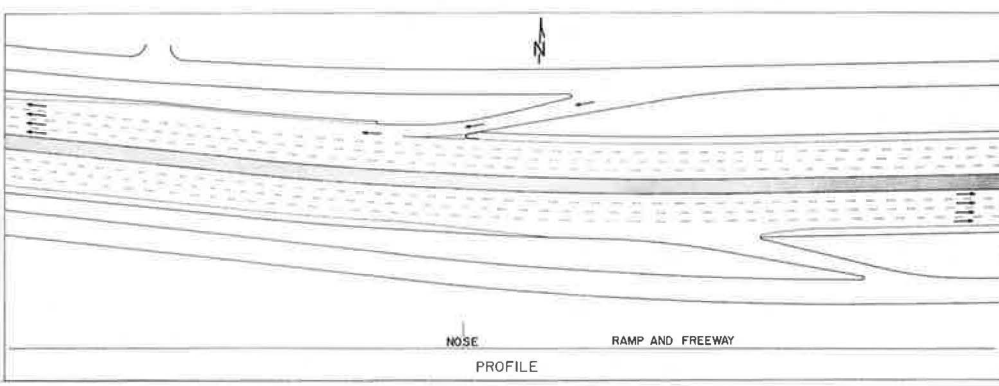
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	240 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	7°
ANGLE OF CONVERGENCE 2FT. OFF PAVEMENT	7°
RAMP GRADE AT NOSE	+0.5%
FREEWAY GRADE AT NOSE	+0.5%
NUMBER OF FREEWAY LANE	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	170 FEET

Figure 23. Geometric characteristics of Dumble eastbound entrance ramp, Gulf Freeway, Houston.



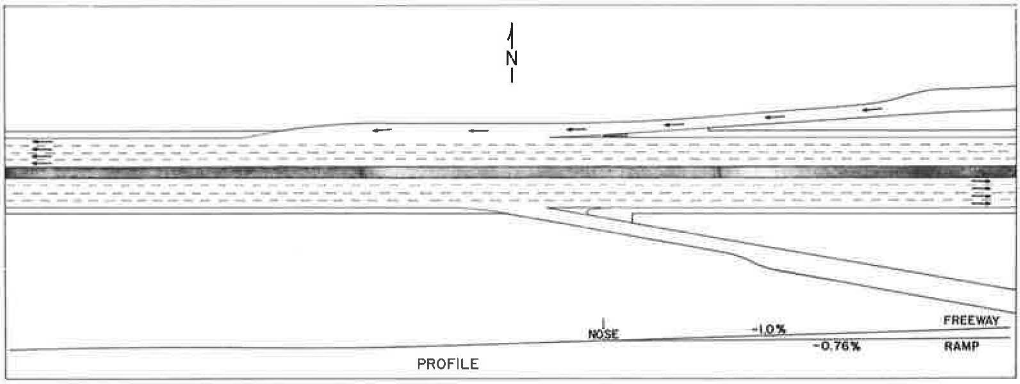
GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	610 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	12° 15'
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	12° 15'
RAMP GRADE AT NOSE	LEVEL
FREEWAY GRADE AT NOSE	-0.40%
NUMBER OF FREEWAY LANES	4
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	103 FEET

Figure 20. Geometric characteristics of Weslayan eastbound entrance ramp, Southwest Freeway, Houston.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	620 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	12°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	12°
RAMP GRADE AT NOSE	LEVEL
FREEWAY GRADE AT NOSE	LEVEL
NUMBER OF FREEWAY LANES	4
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	1° RIGHT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	160 FEET

Figure 21. Geometric characteristics of Buffalo Speedway westbound entrance ramp, Southwest Freeway, Houston.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	970 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	6°
ANGLE OF CONVERGENCE 2FT. OFF PAVEMENT EDGE	6°
RAMP GRADE AT NOSE	-0.8%
FREEWAY GRADE AT NOSE	-1.0%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	600 FEET

Figure 19. Geometric characteristics of Coldwater Canyon Avenue westbound entrance ramp, Ventura Freeway, Los Angeles.

entry (Fig. 12) and a ramp with a short acceleration lane and a large angle of entry (Fig. 13).

After extensive correspondence with highway officials in many parts of the country and reviewing a multitude of plans and profiles, ramps were selected for study in Los Angeles, Sacramento, the San Francisco area, the New York metropolitan area, Detroit, Chicago, St. Louis, and Houston.

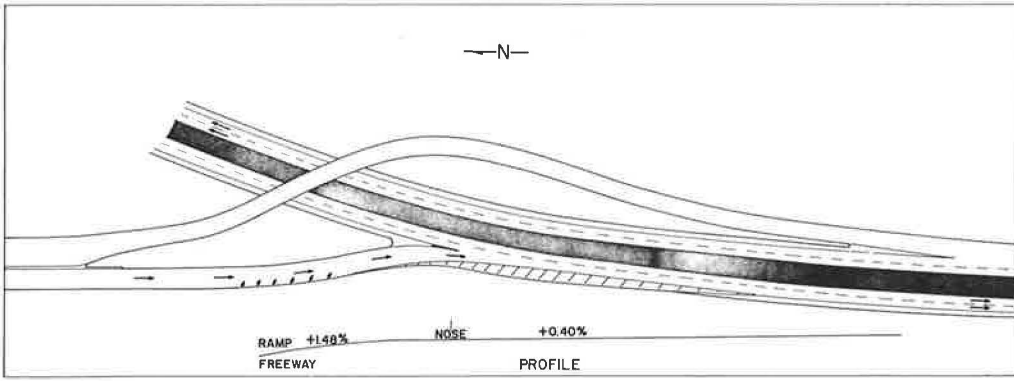
Experimental studies were first carried out in Houston, mainly for the purpose of checking out the equipment and techniques and developing the various procedures to be followed into a smoothly running organization. The first out-of-state study was done in California in November of 1965, where the ramps listed in Table 2 were filmed. The table also shows the number of films taken at each location. The study locations are listed according to the metropolitan area in which they are located, rather than by the exact suburb. Drawings of these study locations, with a table giving the major geometric characteristics, are shown in Figure 11 and Figures 14 through 19.

After the California study, it became clear that the study techniques and equipment had to be reevaluated. Some of the films taken at the locations listed and all of the films taken at some other locations were not usable. This reevaluation led to several improvements. In the first place, the camera, which until this time had a fixed shutter speed, was modified to have adjustable speeds as described earlier. Second, the camera mounting was slightly altered so as to further dampen the effect of vibrations. Third, the use of different kinds of film and different filters was investigated. Finally, the flight pattern was altered. Until this time circles were made at a radius of approximately half a mile. It was found desirable, however, that the circle be pulled tighter so that the films could be taken from more directly above the traffic.

TABLE 3

STUDY LOCATIONS IN HOUSTON

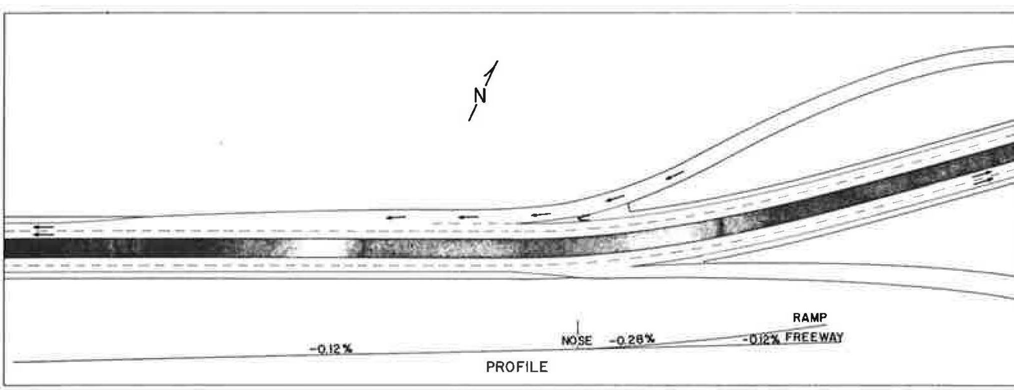
Location of Ramp	Number of Films
Weslayan, eastbound on Southwest Freeway	4
Buffalo Speedway, westbound on Southwest Freeway	2
Cullen, eastbound on Gulf Freeway	3
Dumble, eastbound on Gulf Freeway	5
Telephone, eastbound on Gulf Freeway	3
Wayside, eastbound on Gulf Freeway	4
Broad Street, eastbound on Gulf Freeway	4
Mossrose, westbound on Gulf Freeway	2



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	600 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	4°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	4°
RAMP GRADE AT NOSE	+0.4%
FREEWAY GRADE AT NOSE	+0.4%
NUMBER OF FREEWAY LANES	2
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	SLIGHT LEFT
RAMP CURVATURE	SLIGHT RIGHT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	800 FEET

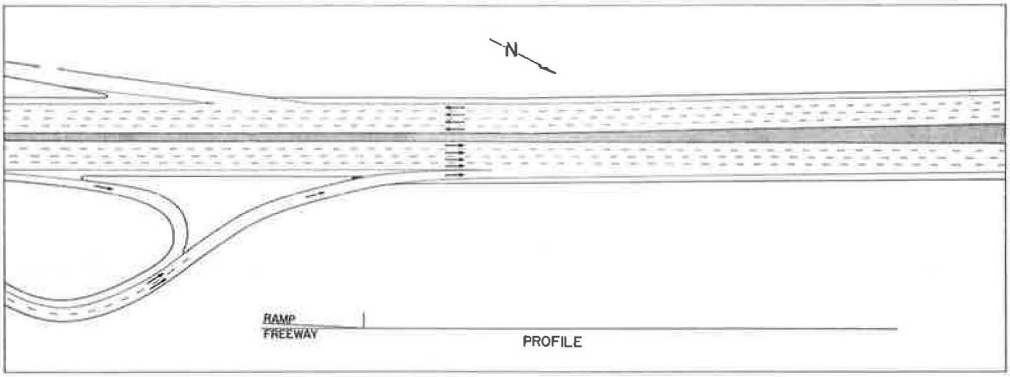
Figure 17. Geometric characteristics of Monument Junction southbound entrance ramp, I-680 in Pleasant Hill (San Francisco).

The coverage of these two geometric elements is perhaps better illustrated by Table 1 which shows the number of ramps studied that fall into each group of acceleration lane lengths and angles of entry. The range of ramp shapes studied is illustrated in Figures 10 through 13, showing the geometric characteristics of a ramp with a long acceleration lane and a small angle of entry (Fig. 10), a ramp with a long acceleration lane and a large angle of entry (Fig. 11), a ramp with a short acceleration lane and a small angle of



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	690 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	4°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	4°
RAMP GRADE AT NOSE	-0.3%
FREEWAY GRADE AT NOSE	-0.12%
NUMBER OF FREEWAY LANES	2
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	SLIGHT RIGHT
RAMP CURVATURE	SLIGHT RIGHT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	500 FEET

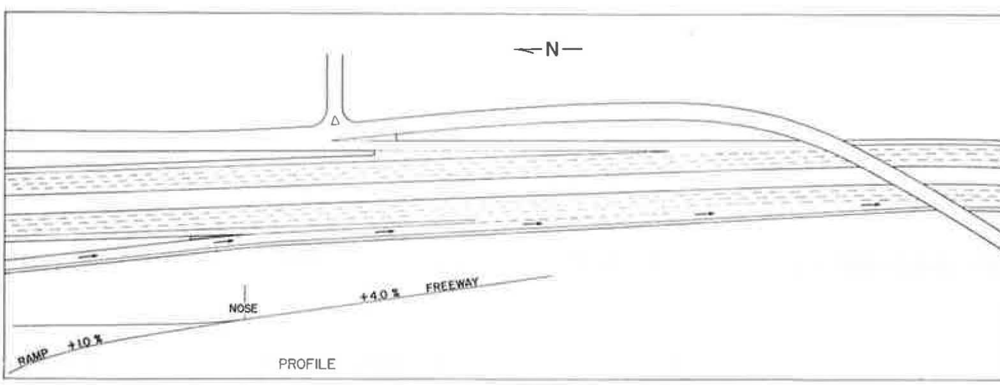
Figure 18. Geometric characteristics of Watt Avenue southbound entrance ramp, I-80, Sacramento.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	1025 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	7 ½ DEGREES
ANGLE OF CONVERGENCE 2FT OFF PAVEMENT EDGE	2 DEGREES
RAMP GRADE AT NOSE	SLIGHT DOWNGRADE
FREEWAY GRADE AT NOSE	LEVEL
NUMBER OF FREEWAY LANES	4
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	SLIGHT RIGHT
CURB OFFSET AT NOSE	10 FEET
LENGTH OF RAMP	300 FEET +

Figure 15. Geometric characteristics of Bayshore Freeway, Broadway entrance ramp, San Francisco.

Geometric considerations in the selection of study sites required the coverage of a wide range of geometrics in the ramps to be studied. Variations in acceleration lane length, angle of convergence and ramp grade were sought, primarily at locations where the freeway alignment was straight and fairly level. The considerable range of geometrics covered among the ramps selected can be seen in Figure 9, which illustrates the variation in acceleration lane lengths and angles of convergence of the ramps studied.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	700 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	5°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	2°
RAMP GRADE	+1.0%
FREEWAY GRADE	+4.0%
NUMBER OF FREEWAY LANES	4
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	10 FEET
LENGTH OF RAMP	450 FEET

Figure 16. Geometric characteristics of Lakeshore southbound entrance ramp, MacArthur Freeway, Los Angeles.

