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Surveillance, and
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7 Reports

Subject Areas

- 52 Road User Characteristics
- 53 Traffic Control and Operations

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Foreword

The ever-increasing traffic congestion being experienced on freeways, especially in urban areas, has caused attention to be focused on methods for improving traffic operations on these facilities. Of paramount interest also is the matter of adequate communications on these facilities. Much of the information in the papers and discussions in this RECORD will, if applied, help to ease some of the problems now being felt for the first time in many areas.

The first paper, by a team of Chicago area researchers, attempts to determine driver preferences for display-type information needed for driving on heavily trafficked freeways. It was found that traffic-type information was preferred and that descriptors relating to an accident ahead or speed were the most desirable. Of least use were terms pertaining to delay and travel time. Three discussers have added their comments regarding the research, and the authors have responded with an appropriate closure.

The second paper, by a California Division of Highways researcher, examines the feasibility of an exclusive transit (bus) lane on the San Francisco-Oakland Bay Bridge. An exclusive bus lane was found not to be feasible because the increased delay to automobile passengers far exceeded the savings to the bus passengers. The study also indicated that factors that kept people off buses were related to inconvenience, discomfort, crowding, and route and schedule problems.

The next paper, by a team of Chicago area researchers, presents empirical research concerning lane-changing on multilane roadways. The findings indicate that randomness characterized the changing, and there were weak but systematic relationships to traffic and roadway conditions.

The fourth paper, by three California Division of Highways researchers, evaluates ramp control on the Harbor Freeway in Los Angeles. Basically the research indicated that such ramp control reduced delay significantly to freeway motorists with little or no travel time increase on adjacent streets. The project was estimated to have saved at least 50,000 vehicle-minutes per day. Some 92 percent of motorists indicated approval of the metering controls.

The next paper, by two men from Polytechnic Institute of Brooklyn, describes some analytic results obtained from a study of a freeway control and surveillance project. Various methods of ramp control modes were examined in detail and observations are presented as to the efficacies of the various types, especially in light of the difficulty of controlling all the variables present in the freeway merging situation.

A Michigan Department of State Highways researcher studied usage of a motorist-aid telephone system on a rural freeway and presents his findings in the next paper. He found that 52 percent of drivers needing aid used the phones and some 97 percent of those aided believed the system should be expanded to cover additional mileage. Quantitative values are given in the report as to detailed usage and characteristics.

The last paper, by a member of a New York commercial research firm, sets forth design considerations for a type of motorist-aid system called FLASH. The report gives the concepts and methodology used, and describes a prototype FLASH installation in Florida.

The papers in this RECORD provide interesting theoretical and case study approaches to problems of operation, communication, and surveillance on limited-access facilities. As congestion increases and possibilities of constructing new highways decrease, methodology as set forth in these papers must be increasingly applied.

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An Analysis of Driver Preferences for Alternative Visual Information Displays

KENNETH W. HEATHINGTON, Purdue University;
RICHARD D. WORRALL, Peat, Marwick, Mitchell and Company; and
GERALD C. HOFF, Chicago Area Expressway Surveillance Project

Several traffic information descriptors have been evaluated as to how desirable they are to drivers residing in the Chicago metropolitan area. These descriptors are messages about the conditions on a freeway and are displayed by electronic signs. Evaluated were 7 descriptors at the level of heavy congestion, 6 at the level of moderate congestion, and 6 at the level of no congestion. The messages contained descriptive information, quantitative information, and no information. For all levels of congestion, traffic information was preferred to no information about traffic conditions. For the level of heavy congestion, information about an accident having occurred and causing heavy congestion was the descriptor most preferred by respondents in the total sample. The speed descriptor ranked second (the accident descriptor ranked first) for the level of heavy congestion and first for the other two levels of congestion. The two remaining descriptors (excluding the accident descriptor) were scaled fairly high, but were less desirable than the speed descriptor. The two quantitative descriptors, delay and travel time, had relatively low scale values and were simply not desired by the respondents.

•IN THE UNITED STATES the number of urban residents increased by 30 million from 1955 to 1965 (1). During this same time, vehicle ownership in the United States increased to 85 million vehicles and transit patronage decreased by over 3 billion rides annually. The result has been increasing congestion on urban highway systems, increasing transportation costs, and increasing calls for more efficient use of existing streets and highways to counter the considerable costs of constructing new facilities. (Transit includes surface street car, bus, or trolley bus in local urban service as well as subways or elevated rail rapid transit. Public transportation includes transit plus rail commuter services and taxis. Highway facilities or highway system includes all types of facilities available for use by the automobile. The transportation system includes facilities of all modes of transportation.)

Most urban highways, and particularly freeways, operate satisfactorily for a large portion of the day, but they become extremely congested during the morning and afternoon peak periods. Even in low-density urban areas, the freeway volume carried during the morning peak period can exceed 20 percent of the total 24-hour volume (2). The inevitable result of this concentration of demand is a sharp reduction in level of service.

Although freeway facilities may be extremely overloaded at certain times, frequently other parallel elements of the highway system have low utilization during these peak periods (3). One possible method for providing a higher level of peak-period service on existing freeways, therefore, appears to be a redistribution of demand. Conceptually, this could be accomplished by mandatory controls and by voluntary choice on the part

of individual drivers. Mandatory controls enforce the use of designated routes. At the beginning of a journey, a motorist would be assigned a route, and he would be required to take the particular route designated. Voluntary choice would include the provision of information to the motorist that would enable him to select the more appropriate route for his journey. Voluntary choice, though certainly more desirable in principle, requires as an essential additional input the provision of information to the driver on the alternatives available to him and the levels of service they offer. (Information includes information on traffic and road conditions, routing, and directional guidance. Traffic information is information on speed, volume, minutes of delay, and the like associated with a given route.) In other words, a system based on voluntary choice has to give the driver a good reason for changing his route away from the freeway onto an alternate road. At present only primitive techniques are available for transmitting this type of information and are, all too often, both inaccurate and untimely. In the Chicago metropolitan area, helicopter traffic reports are broadcast over local radio stations; in isolated areas, other forms of traffic information are available.

An information system is needed, therefore, that could provide the highway user with real-time or pseudo-real-time information on the operational status of alternative highway facilities. (Real-time information is instantaneous information regarding conditions on the freeway. However, the best interests of the controlling agency or the driver may not be served by furnishing all information in real time. When it is more desirable to delay by some specific time interval the furnishing of information, the delayed information is in pseudo-real-time.) Such a system, if it can be made to work, can well offer substantial returns in terms of improved system performance.

The motorist beginning a journey usually has little or no knowledge of the operating conditions existing on any of the available routes between his origin and destination. Generally, his expectations of the conditions that are likely to prevail and of the alternatives available to him are founded solely on his previous experience. His perceived notions of the traffic conditions on any given route can be the result of several information sources such as the news media, either radio or paper, friends, or his own past driving experience. If he selects a route that is heavily congested, the journey becomes taxing for himself and serves to increase the inefficiency of overall system performance. Better alternative routes may be available to him, but he seldom has sufficient information on which to base an intelligent choice.