TABLE 2

STUDY LOCATIONS IN CALIFORNIA

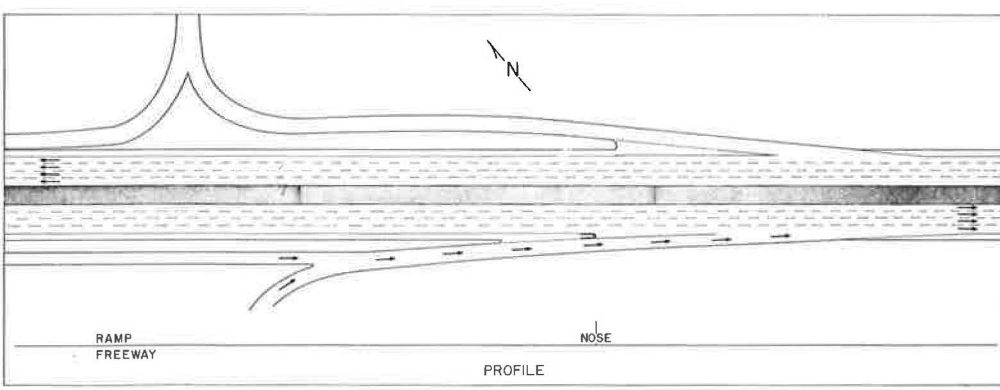
Location of Ramp	Number of Films
San Francisco Area	
Milbrae, southbound on Bayshore Freeway	1
Broadway, southbound on Bayshore Freeway	2
Lakeshore, southbound on MacArthur Freeway	1
Ashby, southbound on Eastshore Freeway	2
Monument Junction, southbound on I-680	2
Sacramento	
Watt Avenue, southbound on I-80	2
Los Angeles	
Coldwater Canyon, westbound on Ventura Freeway	2

The marking tape is first placed in position (Fig. 7) and is then cemented by rolling it down with a special roller applicator (Fig. 8). Upon completion of the study, the tape can easily be removed by simply peeling it from the pavement. It was found that for improved visibility from high altitudes, the most important dimension of the reference mark was its length rather than its width or shape.

Normally, reference marks were placed at 200-ft intervals radiating from the concrete nose of the ramp. However, on short ramps of 400 ft or less, marks were placed at 100-ft intervals.

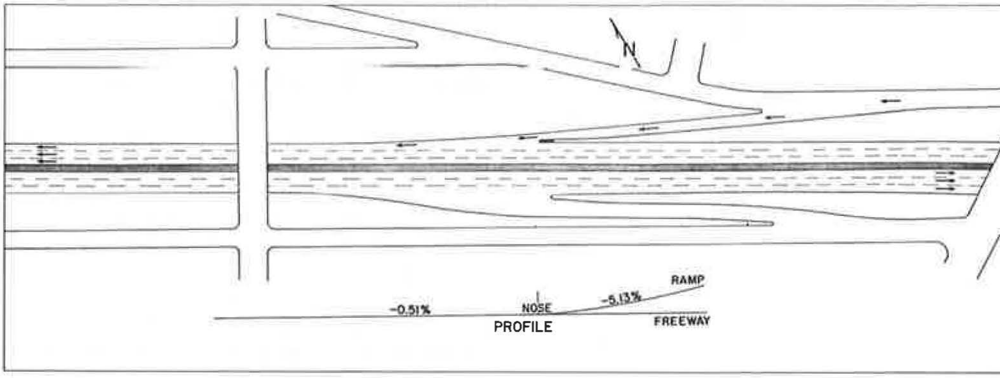
STUDY LOCATIONS

The study locations were selected to meet several operational and geometric requirements. Operational considerations required the selection of study sites operating at or near capacity while unaffected by upstream or downstream conditions, so that a pure merging situation existed. Local authorities were extremely helpful in suggesting study sites and providing volume data to aid in the final selection of sites. Before any filming was done at a location, the operation there was field checked to determine the approximate volumes and to evaluate the effect of any downstream restrictions. Based on the traffic operations during the time of the field observation, the study period was selected so that a free flow merging operation prevailed and so that the merging volume was near capacity. In spite of this careful checking of the study sites, the operation during the film studies was not always the same as during the field checks due to changes in traffic patterns, accidents, or some other temporary situation, with the result that the operational requirements were not always successfully met.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	675 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	3°
ANGLE OF CONVERGENCE 2FT OFF PAVEMENT EDGE	2°
RAMP GRADE AT NOSE	LEVEL
FREEWAY GRADE AT NOSE	LEVEL
NUMBER OF FREEWAY LANES	4
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	10 FEET
LENGTH OF RAMP	350 FEET

Figure 14. Geometric characteristics of Milbrae Avenue southbound entrance ramp, Bayshore Freeway, San Francisco.

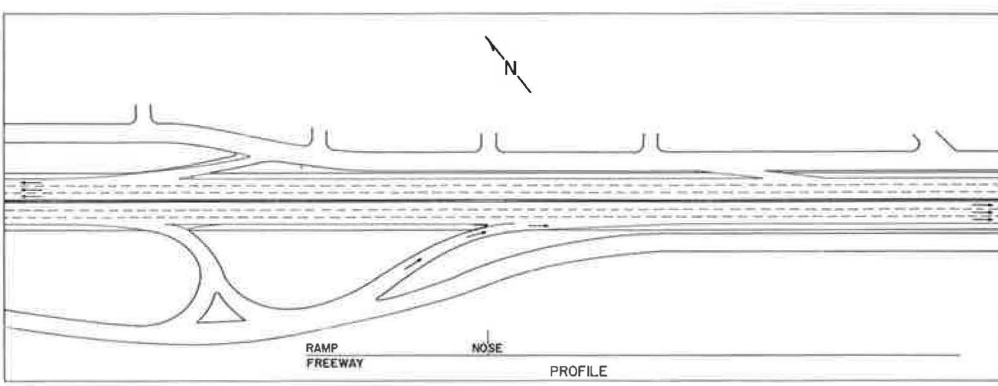


GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	435 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	2°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	2°
RAMP GRADE AT NOSE	-0.51 %
FREEWAY GRADE AT NOSE	-0.51 %
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	380 FEET

Figure 12. Geometric characteristics of Rockaway northbound entrance ramp, Van Wyck Expressway, New York.

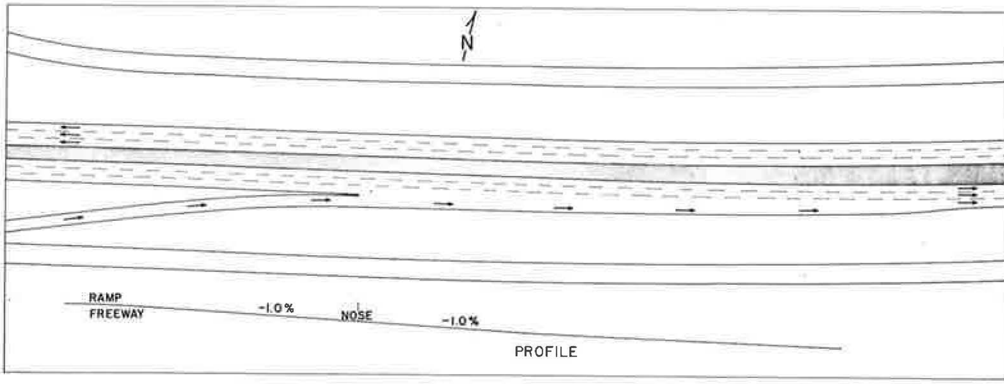
Ground Control Stations

As ground control stations for referencing the position of vehicles as they traverse the study area, marks were made on the shoulder of the freeway and the ramp. Experimentation with various marking materials and mark designs led to the use of 3M Lane Marking Tape in an alternating pattern of arrowhead-shaped and parallel line marks (Fig. 6). The marking tape is a white nonreflective 6-in.-wide tape with a bituminous adhesive on one side, making its application and removal extremely easy and convenient.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	335 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	14°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	14°
RAMP GRADE AT NOSE	LEVEL
FREEWAY GRADE AT NOSE	LEVEL
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	225 FEET

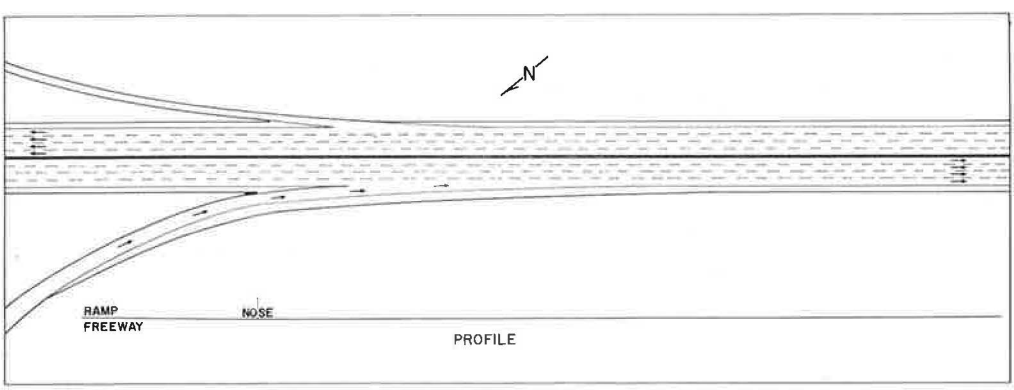
Figure 13. Geometric characteristics of Broad Street southbound entrance ramp, Gulf Freeway, Houston.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	1200 FEET
SHAPE OF ACCELERATION LANE	PARALLEL
ANGLE OF CONVERGENCE AT RAMP NOSE	1°
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	1°
RAMP GRADE AT NOSE	-1.0%
FREEWAY GRADE AT NOSE	-1.0%
NUMBER OF FREEWAY LANES	3
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	SLIGHT RIGHT
CURB OFFSET AT NOSE	2 FEET
LENGTH OF RAMP	600 FEET

Figure 10. Geometric characteristics of Jericho (Rt. 25) eastbound entrance ramp, Long Island Expressway, New York.

International, New York's Kennedy and La Guardia, Chicago's O'Hare and Houston's International. As an additional safety precaution, it was sometimes deemed desirable to have a third person in the airplace as an observer to keep a sharp lookout for other aircraft.



GEOMETRIC CHARACTERISTICS DATA	
LENGTH OF ACCELERATION LANE	1120 FEET
SHAPE OF ACCELERATION LANE	TAPER
ANGLE OF CONVERGENCE AT RAMP NOSE	7°30'
ANGLE OF CONVERGENCE 2 FT. OFF PAVEMENT EDGE	2°
RAMP GRADE AT NOSE	LEVEL
FREEWAY GRADE AT NOSE	LEVEL
NUMBER OF FREEWAY LANES	4
NUMBER OF RAMP LANES	1
FREEWAY CURVATURE	TANGENT
RAMP CURVATURE	TANGENT
CURB OFFSET AT NOSE	10 FEET
LENGTH OF RAMP	500 FEET

Figure 11. Geometric characteristics of Ashby Avenue southbound entrance ramp, Eastshore Freeway, San Francisco.

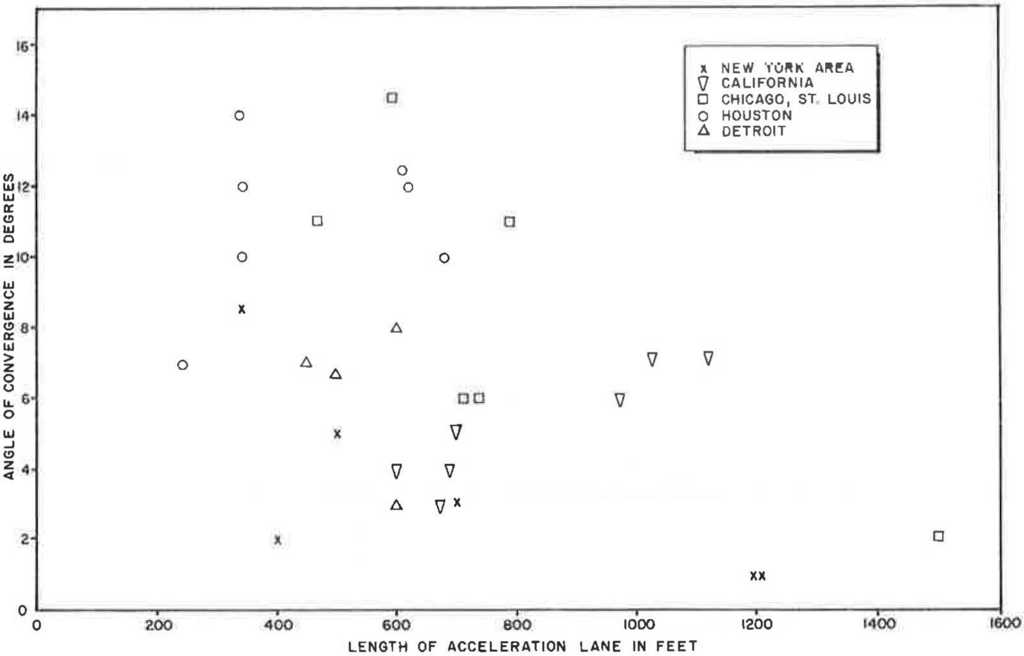


Figure 9. Range of geometrics of ramps studied.

Exposure settings were obtained using a Weston Master V Universal Exposure Meter model 748. It was found desirable to obtain light readings on the ground before filming since exposure settings determined at altitudes above 1000 ft could differ as much as 3 f-stops from that determined at ground level on days of extreme haze conditions, resulting in overexposure of the film. The best image definition was obtained by filming just after a frontal passage or following a rain shower when there is little atmospheric haze.

After the initial teething troubles, this filming technique proved extremely satisfactory. A crew of two, consisting of pilot and cameraman, could take off in the morning, fly several hundred miles, film a location or two and return to home base with minimum disturbance to the local authorities and negligible preparational furor. This mobility, plus the substantially reduced cost involved originally, led to the choice of a light airplane over a helicopter. One problem encountered in the circling technique was that of changing light conditions when the sun is close to the horizon or when the study area is located close to a large body of water. This could lead to the film being overexposed when the camera is in a certain phase of the orbit and then underexposed when it is on the opposite side. This problem could, however, be overcome with a camera equipped with an "electric eye" that automatically adjusts the aperture, or with a camera which allows this adjustment to be made during filming. Interference with other air traffic did not present much of a problem, mainly through the cooperation of the personnel of the Federal Aviation Agency Air Traffic Control Center in each area filmed. Information regarding approach and departure routes of aircraft and safe altitudes for filming was provided by the local air traffic controller. The fine service rendered by the local radar and control tower personnel permitted the filming of study sites in such high-density air traffic areas as Los Angeles' International, San Francisco's

TABLE 1

NUMBER OF RAMPS STUDIED IN EACH CELL OF ACCELERATION LANE LENGTH AND ANGLE OF ENTRY

Angle of Entry (deg)	Acceleration Lane Length (ft)			
	0-400	400-600	600-800	Above 800
0-4	1	2	3	3
4-8	1	4	3	4
Above 8	4	2	4	0

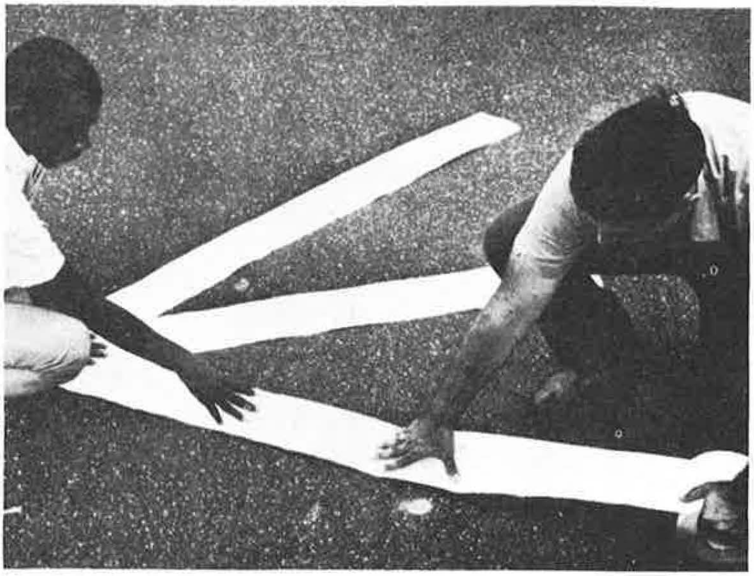


Figure 7. Placing of marker tape.

Figure 8. Tape being rolled down.

In the data collection on this project, three 400-ft magazines were used, while filming was done at a rate of 5 frames per second. This allowed three filming periods of approximately 22 minutes each. Figure 5 shows a sequence of four photographs taken from a film, showing the study area from different positions in the orbit.

Both color and black and white film were used in some of the initial studies, but it was soon found that, at twice the price, color film had little if any advantage over black and white. Kodak Plus-X Panchromatic Type 4231 film having a speed rating of ASA 80 was used mostly, but on overcast days or on days when light readings were low, such as during the peak hours in the winter months, a somewhat faster film, Kodak Double-X Panchromatic Type 5222, with a speed rating of ASA 250, was used. The slower speed film is finer grained than the high-speed film and was therefore preferred whenever light conditions permitted its use.

The filters used to reduce the veiling effect of atmospheric haze, present at all times to a greater or lesser degree, were Wratten No. 8 (K-2), No. 15 (G) or No. 23 (A), depending on the degree of air pollution.

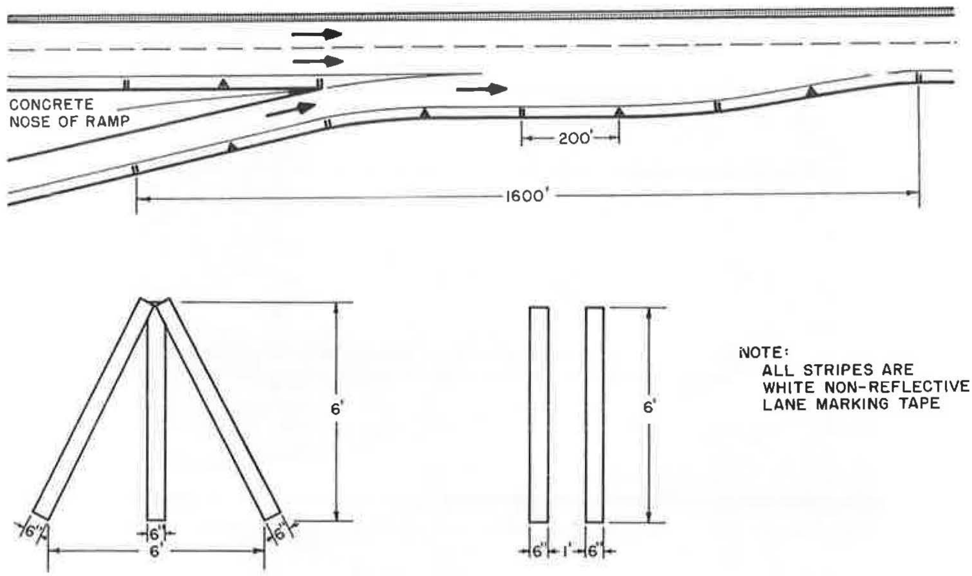


Figure 6. Typical study section.

Aerial Technique and Equipment

The flight pattern consisted of orbiting the study area in a circular or elliptical pattern with a light airplane while training the camera on the center of the study area, usually the nose of the entrance ramp. The aircraft used was a Cessna 206 with the baggage door of the rear passenger compartment modified to accept a large, clear plexiglass window. The left rear seat was removed to permit mounting of the camera inside the aircraft. This allowed filming through the plexiglass window without any noticeable glare or distortion. Figure 2 shows the modification, which has proven quite satisfactory, and Figure 3 shows the camera mounted inside the aircraft.

The camera used was a 35-mm Automax G-2 data recording camera, fitted with Automax Model 250 control head and an Automax 25 intervalometer which could pulse the camera at the rate of ½, 1, 3, 5, or 10 frames per second and with a setting for cine operation at 16 frames per second. Power was supplied to the camera by two 12-volt DC batteries, connected in series to provide sufficient voltage to drive the camera, which maintained a constant speed through a voltage range of 18 to 36 volts DC. The camera has interchangeable lenses, is adaptable to a 400-ft or a 1000-ft film magazine and has a variable shutter speed ranging from $^1/_{64}$ to $^1/_{1000}$ second. The camera also has a data chamber containing a clock with a sweep-second hand, a frame counter and a data slate, all of which is projected as an image on each frame of the film being taken.

The camera was mounted inside the aircraft on a small tripod fitted with a Miller fluid head. The tripod was mounted on special rubber "shoes" to dampen vibrations and secured to the floor of the aircraft with adaptable wires fitted with turnbuckles. To further dampen vibrations, a piece of medium density rubber was installed between the fluid head and the top of the tripod.

Experimentation with lenses of different focal lengths used at different flight altitudes led to the development of the chart shown in Figure 4. This chart was used throughout the study to avoid conflict with other air traffic. It was found that when a distance on the freeway of more than 1800 ft was covered, the film analysis became extremely cumbersone because it was difficult to follow a single vehicle through the study area under conditions of high traffic density. This difficulty was due to shadows, vehicle image size and general lack of definition, caused mainly by atmospheric haze.

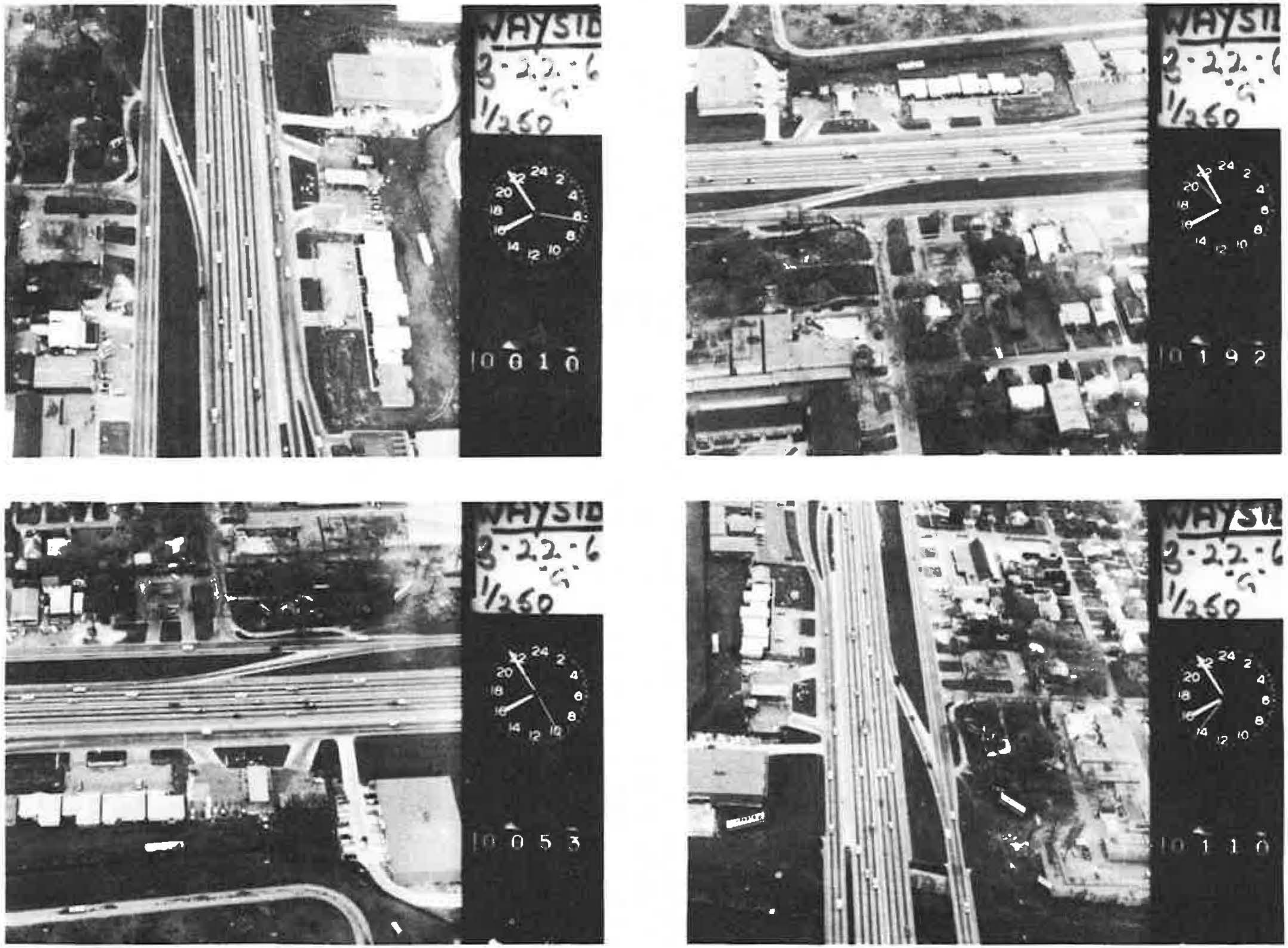


Figure 5. Typical frames from data film.

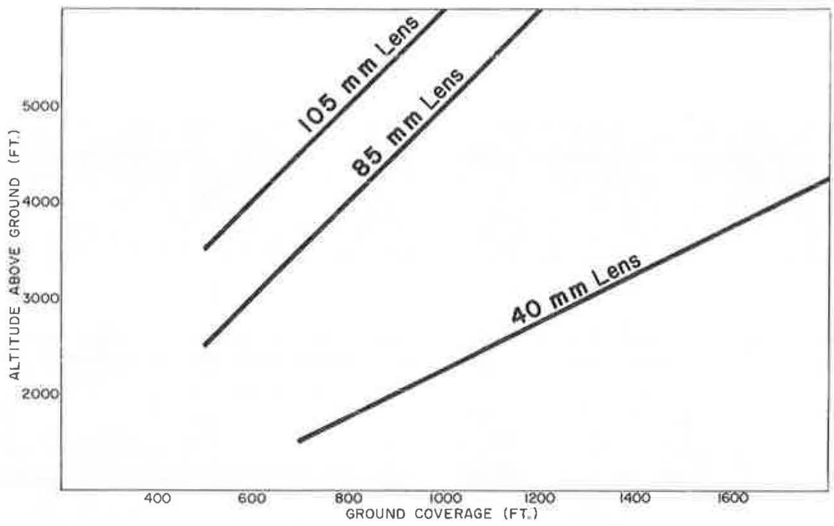


Figure 4. Altitude-ground coverage curves.

ranging over the length and breadth of the continental United States. The study had to be completed in the relatively short time of 18 months, with the result that close attention had to be paid to proper scheduling of the many activities involved in order to attain the wide-ranging objectives. Figure 1 shows the critical path scheduling envisioned at the start of the project.