Once a driver has begun his journey, he may encounter congestion caused either by the heavy demand placed on his route or by the occurrence of an incident, not necessarily an accident, that causes a breakdown in flow. (An incident may be an accident or the placing of warning signs near the roadway by a road repair crew. An incident is anything that causes a breakdown in operation.) Under such circumstances some drivers will voluntarily divert to other routes to avoid the congested conditions. Others will simply remain on the preselected route. In either case, the individual driver has very little real-time information available to help him make a wise decision as to the correct procedure to follow. The provision of such information at the appropriate point along a route could, and probably would, cause a voluntary diversion of a proportion of drivers and cause in turn a shift in demand and a more rapid return to an efficient level of overall system operation.

DESIGN OF STUDY

A research study was undertaken to evaluate specific traffic information descriptors (outputs) to be used by a Freeway Driver Information System (FDIS). As conceived here, the FDIS would operate only on selected freeways and at selected locations adjacent to these freeways. It is designed to increase the efficiency of existing facilities by promoting a voluntary redistribution of demand so that new facilities do not have to be constructed. The FDIS will operate in real-time, furnishing information about traffic conditions to freeway users. Psychological scaling techniques were used to evaluate driver preferences for selected visual information displays. This research was directed toward evaluating visual displays rather than audio broadcasts; however, findings are thought to be applicable to both.

In the following discussion, the level of information refers to heavy, moderate, or uncongested traffic conditions. The form of information may be descriptive such as moderate congestion or quantitative such as the number of minutes of travel time. The method of paired comparisons used to evaluate the level and form of information preferred by drivers is described in detail in the literature (4, 5). A sample of 732 drivers residing in Cook County, Illinois, was selected, and each driver was administered a structured questionnaire to obtain responses for the paired comparison evaluation. Each driver owned a car and made a regularly scheduled automobile trip to a place of employment, school, or other location. Residences of the drivers were uniformly distributed throughout Cook County. The 7 descriptors for heavy congestion, 6 for moderate congestion, and 6 for no congestion are given in the following list:

Heavy Congestion

1. Speed 5 to 15 mph—next 3 miles
2. Heavy congestion—next 3 miles
3. Stop and go traffic—next 3 miles
4. Extra delay 10 to 20 minutes—next 3 miles
5. Accident—heavy congestion—next 3 miles
6. Travel time 15 to 25 minutes—next 3 miles
7. Blank sign

Moderate Congestion

1. Speed 20 to 30 mph—next 3 miles
2. Moderate congestion—next 3 miles
3. Heavy, steady, traffic flow—next 3 miles
4. Extra delay 0 to 10 minutes—next 3 miles
5. Travel time 5 to 15 minutes—next 3 miles
6. Blank sign

No Congestion

1. Free-flowing traffic—next 3 miles
2. Uncongested—next 3 miles
3. Extra delay 0 minutes—next 3 miles
4. Speed 45 to 55 mph—next 3 miles
5. Travel time 3 to 8 minutes—next 3 miles
6. Blank sign

Each descriptor at each level of congestion was paired with all other descriptors at that level. The respondent was told that the signs (descriptors) in each pair described the same traffic conditions on an expressway. The respondent was asked to indicate whether he preferred sign A or sign B as a form of traffic information.

The number of pairs to be compared at each level is $n(n - 1)/2$ where n is the number of descriptors. A total of 51 pairs were generated and viewed by each respondent: 21 pairs and 15 pairs each for moderate congestion and uncongested conditions. Figure 1 shows typical signs that were presented to respondents. The paired comparisons and the descriptors within each pair were randomly ordered for different respondents to avoid bias caused by respondent fatigue or the tendency of some respondents to always choose the first or second descriptor in each pair.

The messages of the descriptors contained (a) descriptive information, (b) quantitative information, and (c) no information. The descriptors giving speed, travel time, and delay information were considered to be quantitative, and the blank sign gave no information. The remaining signs gave descriptive information.

DRIVER PREFERENCES FOR DESCRIPTORS

The P_{JK} matrix for heavy congestion, moderate congestion, and no congestion is given in Tables 1, 2, and 3 respectively. These tables give the percentage of the time



Figure 1. Typical traffic information descriptors.

TABLE 1
PERCENTAGE OF TIME DESCRIPTOR J IS PREFERRED OVER
DESCRIPTOR K FOR HEAVY CONGESTION

K Descrip- tors	J Descriptors						
	1	2	3	4	5	6	7
1	—	47.59	46.32	36.97	66.71	26.49	13.31
2	52.41	—	46.74	37.11	77.05	33.29	13.74
3	53.68	53.26	—	32.86	70.82	33.43	17.71
4	63.03	62.89	67.14	—	76.35	41.08	22.95
5	33.29	22.95	29.18	23.65	—	28.05	12.46
6	73.51	66.71	66.57	58.92	71.95	—	29.32
7	86.69	86.26	82.29	77.05	87.54	70.68	—

TABLE 2
PERCENTAGE OF TIME DESCRIPTOR J IS PREFERRED OVER
DESCRIPTOR K FOR MODERATE CONGESTION

K Descrip- tors	J Descriptors					
	1	2	3	4	5	6
1	—	44.33	41.93	27.20	21.25	18.98
2	55.67	—	52.97	37.82	37.82	23.37
3	58.07	47.03	—	28.75	27.20	24.50
4	72.08	62.18	71.25	—	49.72	29.18
5	78.75	62.18	72.80	50.28	—	37.68
6	81.02	76.63	75.50	70.82	62.32	—

TABLE 3
PERCENTAGE OF TIME DESCRIPTOR J IS PREFERRED OVER
DESCRIPTOR K FOR NO CONGESTION

K Descrip- tors	J Descriptors					
	1	2	3	4	5	6
1	—	43.06	18.98	57.08	27.62	35.98
2	56.94	—	21.81	58.22	26.06	33.43
3	81.02	78.19	—	79.75	46.03	35.84
4	42.92	41.78	20.25	—	20.40	20.96
5	72.38	73.94	53.97	79.60	—	43.06
6	64.02	66.57	64.16	79.04	56.94	—

each descriptor J was preferred over descriptor K at a given level of congestion. For heavy traffic (Table 1), descriptor 1 (speed 5 to 15 mph—next 3 miles) is preferred over descriptor 2 (heavy congestion—next 3 miles) 52.41 percent of the time, descriptor 5 (accident—heavy congestion—next 3 miles) is consistently preferred over all other descriptors, and descriptor 7 (blank sign signifying that information is not furnished) is generally the least preferred. For all 3 levels of congestion, a blank sign is the least preferred of all the descriptors.

The P'_{JK} matrix was used to develop a ranking or scaling of the descriptors for each level of congestion. These scale values for each descriptor, shown in Figures 2, 3, and 4, have several similar characteristics. At all 3 congestion levels, the blank sign has the lowest scale value and is, therefore, taken to represent a scale value of zero. The respondents interviewed preferred some form of traffic information on expressways regardless of the level of congestion. Even when there is no congestion and traffic is free-flowing, the respondents exhibited a preference for traffic information to be displayed. This seems to indicate that drivers want to be kept informed at all times about conditions to be encountered ahead of them on an expressway.