The objectives of this project are so varied in scope and comprehensive in nature as to prohibit their being covered in their entirety in this report. The main purpose of this paper is to report on the data collection and reduction techniques employed, to describe the ramps under study and to indicate the nature of the data available for purposes of future reference. Furthermore, the authors wish to report on the editing and preliminary analysis of the data to present a comprehensive summary of merging operations at 32 ramp-freeway connections specially chosen to include diverse geometric, geographic, environmental and operating conditions.

DATA COLLECTION

The data collection phase of this study was carried out at 32 entrance ramps located in 8 major metropolitan areas in 6 states. Data were collected on a continuous basis at each ramp to provide the researchers with information on all the variables that could possibly influence the merging maneuver. The many variables involved clearly necessitated the need for a permanent record of the operation, making continual back-reference feasible. Perhaps the best such permanent record is provided by photographic techniques, and the decision to use time-lapse photography was therefore almost a foregone conclusion. Furthermore, the need for collecting data at many different locations, in different parts of the country, made the use of one or more stationary cameras impractical. The data collection procedure had to be independent of the availability of vantage points such as conveniently located tall buildings or overhead structures, and had to be extremely mobile, requiring little equipment and few personnel. Since the studies were to be carried out in areas under the jurisdiction of various highway authorities another requirement of the procedure was that it should interfere as little as possible with the day-to-day operation of the organization concerned. All these considerations led to the development of an aerial photographic technique for the continuous study of a length of highway.

Figure 2. Aircraft door modification.

Figure 3. Camera mounted in aircraft.

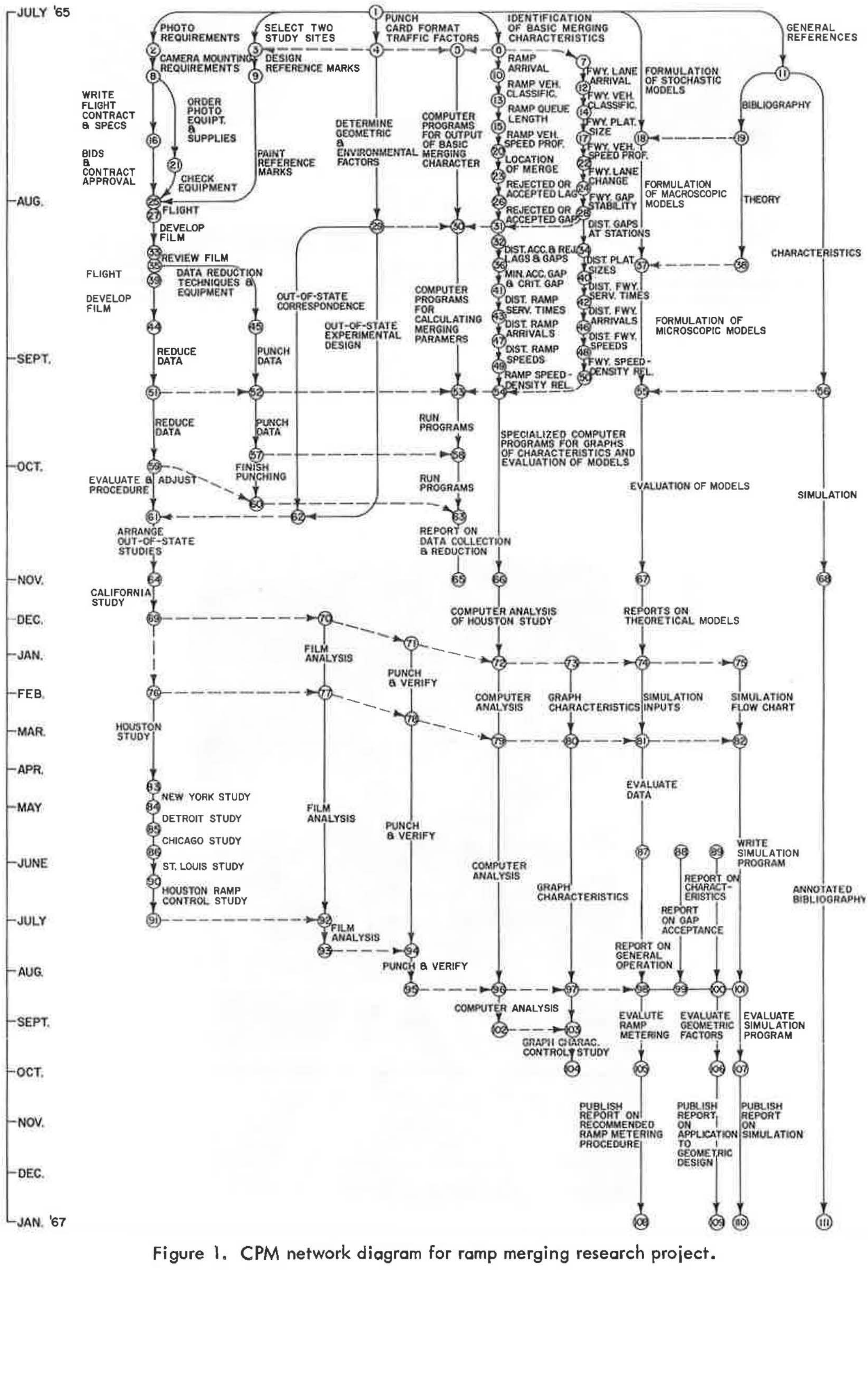


Figure 1. CPM network diagram for ramp merging research project.

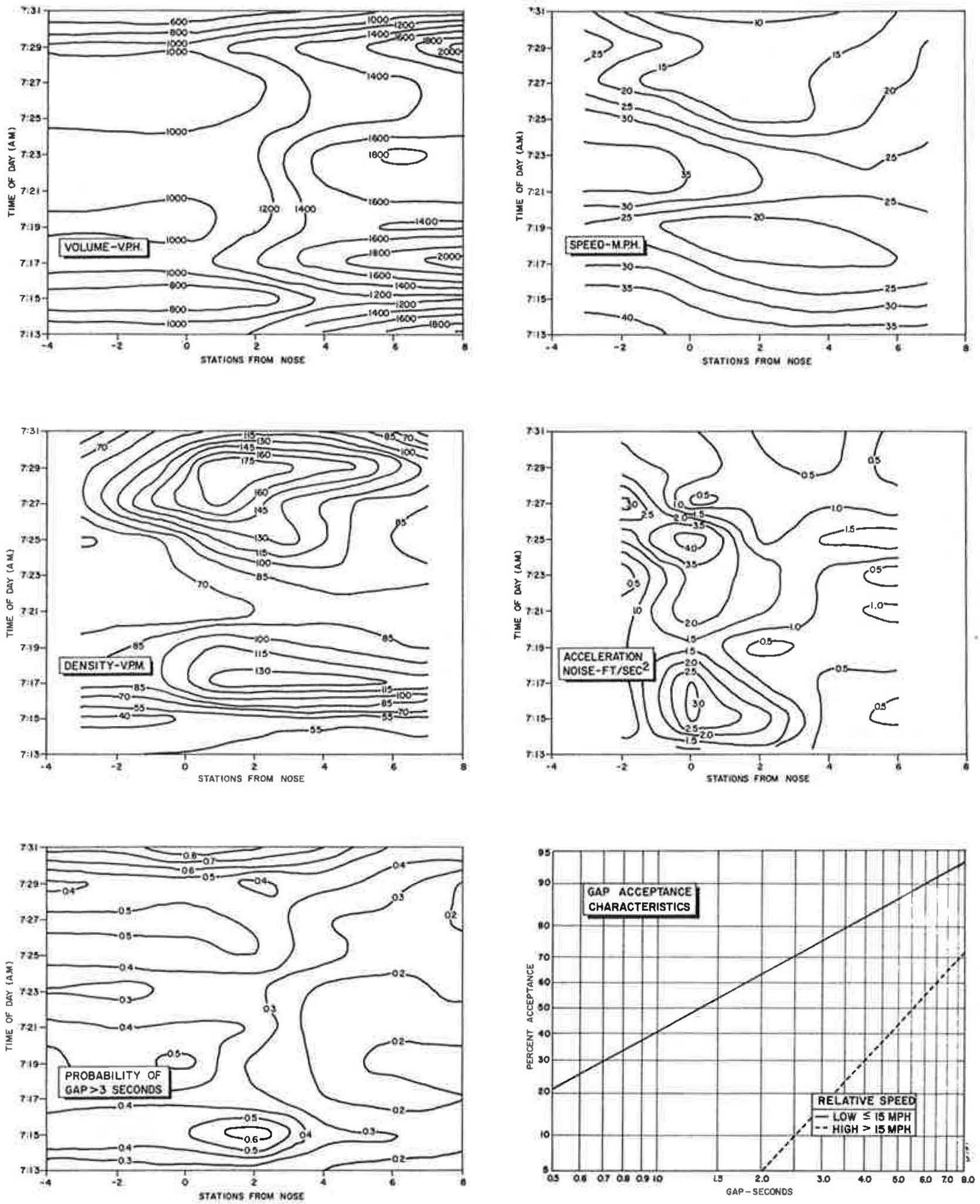


Figure 42. Traffic characteristics of Ashby Avenue southbound entrance ramp, Eastshore Freeway, San Francisco.

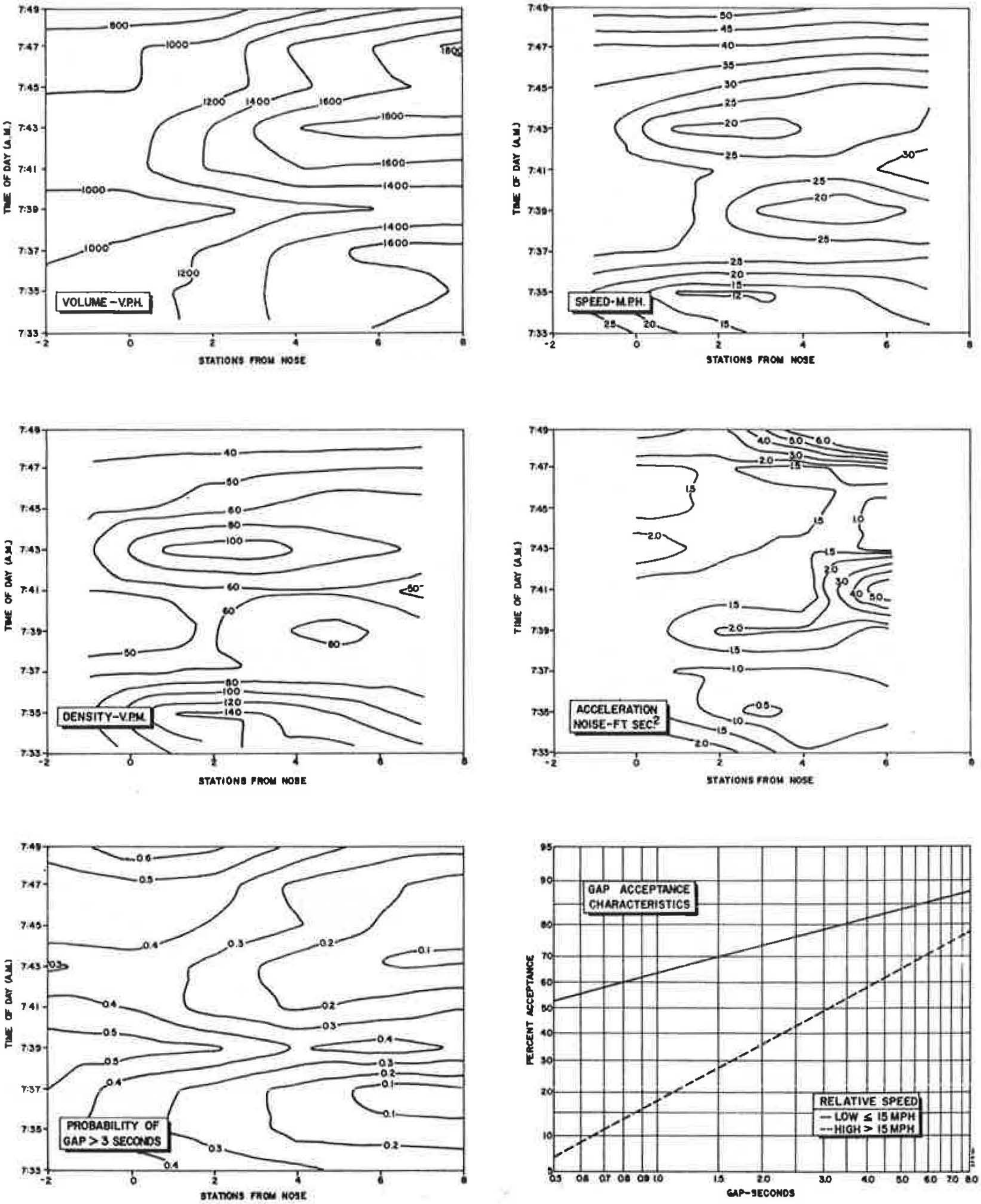


Figure 43. Traffic characteristics of Broadway northbound entrance ramp, Bayshore Freeway, San Francisco.

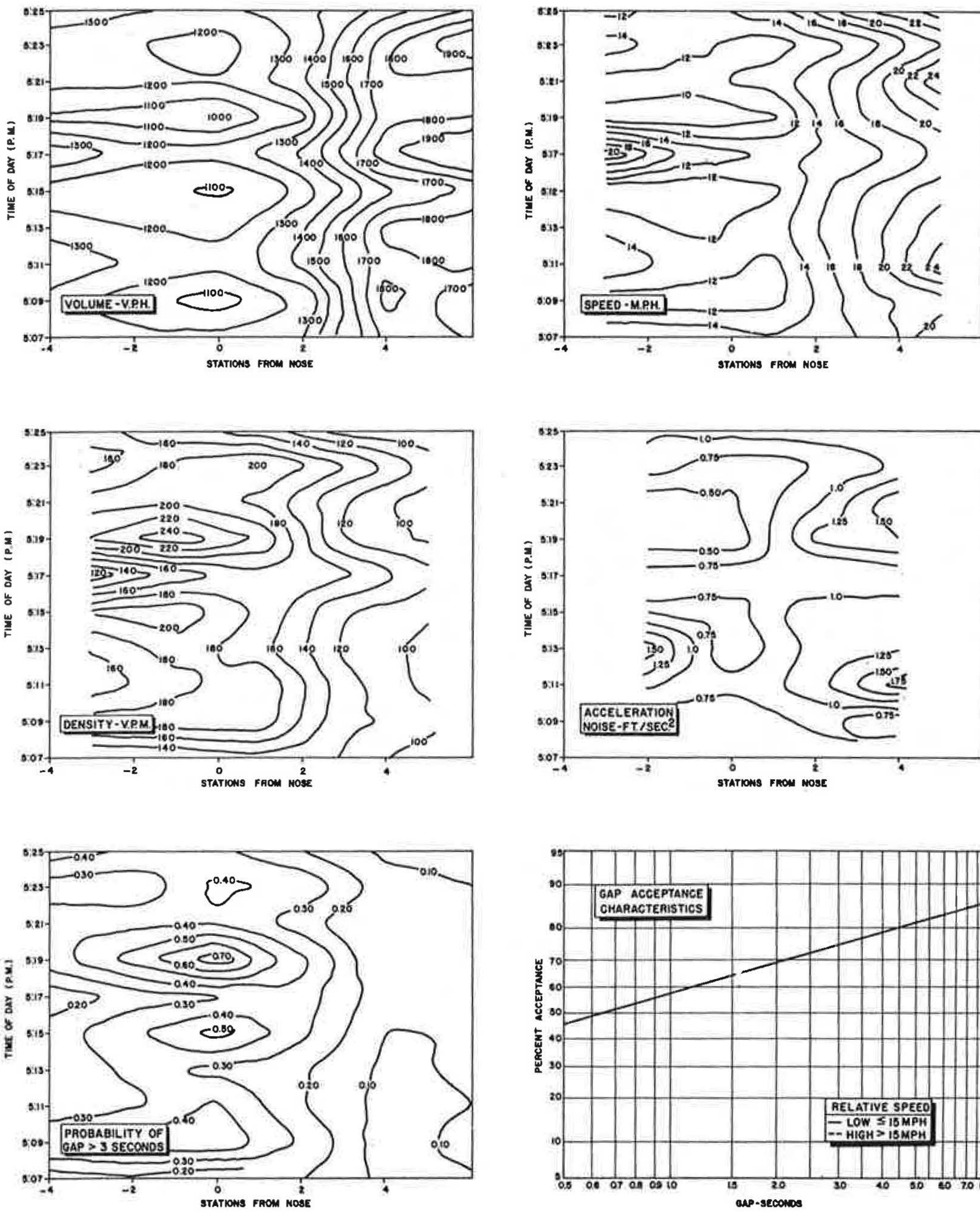


Figure 44. Traffic characteristics of Linwood northbound entrance ramp on the John Lodge Expressway, Detroit.

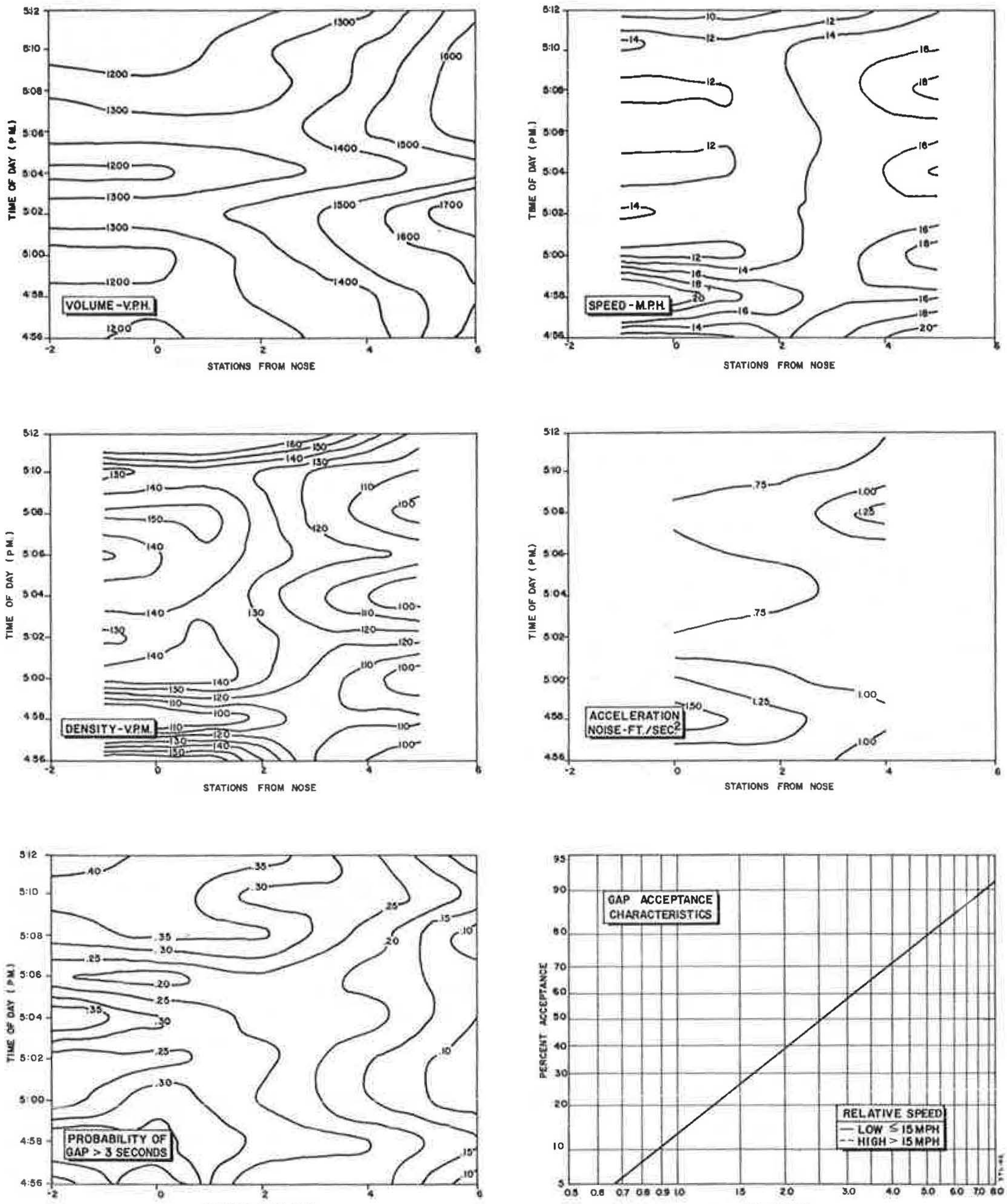


Figure 45. Traffic characteristics of Brentwood westbound entrance ramp to Daniel Boone Expressway, St. Louis.

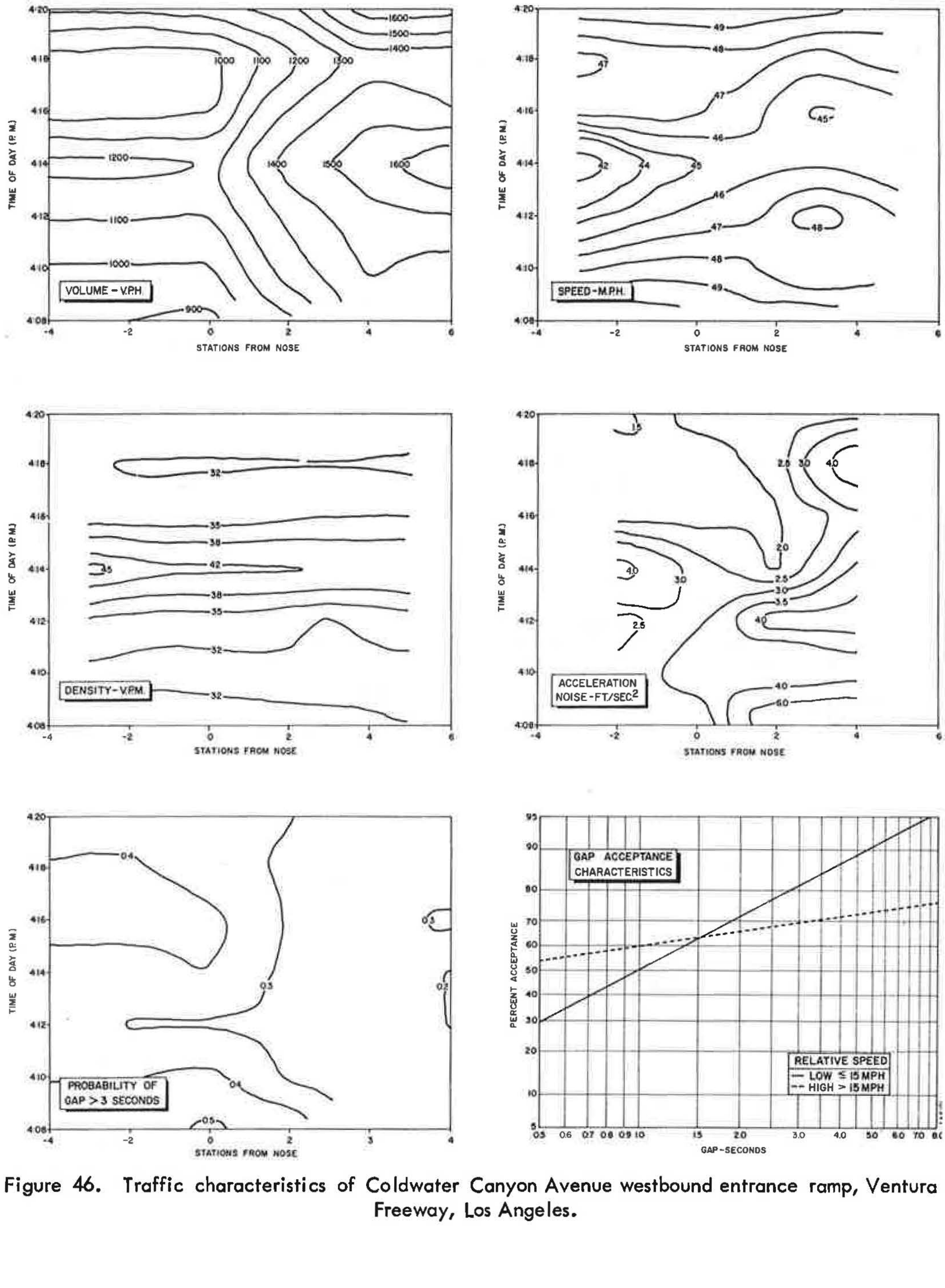


Figure 46. Traffic characteristics of Coldwater Canyon Avenue westbound entrance ramp, Ventura Freeway, Los Angeles.