At all 3 levels of congestion, 2 quantitative descriptors were relatively undesirable to the respondents and had little meaning to them relative to the other descriptors.

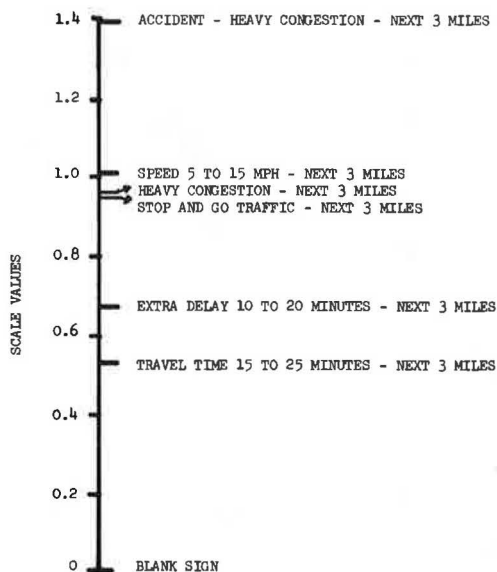


Figure 2. Scaling of descriptors for heavy congestion.

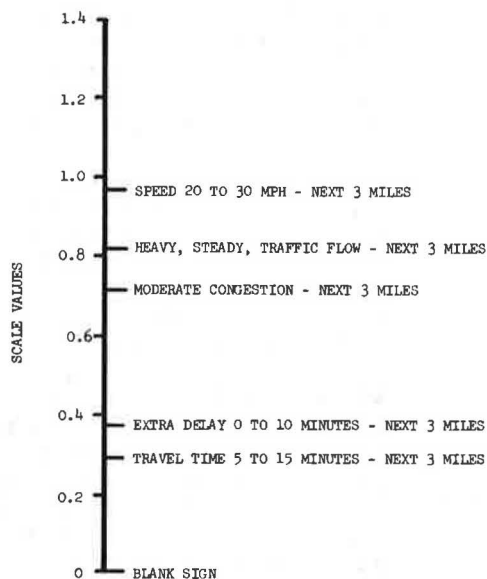


Figure 3. Scaling of descriptors for moderate congestion.

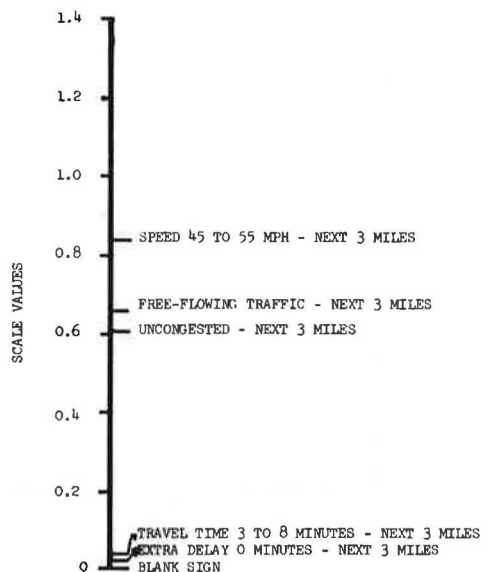


Figure 4. Scaling of descriptors for no congestion.

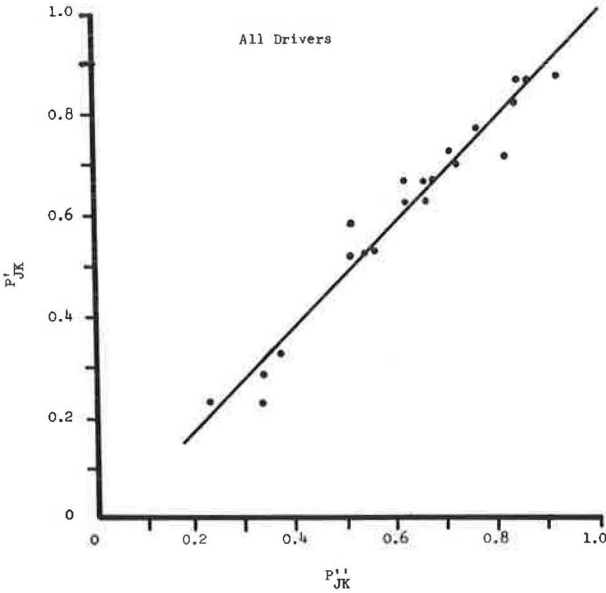


Figure 5. Calculated versus observed proportions of preference of descriptor J over K for heavy congestion.

The quantitative term of speed was highly preferred. At the levels of moderate congestion and no congestion, it was the most preferred of all the descriptors. That the drivers interviewed scaled the speed descriptors as they did is perhaps not too surprising because all the driving experience of each respondent has been related in some form to a speedometer. The driver often measures his ability to negotiate a given route by the speed at which he can drive, rather than by the extra delay involved or the travel time required. The speedometer is always available for reference and probably is more available than a reference to time either in delay or in total travel.

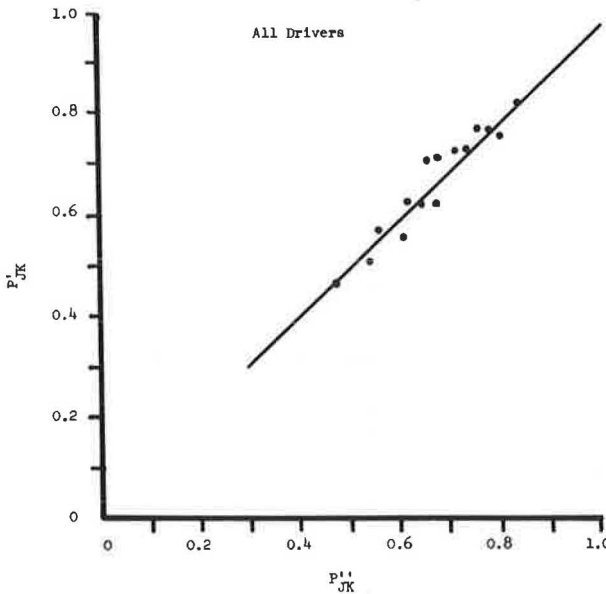


Figure 6. Calculated versus observed proportions of preference of descriptor J over K for moderate congestion.

These 2 represent minutes of travel time and delay, and their scale values at the level of no congestion (Fig. 4) are approximately equal to that of a blank sign.

For all 3 levels of congestion, the 2 descriptors pertaining to traffic conditions were preferred to the travel time, delay, and the blank sign descriptors. The traffic-condition descriptors have a relative scale value at each level of congestion that places them in the upper priority of traffic information descriptors. They are simple, concise, and easily comprehended. The driver does not have to make any mental calculations to translate them into meaningful information. It may be questionable whether a given descriptive term means exactly the same thing to each respondent, but the preferences of all respondents gave these descriptive terms relatively high priority ratings.

One additional descriptor used for the level of heavy congestion informed the driver that an accident had occurred and heavy congestion was on the expressway ahead. At the level of heavy congestion, this descriptor was preferred above all other descriptors, even to the one for speed. One might intuitively suggest several reasons why respondents prefer to know that an accident

has occurred on the expressway ahead. In the Chicago area, accidents on major expressways can cause extreme congestion and long delays. Many times these accidents are minor but, because of a gapers' block by drivers going in both directions, traffic tie-ups can be extremely critical. A gapers' block is the situation where drivers slow down to a crawl speed to view an accident. The accident can be a minor or major one, but the disruption to traffic is caused not by the lanes of traffic being blocked by the accident but by drivers slowing down to view the accident. When the exit ramps are located at fairly long spacing intervals, a driver can easily become ensnared in a traffic tie-up and lose a significant amount of time. It would be difficult to imagine that anyone who has driven in the Chicago area for any length of time has escaped

being in a traffic tie-up caused by an accident. One could perhaps well argue that one involvement is sufficient to cause drivers to desire information that might help them avoid such incidents. Some may argue that information on accidents would draw curiosity seekers, but the authors believe that the desire to avoid a traffic tie-up is the primary reason that drivers gave this descriptor the highest scale value.

EVALUATING THE MODEL

A variety of checks may be made to evaluate the internal consistency of the scaling technique. The least squares estimates of the scale values were evaluated for each level of congestion. Once the least squares estimates of the scale values are determined, a P'_{JK} matrix representing the derived percentages that descriptor J is preferred over descriptor K is formed. If the least squares estimates are unbiased, the P'_{JK} values form a linear relationship with the observed P'_{JK} entries. Figures 5, 6, and 7 show the relationship of the P'_{JK} entries to those of the P'_{JK} matrix.

The plots of the P'_{JK} and the P'_{JK} matrices indicate a fairly good fit of the model to the observed data for heavy and moderate congestion, although there is some scatter

of the data for uncongested conditions. The chi-square test, however, showed the model to be statistically acceptable at all 3 levels of congestion at the 0.05 confidence level (6, 7, 8). Table 4 gives the results of the tests.

TABLE 4
RESULTS OF TEST FOR FIT OF MODEL AND OBSERVED DATA

Level of Congestion	Degrees of Freedom	Chi Square		Significant Difference
		Calculated	0.05	
Heavy	15	1.92	25.00	No
Moderate	10	0.70	18.31	No
None	10	2.29	18.31	No

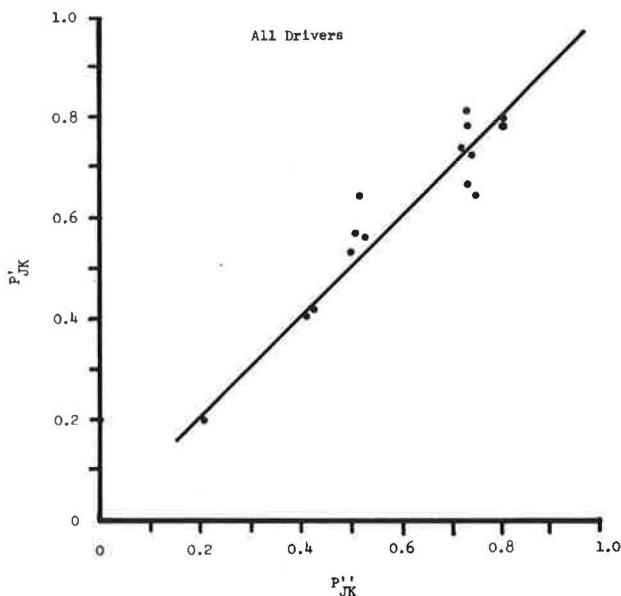


Figure 7. Calculated versus observed proportions of preference of descriptor J over K for no congestion.

ANALYSIS OF DATA BY SUBGROUPS

For each level of congestion, the sample was divided into the subgroups based on the respondents who (a) are male drivers, (b) are female drivers,

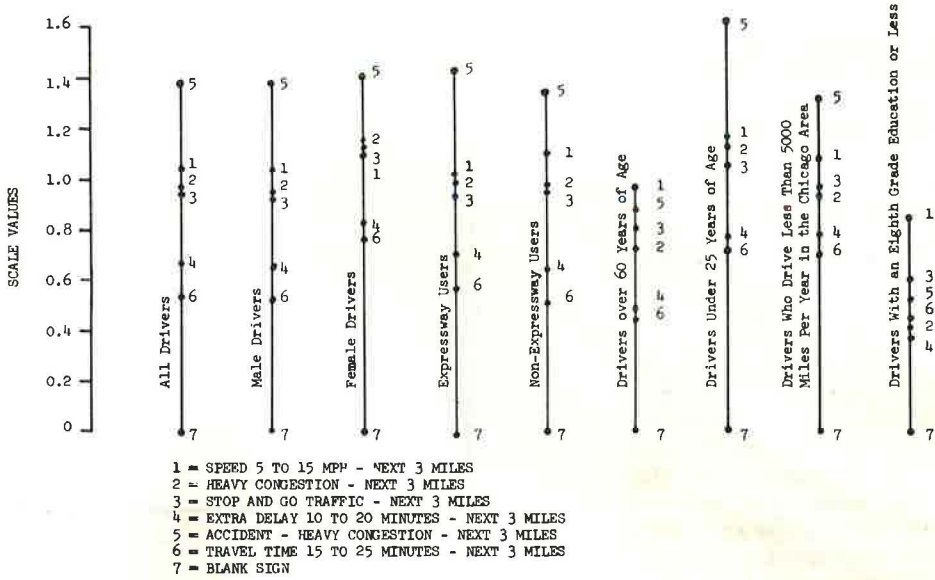


Figure 8. Scaling of descriptors for subgroups for heavy congestion.

(c) use expressway for journey to work, (d) do not use expressway for journey to work, (e) have an eighth grade education or less, (f) drive less than 5,000 miles per year in the Chicago area, (g) are under 25 years of age, and (h) are over 60 years of age. Scale values for the traffic descriptors for each level of congestion were determined for each subgroup and then compared to those of the total sample. A chi-square test was made for significant differences between each subgroup and the total sample. The

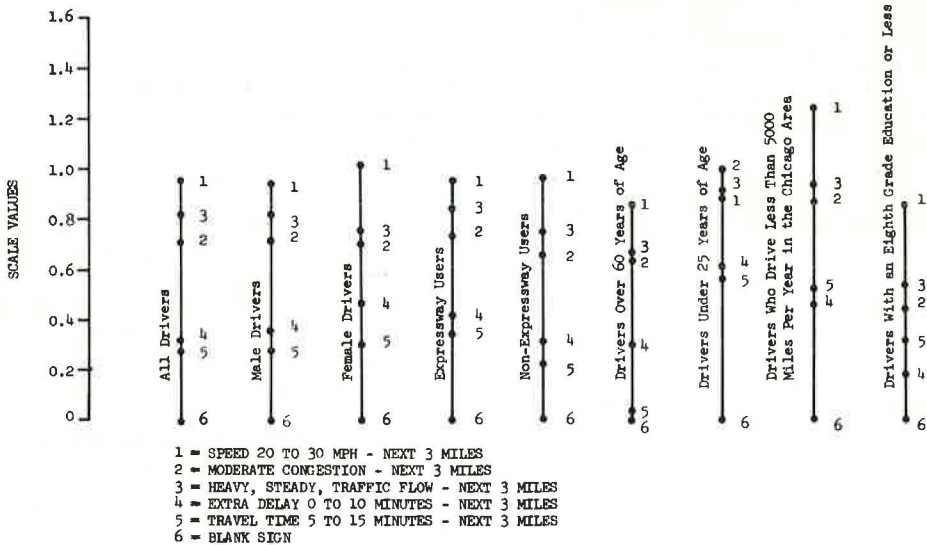


Figure 9. Scaling of descriptors for subgroups for moderate congestion.