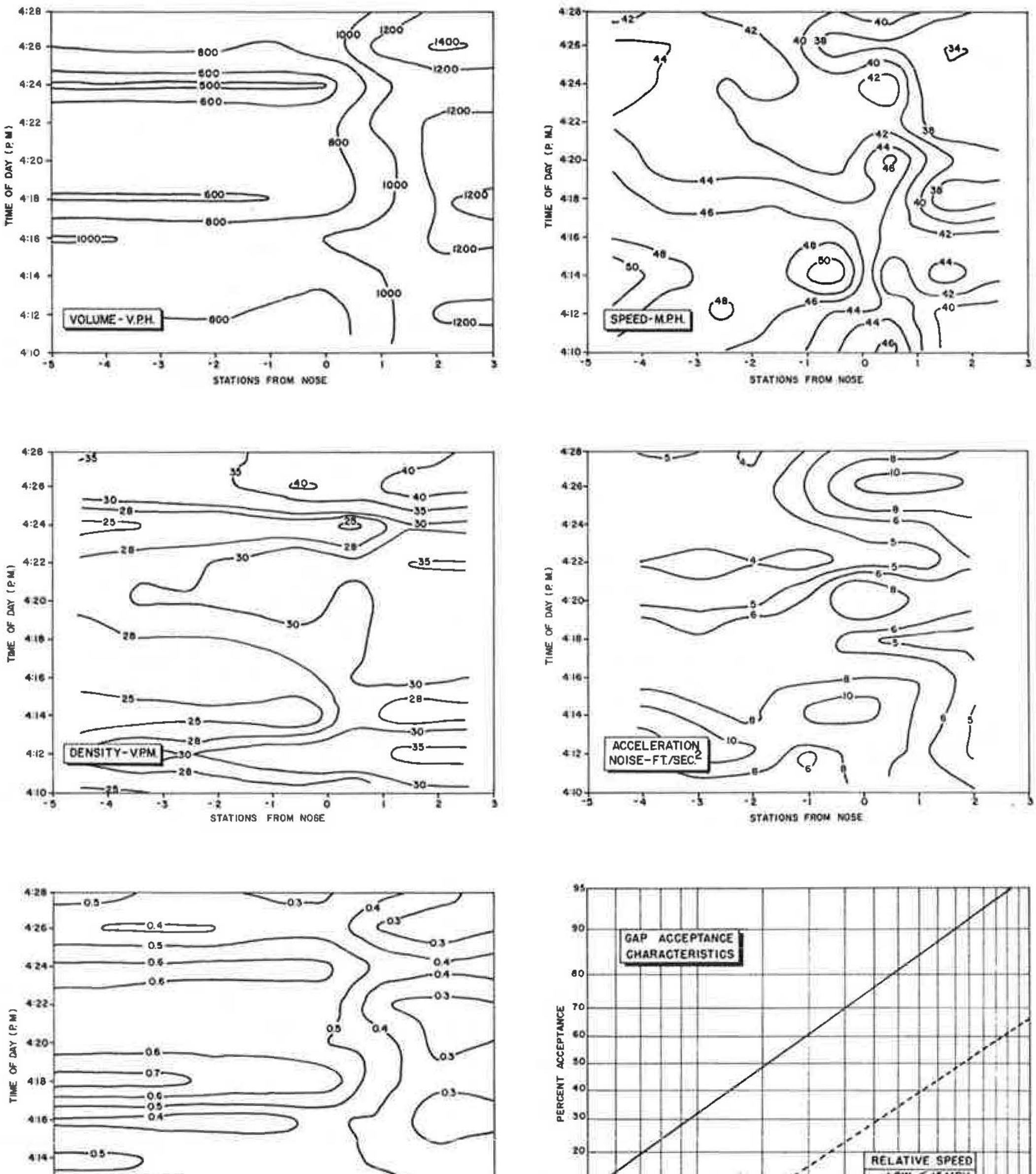


Figure 47. Traffic characteristics of Dumble eastbound entrance ramp on the Gulf Freeway, Houston.

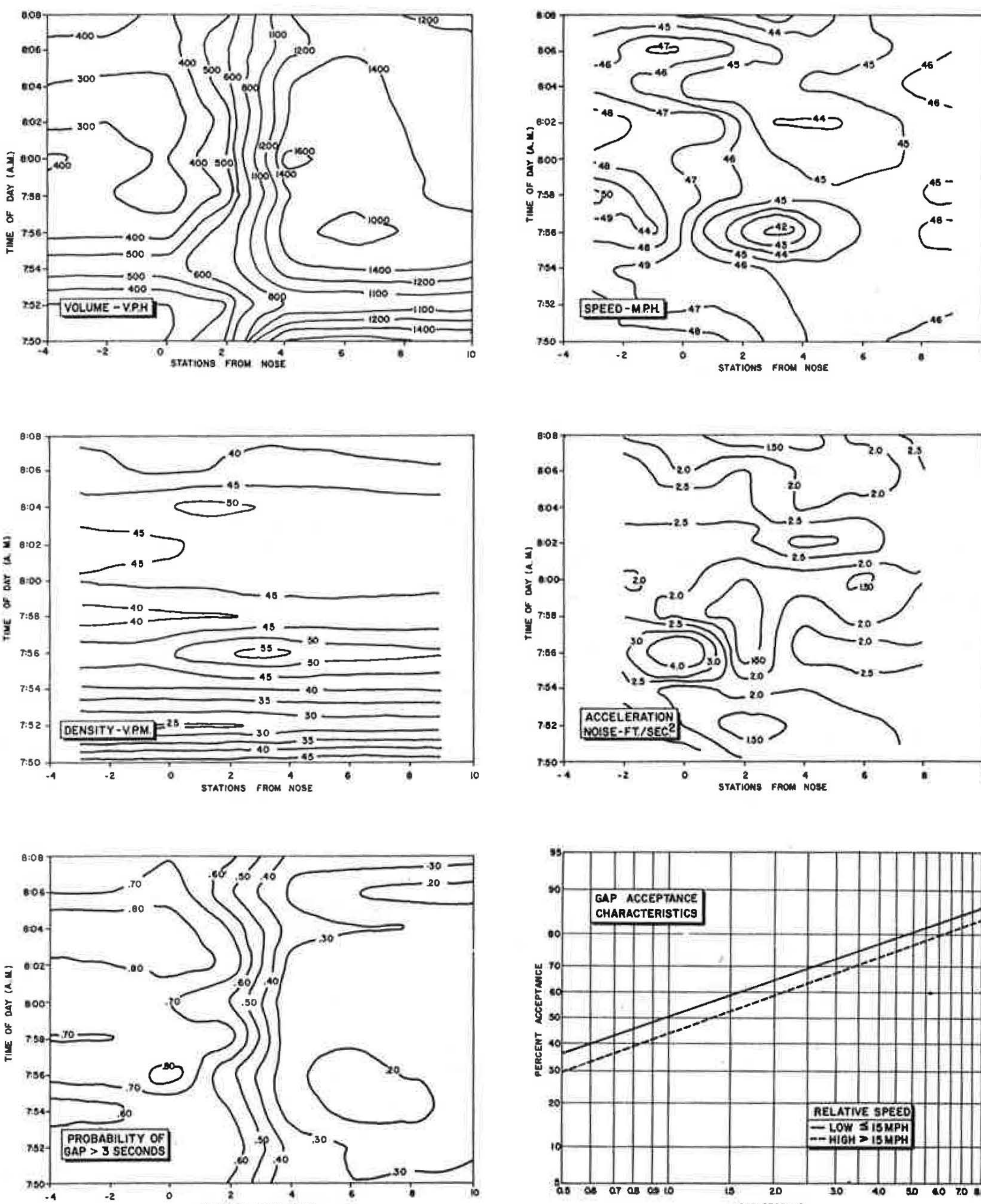


Figure 48. Traffic characteristics of Long Island Expressway northbound entrance ramp to Cross Island Expressway, New York.

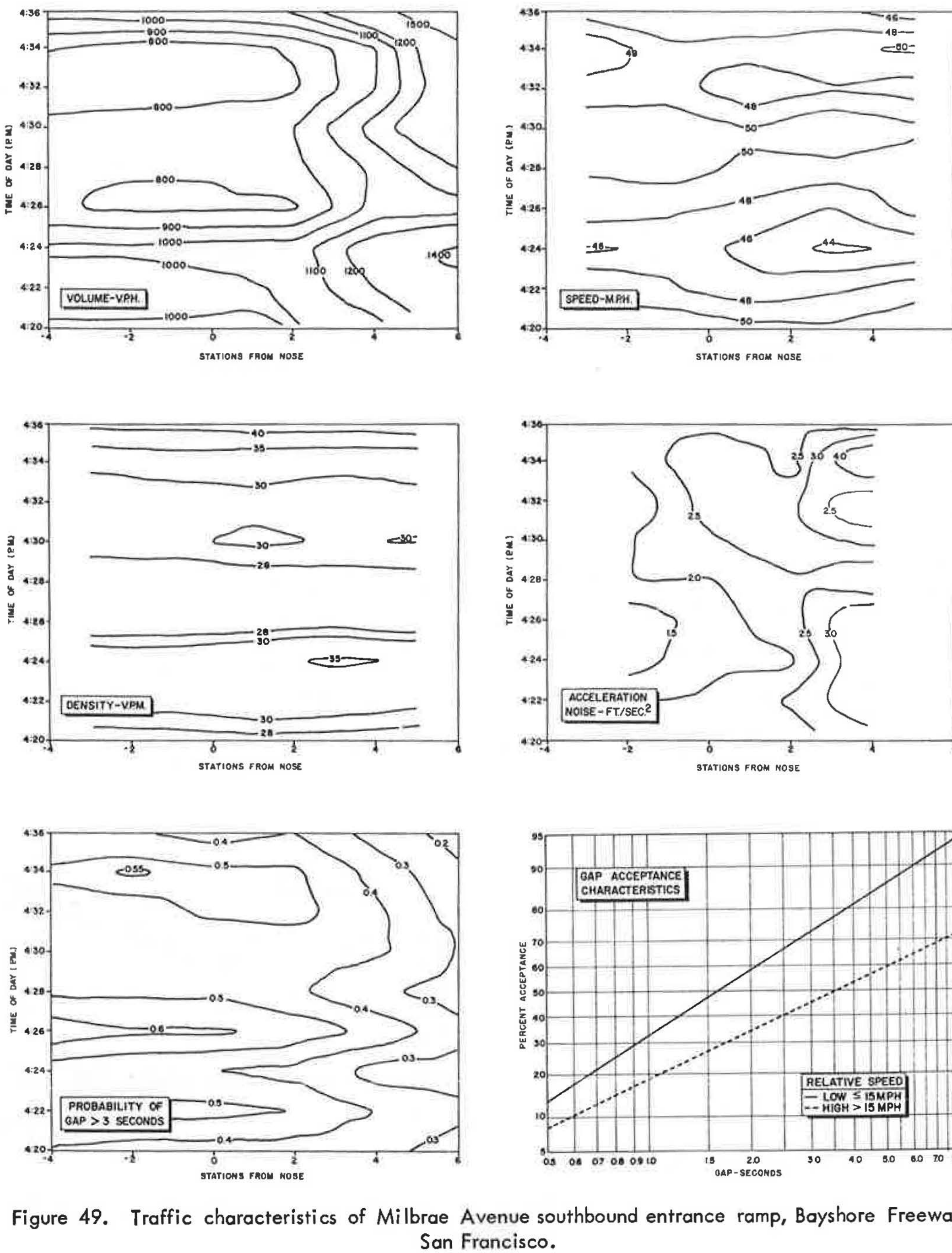


Figure 49. Traffic characteristics of Milbrae Avenue southbound entrance ramp, Bayshore Freeway, San Francisco.

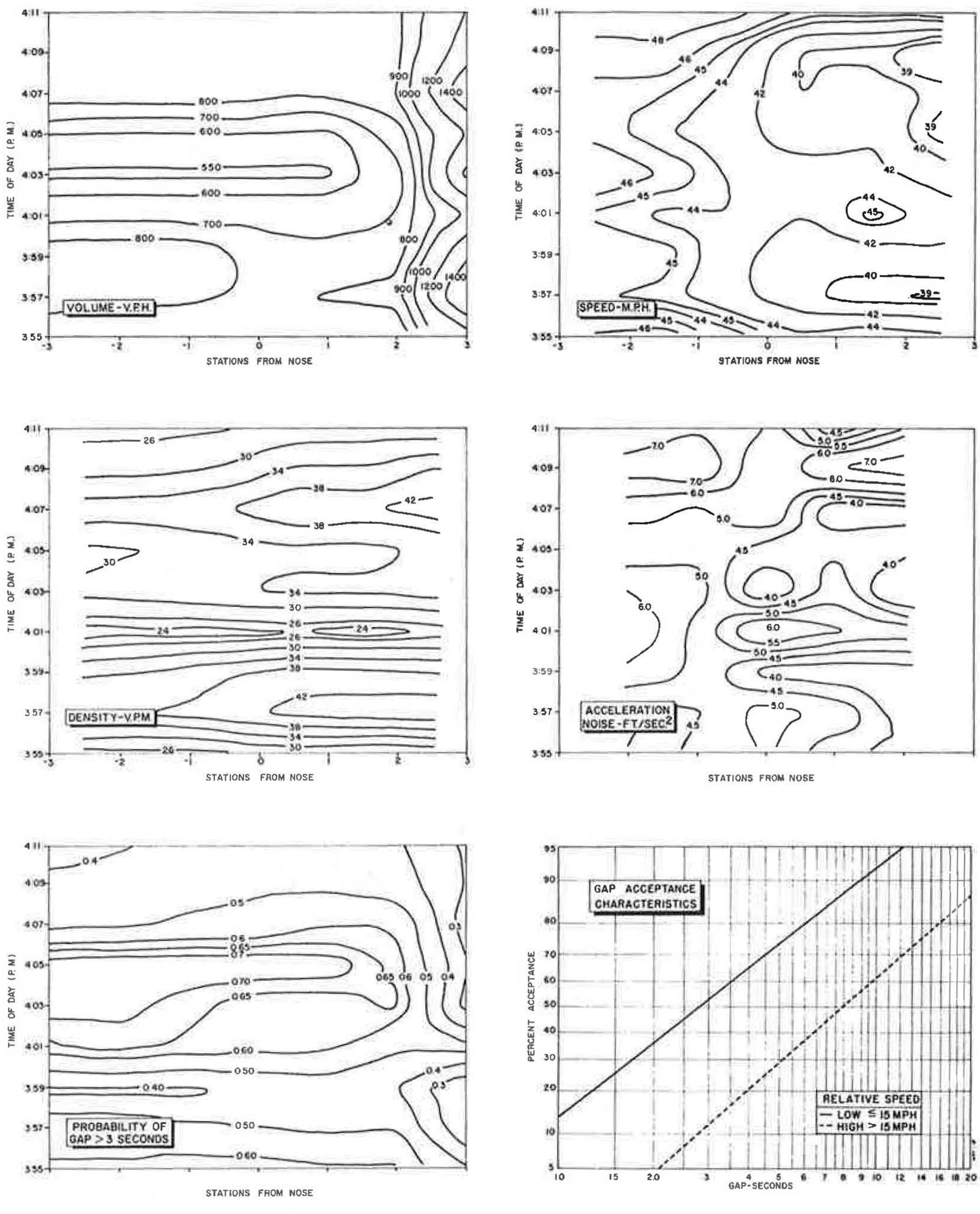


Figure 50. Traffic characteristics of Wayside eastbound entrance ramp on the Gulf Freeway, Houston.

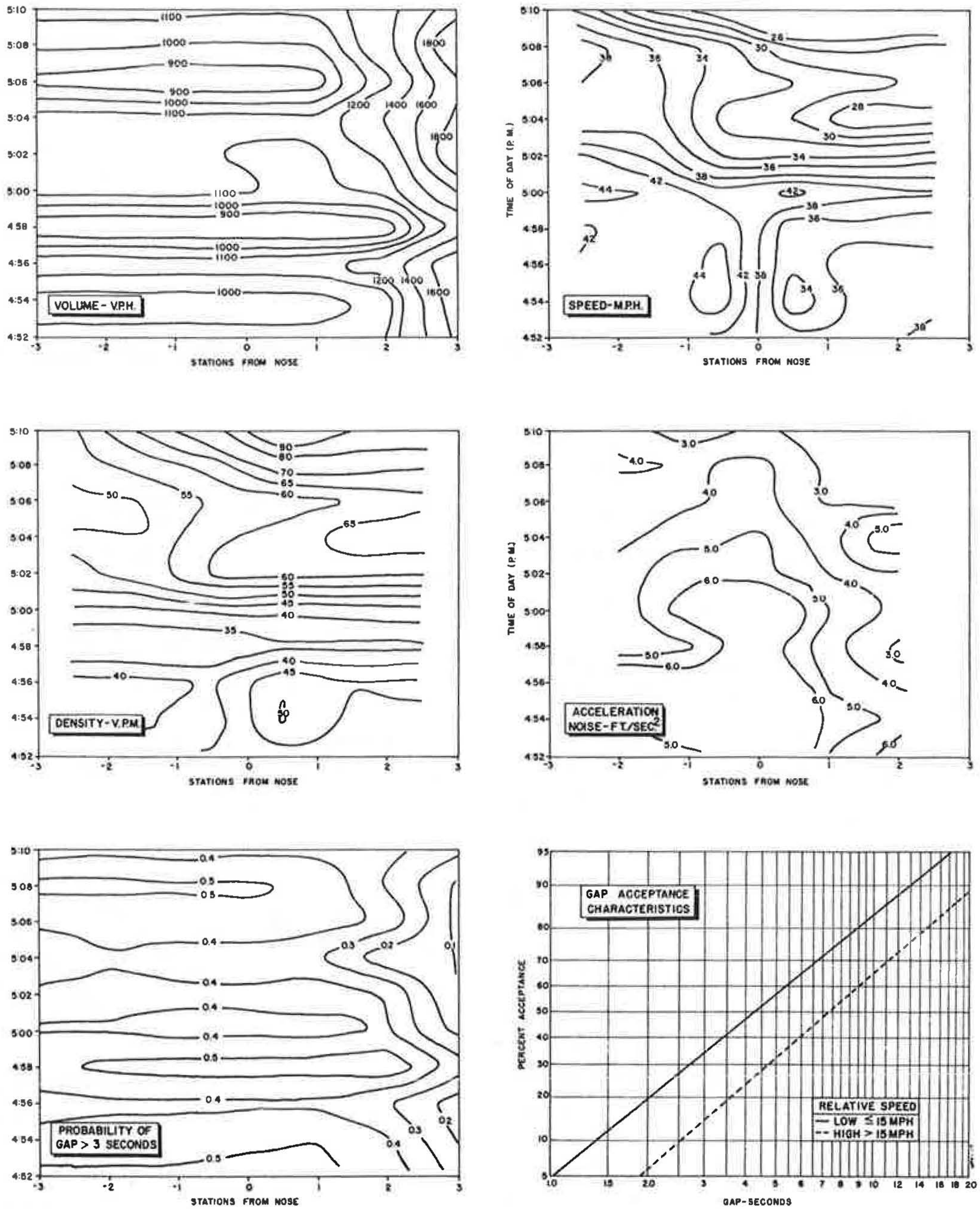


Figure 51. Traffic characteristics of Wayside eastbound entrance ramp on the Gulf Freeway, Houston.

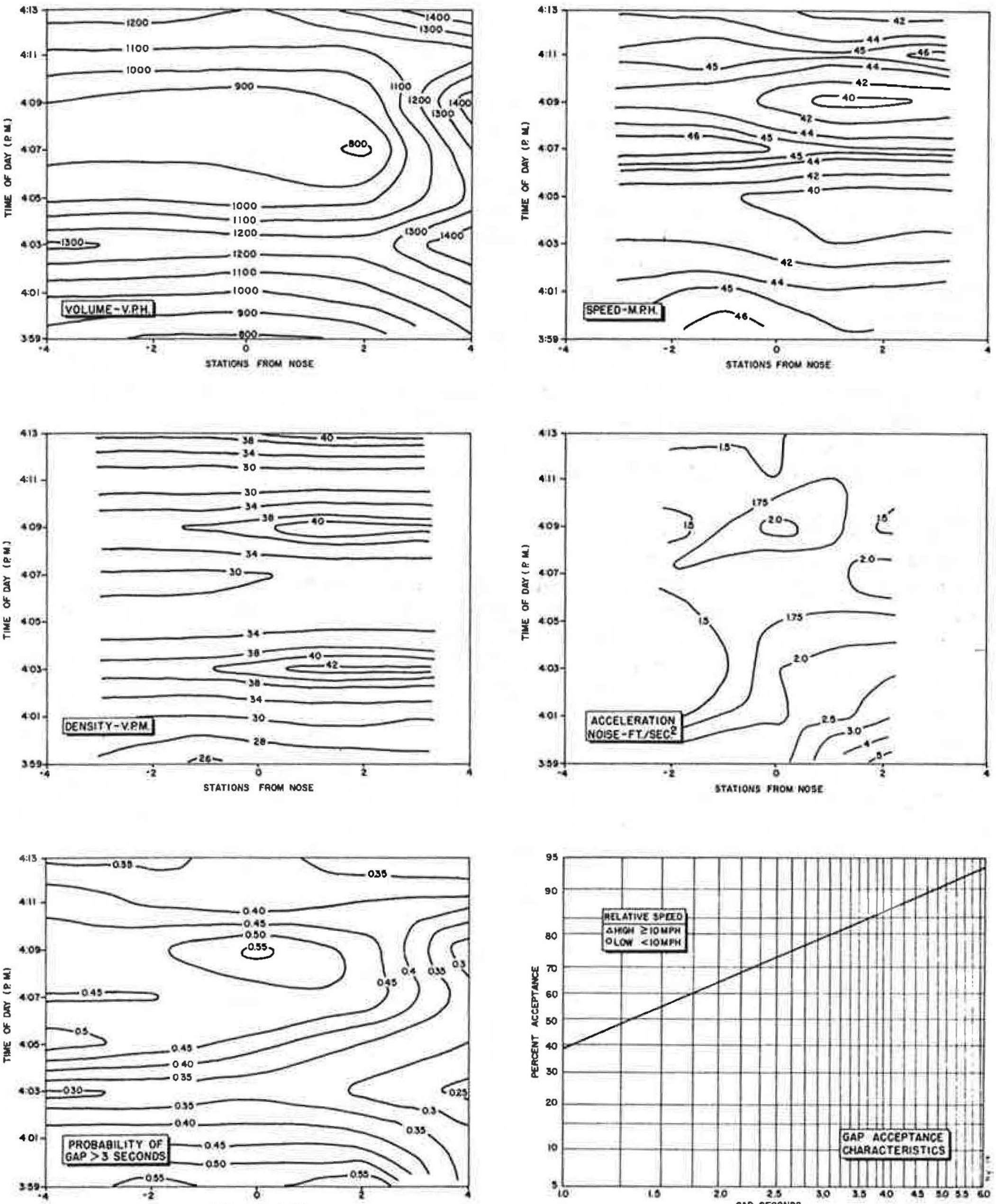


Figure 52. Traffic characteristics of 69th Road southbound entrance ramp on the Grand Central Parkway, New York.

Effect of Acceleration Lane Length

The effect of the length of acceleration lane is illustrated by Figures 46 and 47. These two figures were selected not only for the similarity of general geometric features other than acceleration lane lengths of the ramps to which they apply, but also for the similarity in the ranges of speeds and densities. The freeway volume was lighter but since both merging areas were operating at similar speeds and densities, the effects of the traffic demand on the acceleration noise are minimized. Perhaps the most striking difference in the operations of the merging areas is the general smoothness of flow at the ramp with the larger acceleration lane as exhibited by all the contour diagrams. A comparison of the volume contours clearly reveals the gradual increase in volume level over the length of the acceleration lane of the Coldwater Canyon ramp as compared to the sudden increase over a short distance at the Dumble ramp, resulting in considerable turbulence in speeds and densities. This turbulence caused the higher acceleration noise observed at the Dumble ramp. Even the probability contours display this smoother operation at the Coldwater Canyon ramp. These two figures form a good demonstration of the desirability of the longer acceleration lane.

The operation at a high-volume ramp with a very long acceleration lane is illustrated by Figure 48. Notice that although the acceleration lane is 1200 ft long, by far the largest proportion of ramp vehicles chose to enter the freeway within the first 400 ft. This type of operation was also evidenced by other studies at this and another similar ramp. However, in spite of the extremely high entrance ramp volume, high speeds were maintained and the acceleration noise remained at a low level.

Effect of Angle of Convergence

In order to demonstrate the effect of the angle of convergence two sets of data were selected that were gathered at ramps with geometrics that differed mainly in the angle of convergence, during periods when speeds and densities were fairly comparable. These are shown in Figures 49 and 50. The Wayside entrance ramp with the higher angle of entry seems to cause a more sudden increase in volume levels over a shorter distance than the Milbrae ramp with the smaller angle of convergence. This causes a greater turbulence in speeds and densities resulting in a generally higher level of acceleration noise at the Wayside ramp. This may be partially caused by the traffic demand since the freeway volume upstream of the nose is generally lower at the Wayside ramp, giving a higher ratio of ramp volume. However, comparisons with a study period at Wayside when the freeway volumes were higher, such as in Figure 51, show that speeds generally decreased and densities increased, resulting in a slightly lower acceleration noise but which was still generally higher than that at the Milbrae ramp.

Figures 52 and 53 serve as another illustration of the effects of the angle of convergence. In this case the ramp geometrics differ again mainly in the angle of convergence, but both are of such high type designs that the value of 5 degrees, as against 11 degrees, perhaps gives a wrongful impression of the actual difference.

By inspection of the contour diagrams, it can be seen that volumes, speeds and densities were comparable during the study periods at these two ramps. The resulting acceleration noise at the 69th Road ramp with the lower angle of entry seems to be generally lower than at the Peterson Avenue ramp. However, the difference in ramp profiles should be noted and may lead one to conclude that the effect of the difference in angle of convergence at these two ramps is not noticeable in the contour diagrams.

Effect of Grades

Figures 54 and 55 demonstrate the operation at two ramps that are remarkably similar in most geometric aspects. Both have 1200-ft long parallel lane type acceleration lanes with ramps converging at one degree at the nose. They differ only slightly in the grades along the acceleration lane, in that the Jericho ramp is slightly downgrade and the Long Island Expressway ramp slightly uphill.