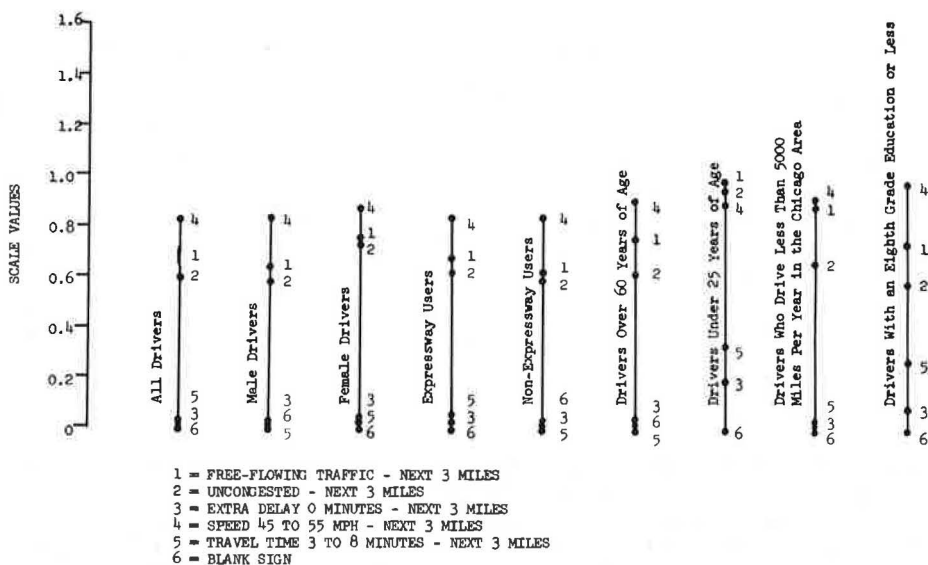


Figure 10. Scaling of descriptors for subgroups for no congestion.

scale values for the total sample and each of the subgroups for heavy, moderate, and uncongested conditions are shown in Figures 8, 9, and 10 respectively.

For the level of heavy congestion, all but 2 of the subgroups gave the highest scale values to descriptor 5 (accident—heavy congestion—next 3 miles). These 2 subgroups included respondents over 60 years of age and those with an eighth grade education or less who selected the speed descriptor as their first choice. Respondents with a low educational level selected descriptor 5 as their third choice. Respondents under 25 years of age gave this descriptor somewhat higher scale value than the other groups did. Generally, for the level of heavy congestion, the most preferred descriptor (the one with the highest scale value) was one that furnished information about an accident. Speed, a quantitative term, was usually the second choice. Speed, heavy congestion, and stop-and-go traffic were very close together, and at times, their scale values were interchanged for some of the subgroups. Two of the quantitative terms, travel time and extra delay, were of minor importance. For all of the subgroups, the blank sign received the lowest scale value of all the descriptors. The respondents, regardless of grouping, preferred information at the level of heavy congestion and were fairly consistent in their particular preferences.

For the level of moderate congestion, all but one of the subgroups gave the highest scale value to the speed descriptor. Those respondents under 25 years of age ranked speed third and moderate congestion first. Generally, for the level of moderate congestion, the descriptive terms—heavy, steady, traffic flow in descriptor 3 and moderate congestion in descriptor 2—were scaled lower than speed. The quantitative terms, extra delay and travel time in descriptors 4 and 5, ranked low in the order of preferences, and the blank sign, descriptor 6, received the lowest scale value of all the descriptors.

For the level of no congestion, all but one of the subgroups preferred the descriptor speed over all other descriptors. Again, those respondents under 25 years of age ranked speed third and free-flowing traffic first. The rest of the subgroups scaled the 2 descriptive terms, free flowing traffic in descriptor 1 and uncongested in descriptor 2, lower than speed. All subgroups valued the quantitative terms, extra delay and travel time, and the blank sign quite low.

TABLE 5
SIGNIFICANT DIFFERENCES BETWEEN SUBGROUP AND TOTAL GROUP FOR HEAVY CONGESTION

Subgroup	Number in Subgroup	Degrees of Freedom	Chi Square		Significant Difference
			Calculated	0.05	
1. Are male	638	15	0.04	25.00	No
2. Are female	68	15	3.33	25.00	No
3. Use expressway	402	15	0.50	25.00	No
4. Do not use expressway	304	15	0.80	25.00	No
5. Have an eighth grade education or less	60	15	36.38	25.00	Yes
6. Drive less than 5,000 miles per year in Chicago area	71	15	3.89	25.00	No
7. Are under 25 years of age	30	15	4.95	25.00	No
8. Are over 60 years of age	55	15	9.74	25.00	No

Note: There were 706 in total group.

A chi-square test was used to ascertain any significant differences in the preference for descriptors between each subgroup and the total sample. There are some obvious limitations when the chi-square test is used in this manner. Basically, each subgroup is a subset of the total group and, depending on the individual subgroup size, will reflect the characteristics of the total sample. As the subgroup becomes large relative to the total group, these characteristics will become more dominant, and the calculated chi square value will decrease. This can result in not rejecting the null hypothesis that there is no significant difference between the subgroup and the total

TABLE 6
SIGNIFICANT DIFFERENCES BETWEEN SUBGROUP AND TOTAL GROUP FOR MODERATE CONGESTION

Subgroup	Number in Subgroup	Degrees of Freedom	Chi Square		Significant Difference
			Calculated	0.05	
1. Are male	638	10	1.43	18.31	No
2. Are female	68	10	1.24	18.31	No
3. Use expressway	402	10	0.22	18.31	No
4. Do not use expressway	304	10	2.79	18.31	No
5. Have an eighth grade education or less	60	10	3.85	18.31	No
6. Drive less than 5,000 miles per year in Chicago area	71	10	2.40	18.31	No
7. Are under 25 years of age	30	10	5.99	18.31	No
8. Are over 60 years of age	55	10	0.89	18.31	No

Note: There were 706 in total group.

TABLE 7
SIGNIFICANT DIFFERENCES BETWEEN SUBGROUP AND TOTAL GROUP FOR NO CONGESTION

Subgroup	Number in Subgroup	Degrees of Freedom	Chi Square		Significant Difference
			Calculated	0.05	
1. Are male	638	10	0.03	18.31	No
2. Are female	68	10	2.38	18.31	No
3. Use expressway	402	10	0.12	18.31	No
4. Do not use expressway	304	10	0.17	18.31	No
5. Have an eighth grade education or less	60	10	3.82	18.31	No
6. Drive less than 5,000 miles per year in Chicago area	71	10	2.00	18.31	No
7. Are under 25 years of age	30	10	9.55	18.31	No
8. Are over 60 years of age	55	10	1.43	18.31	No

Note: There were 706 in total group.

sample when at times perhaps it should be rejected. Therefore, the chi-square test results given in Tables 5, 6, and 7 should be used merely as indicators and not as rigid tests for significant differences.