The general pattern of operation is also remarkably similar at the two ramps, as evidenced by the contour diagrams. At both locations, most of the ramp vehicles

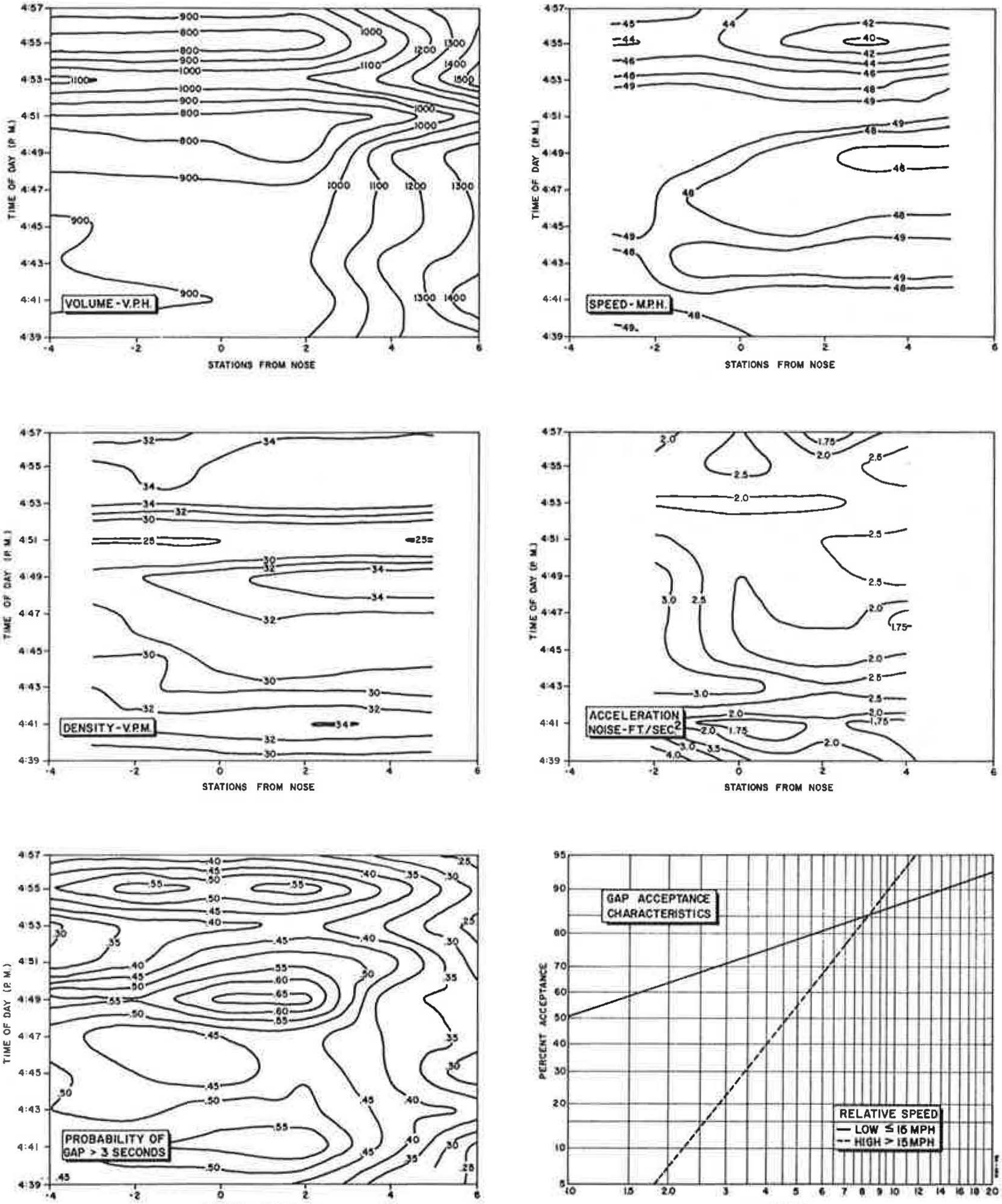


Figure 53. Traffic characteristics of Peterson Avenue northbound entrance ramp to Edens Expressway, Chicago.

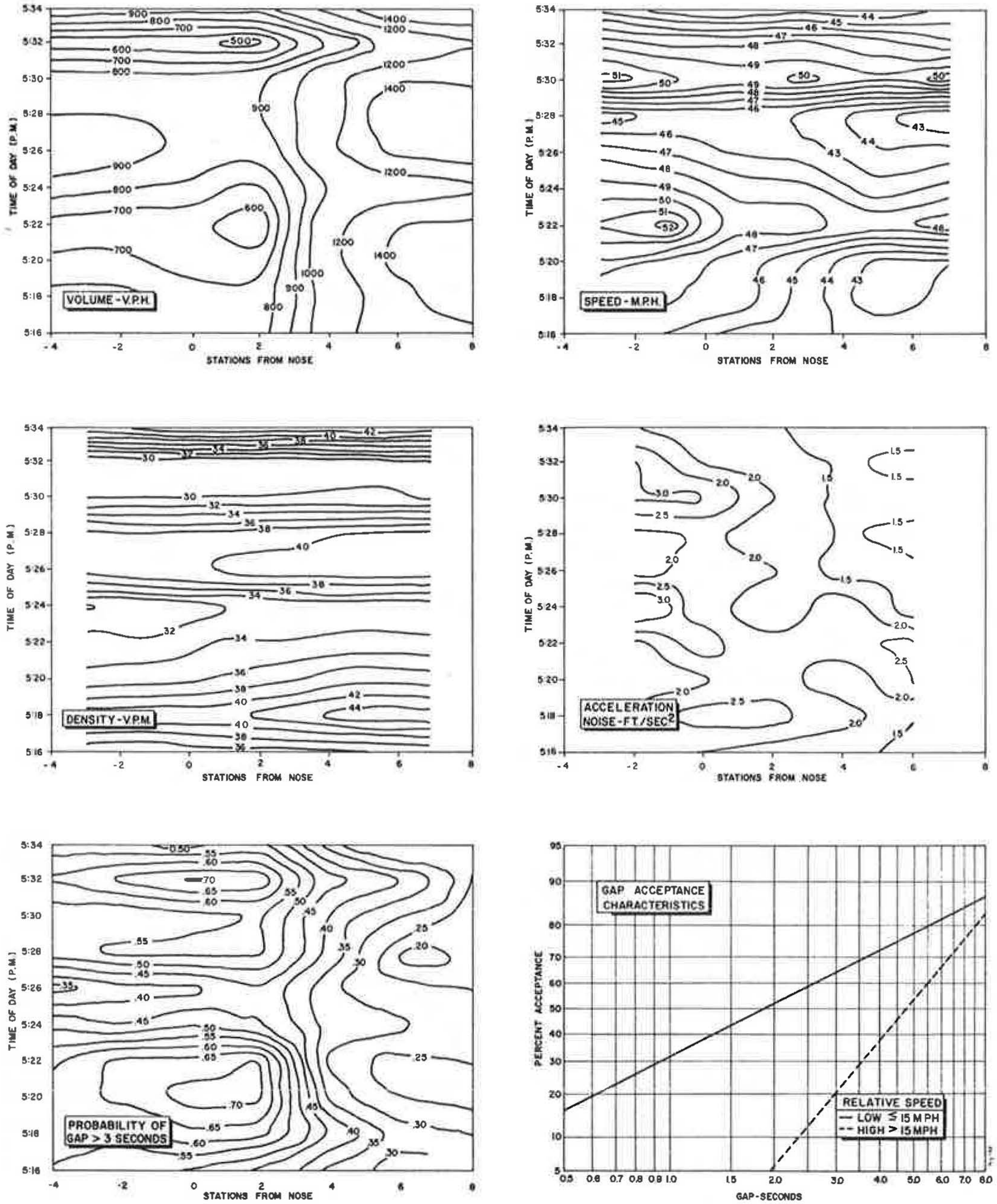


Figure 54. Traffic characteristics of Jericho (Rt. 25) eastbound entrance ramp on Long Island Expressway, New York.

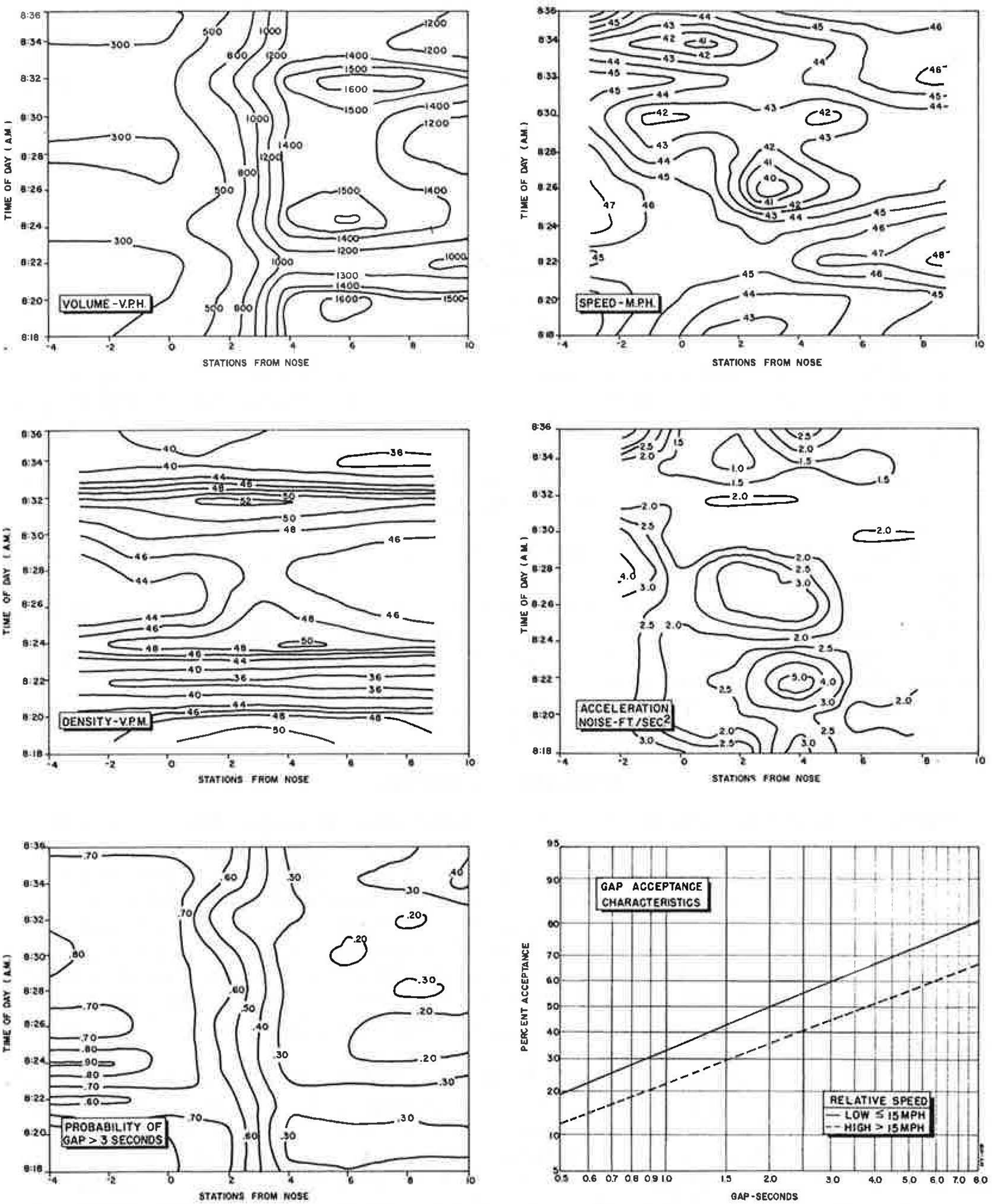


Figure 55. Traffic characteristics of Long Island Expressway northbound entrance ramp to Cross Island Expressway, New York.

entered the freeway within the first 500 to 600 ft of the acceleration lane length. The total merging volumes at the two locations were comparable, but the ramp volume at the Long Island ramp far exceeded that at the Jericho ramp, giving rise to slightly lower speeds and higher densities, resulting in a higher level of acceleration noise. Because of this uneven split in the traffic demands, it is difficult to isolate the effect, if any, of the slight difference in grades. It does seem, however, that at the upgrade ramp, vehicles entered the freeway generally sooner than at the Jericho ramp, which may well have been caused by the grade.

COMMENTS

1. An aerial photographic technique has been developed for collecting large masses of comprehensive traffic data with relative economy.
2. The manner and mobility of the photographic technique make it generally applicable to almost any study site located anywhere.
3. Since no distances are measured off the films, the study technique is not dependent on maintaining a strict flight altitude or flight path and is not affected by highway elevation changes.
4. Problems encountered in the filming studies included the mounting of the equipment in the aircraft, development of the proper flight path, evaluation and acquiring of the proper hardware and software, and coping with the changing light conditions during circling.
5. Problems encountered in the data reduction included the careful training of film analysis personnel, lack of definition in some films and the problem of missing data points.
6. In the study of the effects of geometrics on traffic operation, care should be taken to avoid periods of forced flow.
7. Time-space diagrams of vehicle paths, such as illustrated in Figure 40, not only reveal the operation on the facility at a glance, but also are an invaluable aid in the editing of data, the writing and debugging of more complicated computer programs and meeting the problem of missing data points.
8. Contour diagrams are an effective way of illustrating the operation of a facility on a continuous basis in both time and space.

ACKNOWLEDGMENTS

The success of this phase of the project with the national scope of its field studies is due to a large extent to the cooperation of the Federal government, six State governments, and many local governmental agencies. The aid of the following personnel, their staffs and agencies is noted: James E. Wilson, Karl Moskowitz and Leonard Newman of the California Division of Highways; Charles H. McLean of the Illinois Division of Highways; Joseph Marlowe of the Michigan State Highway Department; James Little and James Roberts of the Missouri Highway Department; Dale D. Marvel, John N. Lipscomb and William V. Ward of the Texas Highway Department; Alger F. Malo of the Detroit Department of Traffic; and Edward Bonelli of the New York City Department of Traffic.

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