For all 3 levels of congestion, only one subgroup appeared to differ significantly from the total sample. This was the respondents with an eighth grade education or less for level of heavy congestion.

In addition to the chi-square results given in Tables 5, 6, and 7, a plot was made of the P_{JK} of the total group and the P_{JK} of each subgroup. As expected, the greater the departure from linearity, the higher was the value of the calculated chi square value. Consequently, these 2 procedures complemented one another.

SUMMARY

For all levels of congestion, traffic information was preferred to no information about traffic conditions. For the level of heavy congestion, information about an accident that had occurred and was causing heavy congestion was the most preferred descriptor of the total sample. The speed descriptor ranked second to the accident descriptor for the level of heavy congestion and first for the other two levels of congestion. The 2 descriptive terms (excluding the accident descriptor) were scaled fairly high, but were less desirable than the speed term. The 2 quantitative terms, delay and travel time, had relatively low scale values and were simply not desired by the respondents.

Only a limited number of traffic information descriptors were evaluated. The method of paired comparisons can be used to extend this evaluation to any number of descriptors. Although this research was applied only to visual formats, it can be extended to audio descriptors. A desirable information format, whether audio or visual, would be one that is satisfactory to a wide range of drivers and yet will not discriminate against any particular subgroup. The capability of furnishing the desired information becomes a function of design.

ACKNOWLEDGMENTS

The U.S. Bureau of Public Roads and the Illinois Division of Highways provided funds for the data acquisition and financial support. The Department of Civil Engineering of Northwestern University provided the computer time used in this analysis. The School of Civil Engineering of Purdue University provided services for typing and reproducing the manuscript.

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Discussion

DONALD E. CLEVELAND, Department of Civil Engineering, University of Michigan—Various techniques can be used to serve a fixed set of traffic demands. Until recently, little attention has been given to the use of traffic information and control methods to achieve a redistribution of flows from over-utilized freeways to under-utilized major arterial streets. Under certain circumstances and as a result of this redistribution, most travel can be accomplished faster and more smoothly. The authors have contributed a valuable study that will aid in the evolution of this method and design of operating systems.

They believe that traffic redistribution should and can be accomplished by providing current information on flow conditions on alternate routes and that the message displayed is important in providing this information. They have conducted an experiment in which as many as 6 pictorial displays generally purporting to contain the same informational content were shown to 706 subjects, all of whom were Chicago area automobile owners on a pseudo-freeway. The subjects were shown displays for 3 widely varying levels of service.

Their findings show that the subjects strongly desired any information on flow conditions ahead. This desire increased as traffic conditions became more congested. The information regarding average speed was preferred generally except that under heavy traffic conditions information that an accident had occurred ahead was considered to be of more value than speed information. Generally, stratification of the subjects by sex, expressway usage habits, age, educational level, and annual local mileage showed no significant differences in these findings.

The study appears to have been well executed; the analysis techniques are satisfactory and widely used; the sample is adequate; the findings appear to be integrable with operational studies such as those we are making in Detroit in which several types of displays on alternate routes are being used. There does not seem to be any reason why similar results would not be attained elsewhere.

Their findings that information is preferred to no information and that speed information is the most widely appreciated for describing conditions ahead are not surprising. In some sense, however, the preference for accident information to specific speed information under heavy traffic-flow conditions is puzzling. There appears to be an inconsistency in drivers' preferring the vague information that there is an accident ahead to the specific speed that can be anticipated. It would be reassuring to have the authors restate the specific instructions and format of this question. The motorists may possibly have interpreted the accident data as not describing the same set of operating conditions conveyed by the speed display. In fact, a number of minor inconsistencies in the quantitative data used in the study support such a request.

It would be appreciated if the authors would provide the benefit of their insight gained from this and other studies by responding to the following questions:

1. What are the relative improvements that can be expected from this type of demand redistribution in a cost-effectiveness sense as compared to other techniques available to engineers?
2. Why should voluntary choice instead of mandatory control be used in achieving this type of demand redistribution?
3. Demand redistribution is not achieved solely by conveying a message. Is it possible that the driver's decision-making process is so complex that different displays of the type used in this study may be optional under certain environmental states?

PAUL FOWLER, Automobile Club of Southern California—The American motorist clearly needs and wants more meaningful information concerning the driving task confronting him. This conclusion presented by the authors from their questionnaire sur-

vey of Illinois motorists has also been corroborated by previous surveys throughout the country concerning motorists' reactions to highway signing and radio traffic broadcasts. Although some experimentation is being done with various types of changeable-message signs, this study is especially significant because it attempts to determine what the motorist himself believes to be the most meaningful type of information. Previous research invariably attempted to measure only driver response to arbitrarily selected specific messages. The authors have identified driver preferences for visual real-time information within a broader range of alternative types of displays. Within this limited range of possibilities, the authors have tested the study results for significance and presented their conclusions in clearly understandable terms.

First in importance, the motorist wants advance information of unusual conditions such as accidents or other unpredictable disruptions ahead along his intended route. Second, he desires quantitative information in some meaningful form concerning the relative speed or travel time he may expect.

There is room for serious question that the expressed preferences for visual displays would, in fact, apply equally to audio broadcast messages as the authors assert. To be effective, visual descriptors must be concise and limited in number. On the other hand, audio messages permit greater detail, explanation, and even emphasis. Air Force research has led to adoption of pre-recorded audio messages to alert pilots to specific emergencies in preference to certain visual displays. Recent polls of radio listenership in the Los Angeles area seem to reflect significant shifts to one station that has adapted computerized techniques for broadcasting periodic estimates of travel times on specific segments of freeways during the commuter rush hours. Although these travel times are computed from historical data and are adjusted according to reported accidents or stoppages without any real-time input, the program ratings reflect the motorist's belief that travel time in minutes is meaningful information that is either helpful or reassuring to him. This indication of subjective popularity suggests that quantitative information in speed or minutes is useful, particularly if it is presented in detail or adequately explained. Certainly the possibilities inherent in audio broadcasts must be considered separately on their own merits and investigated in further research.

The authors presented to each of the motorists a visual display message in a sample photograph of a particular type of changeable-message sign rather than a simple type-written or printed descriptor. This departure from pure research may be unfortunate in that it appears to inject the possibility of a driver bias that results from personal reactions to the particular type of sign but that might be more applicable to certain types of descriptors. Previous studies of driver response to similar changeable-message signs in the Detroit and New York areas raise some question that the readability or impact of that particular hardware could affect the stated preferences of the respondents tested. If specific limitations in the hardware could in fact affect the motorist's selections, then further studies may be necessary involving more legible messages, alternative types of hardware, or simple, printed displays.

As a final point of discussion, there is a need for further verification of driver information preferences in other metropolitan areas. For instance, the driver's needs and responses may well reflect the relative sophistication that he has attained. Conceivably this would vary throughout the country depending on the availability and complexity of the freeway network, the availability of other travel modes, and the average commuter trip-length. The motorist's needs and ability to respond in a low-density, automobile-oriented community may well be quite different from those in a high-density, geographically constrained metropolitan area.

This research effort into the subject of visual information displays is timely and represents an extremely important step in the development of improved driver information systems. It could represent the first phase of a three-phase project. The second phase would logically involve simulation tests to measure actual driver response to specific types of messages, as distinguished from this sampling of driver preferences. The final stage must necessarily involve full-scale field testing of prototype hardware, including real-time data collection systems. The need for operational improvements to obtain optimum efficiency from urban freeways, particularly during

periods of peak traffic demand, is readily apparent if we are to keep pace with shifting urban population. The authors are to be commended for their attack on this crucial problem and for their forthright presentation of the study results.

JOSEPH A. WATTLEWORTH, University of Florida, Gainesville—In this questionnaire study of the preferences of drivers for alternate visual displays, the significant findings were that (a) drivers preferred to have information about freeway traffic conditions; (b) displays presenting speed information or descriptive terms were preferred to displays presenting quantitative travel time or delay information; and (c) for heavy congestion, the drivers preferred the addition of an accident descriptor in the display.

In my view, the most significant result is that displays that present speed information or descriptive terms were preferable to displays that present quantitative travel time or delay information. This would indicate that motorists do not evaluate alternate routes in terms of travel time and delay, but rather that they tend to think in terms of speed or some other reference parameter—perhaps comfort and convenience. This is not necessarily to say that the motorists do not end up taking the minimum-travel-time route, but merely that they do not evaluate the alternate specifically in terms of travel time. Because most traffic assignment procedures are based on some minimum-travel-time principle, this point is quite significant and suggests that further investigation into the philosophy of traffic assignment is warranted.

This research finding is significant for another reason. For years many traffic experts have argued that drivers do, indeed, think in terms of minimum travel time when they are selecting from alternate routes. These experts have based many decisions on this premise. Research has shown that, in this case, expert professional opinion was wrong and one must wonder how many more similar examples one could find. Too many standards, policies, and guidelines are established by committee consensus or by a similar edict by knowledgeable people who are forced to act without thorough factual documentation of the wisdom of their course of action. Too often, then, these standards, policies, and guidelines are never questioned because of the professional esteem of the originators. These comments are intended not to question the professional abilities of anyone but to emphasize the value of research in providing more factual information on which to base or check traffic engineering decisions. Specifically, it is suggested that further research of this type be conducted to evaluate driver preferences for information in other signing situations.

Good research, in addition to answering the initial question or questions, should serve to raise other questions for further investigation, and this research has done this. Many of these questions are related to the systems aspects rather than to merely the display aspects of a freeway driver information system and are concerned with system operation, system performance, and driver reaction, all of which are beyond the scope of the present investigation. Several of these questions are discussed in the following paragraphs.

Will a significant number of drivers who are told of heavy freeway congestion actually divert to alternate routes? Numerous studies in Houston (9) and elsewhere have indicated that many motorists actually increase their trip time and distance in order to use freeways for parts of their trips. If the freeway is this attractive to the motorists, they may be reluctant to divert from it.

Is it sufficient to present information only regarding freeway conditions, or must information also be presented on alternate route conditions? The descriptions of the Freeway Driver Information System in the paper suggested that only freeway information would be presented. This still leaves the driver in somewhat of a quandary regarding conditions on the alternate route unless the operation of the alternate route is unusually consistent and insensitive to changes in demand caused by the diversion. The driver is interested, it would appear, in information on which he can base a selection of an alternate route, and merely having knowledge of freeway conditions may not be

adequate in most instances. If an attempt is made to provide information on the best alternate route to a point, system stability must be considered. In many freeway corridors in which the demand is fairly close to capacity, shifts in some demand may cause congestion on the best route, thereby ending its status as best route. The information system may then oscillate between the alternate routes in designating them as best routes.

Should the accident descriptor be used to emphasize the heavy-congestion message only when there actually is an accident? It would appear that indicating an accident when there is none would adversely affect the drivers' confidence in the system. The use of the accident descriptor should probably be questioned from another point of view. Although the drivers indicated a preference for this additional information, it is not clear how it would be used. Specifically, it is not clear whether this additional information would encourage diversion from the freeway or whether it would tend to encourage drivers to use the freeway to be able to see the accident.

In summary, the paper was quite good and presented some very important findings regarding the display portion of the Freeway Driver Information System. Several other questions relating to system operation, system performance, and driver reactions to such a system have been left for subsequent investigations.

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KENNETH W. HEATHINGTON, RICHARD D. WORRALL, and GERALD C. HOFF, Closure—The authors wish to express their appreciation to the 3 discussers for the manner in which they conducted their reviews. Basically, we believe all of the questions raised are very pertinent to the research at hand. Some of these questions, perhaps, we can answer. Others are simply not covered in our research. Because this presentation represents only a small part of the overall project performed at the Expressway Surveillance Project, it does not address some of the questions that have come forth. These would tend to be explained by an exploration of other parts of the research. In a brief manner, however, we will attempt to make some comments that should help in clarifying some of the points raised.

Our statement that the results of this research might be applicable to audio systems was to imply that the information areas of interest should be compatible. That is, if information on speed is highly preferred in a visual display, perhaps this same type of information would be highly preferred in an audio broadcast. Furthermore, we do not feel that the signs biased the answers of the respondents in any manner. The method of paired comparisons forced the respondent to choose between 2 signs that differed only by their messages. Thus the respondent really had a choice only between messages.

We believe (and this is based partly on conversations with a few of the respondents) that drivers in the Chicago area are deeply concerned about being caught in a traffic tie-up caused by an accident. The accident descriptor probably conveys to the respondents a mixture of conditions such as inconvenience, discomfort, slow speeds, stalled traffic, and long delays. All of these, of course, the driver would like to avoid. Although each respondent was specifically instructed that the paired signs were intended to convey exactly the same conditions (with the exception of a blank sign), it is possible that some respondents may have mentally interpreted them differently.

The relative improvements that can be expected from a demand redistribution have not been evaluated in a cost-effectiveness framework. Preliminary examination, however, indicates that the cost of information systems may be substantially less than that of other techniques appropriate for redistributing the demand. The important point to

recognize is the ability of a particular information system to significantly influence the redistribution of demand. We believe that an information system can be effective but, of course, our research has not progressed sufficiently to substantiate this belief. We certainly subscribe to a voluntary rather than a forced redistribution of demand for the more obvious reasons of cost, capabilities, practicality, and user acceptance. One can, of course, present further arguments for a voluntary redistribution of demand.

We would prefer a very elaborate information system operating in real time. Conceptually, this information system should cover all modes and all facilities so that conditions are known with some degree of certainty at any time a query is made. We realize, however, that it is difficult to begin with such an elaborate system. For this reason, the overall research was directed toward a system that is within our present capabilities of resources and technology.

Feasibility of an Exclusive Lane for Buses on the San Francisco-Oakland Bay Bridge

DARRYL B. MARTIN, California Division of Highways

The study was initiated by a request from the U. S. Bureau of Public Roads for an evaluation of the potential for reserving one or more exclusive bus lanes on the San Francisco-Oakland Bay Bridge. This section of highway has the greatest peak-hour bus concentration in California, and delay occurs because demand exceeds capacity during peak periods. The existing traffic conditions were surveyed for both morning and evening peak periods. The data obtained included capacity of bridge, number of persons using each mode, volumes (automobiles and buses), travel times, and demand. This information was used to determine the present person-delay being suffered. These data were then used in a simulation of conditions with an exclusive bus lane in effect. The assumption was made that the lane used exclusively for buses was previously a lane used for mixed traffic in the same direction. A graph was developed that showed person-delay with and without an exclusive bus lane as a function of modal split. This paper concludes that an exclusive bus lane on the Bay Bridge is not feasible because the increased delay to automobile users would far exceed the savings to the bus passengers. At the present demand there is no modal split that would make an exclusive bus lane feasible. The conclusions were based principally on recurrent congestion. If nonrecurrent congestion such as that caused from breakdowns and accidents had been included in the delay, the exclusive bus lane alternative would have been even more detrimental.

●THIS STUDY was initiated when the Bureau of Public Roads requested that an evaluation be made of the potential for reserving one or more exclusive bus lanes on the San Francisco-Oakland Bay Bridge (SFOBB). The position of the Federal Highway Administration on the reservation of freeway lanes for buses is stated in the Bureau's Instructional Memorandum 21-13-67. The SFOBB is a 10-lane facility with 5 westbound lanes on the upper deck and 5 eastbound lanes on the lower deck. The bridge has no shoulders, and when accidents or stalls occur the capacity of the bridge is reduced by at least 1 lane. The area in which the bridge is located is shown in Figure 1.

Data for this report were obtained from several sources. The capacity of the bridge was determined from mainline counts taken near the tunnel on Yerba Buena Island by the Division of Bay Toll Crossings and was verified by counts and aerial photographs of queuing at the east end of the bridge. Counts of volumes on the ramps feeding the bridge were obtained from the San Francisco office of the Division of Highways. Bridge data, including a very comprehensive bus travel-time study during peak periods, were also furnished by the Division of Bay Toll Crossings. Other data, including bus counts and occupancy, were obtained from semi-annual traffic surveys conducted by the University of California.

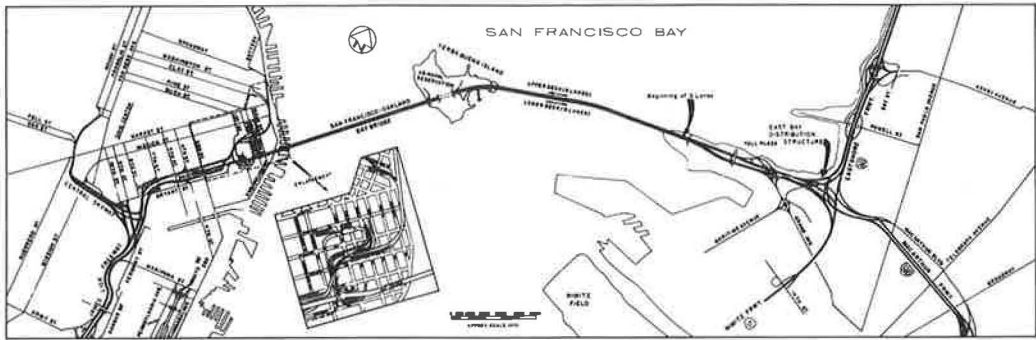


Figure 1. Location of the San Francisco-Oakland Bay Bridge.

PRESENT CONDITIONS

The present volume and composition of traffic during the peak congested periods are given in Table 1. The present average off-peak travel time for buses in the westbound direction from the Bay Bridge Toll Plaza to the San Francisco Terminal Building is 8.25 min and the travel time in the eastbound direction from terminal building to toll plaza is 8.75 min. Details on travel time and delay for buses are given in the Appendix. Eastbound, off-peak travel takes 30 sec longer than the westbound, off-peak travel because the buses are starting from almost a complete stop on an approximately 3 percent grade.

During the peak period under ideal conditions (that is, good weather, daylight, and no lane blockage), the average travel time for buses in the westbound direction was approximately 11 min and in the eastbound direction, 11.3 min.

The maximum delay to buses leaving the toll plaza in the westbound direction occurred between 7:35 and 7:45 a. m. This is the period when queuing from the merge section is greatest. The maximum delay obtained from bus travel times on 9 different weekdays amounted to 4.75 min. Most of this delay is incurred in the merge section between the bridge and the toll plaza.

The delay for buses in the eastbound direction is fairly constant throughout the latter part of the peak period (5:00 to 6:00 p. m.). This is because the majority of the delay being experienced is running delay on the bridge. (Running delay is the delay caused by the natural reduction in speed as flow rates increase, and queuing delay is that caused by demand exceeding capacity.) Variability is observed only when accidents or stalls

TABLE 1
VOLUME AND COMPOSITION OF TRAFFIC ON SAN FRANCISCO-OAKLAND BAY BRIDGE

Time	Vehicles			People				
	Buses	Automobiles ^a	Total	In Buses		In Automobiles		Total
				Number	Percent	Number	Percent	
Peak hour ^b 7:10 to 8:10 a. m.	340	8,461	8,801	11,996	48.7	12,607	51.3	24,603
Peak period 8:55 to 8:25 a. m.	433	12,515	12,948	14,797	44.2	18,647	55.8	33,444
Peak hour ^b 4:35 to 5:35 p. m.	303	8,503	8,806	11,858	48.5	12,669	51.5	24,527
Peak period 4:10 to 5:45 p. m.	374	13,102	13,476	14,073	41.9	19,522	58.1	33,595

^aIncludes trucks.

^bHour of maximum flow of people across the bridge and not necessarily hour of maximum flow of vehicles.

occur on the bridge. On one of the data collection days 2 separate lane blocks in the eastbound direction lasted a total of 27 min and caused maximum delays of 8.5 min per bus. Although nonrecurrent congestion occurs frequently and must be anticipated, it is not included in this analysis because of its extreme day-to-day variability.

At the present time the buses are suffering less delay than the automobiles during the peak congested periods. This is true in both directions. In the eastbound direction the buses presently have an exclusive entrance to the bridge, and the delay they incur in getting onto the bridge is negligible. The automobiles, on the other hand, queue up on every ramp leading to the bridge.

The westbound direction generally has a continuous flow except at the merging section near the beginning of the bridge. Six lanes feed into the toll plaza area, but only 5 lanes are on the bridge (Fig. 2). Because the capacity on the bridge is less than the capacity at the toll booths, queuing occurs between the bridge and the toll booths. Even though it is not mandatory, the buses predominantly use the right lanes in the 17-lane toll booth area because the drivers have found from experience that operation is smoother in the right lanes where the traffic volumes are lighter. The queuing delay being experienced between the toll booths and the bridge, a distance of 3,750 ft, is less for the buses than for the automobiles. The average queuing delay per automobile during the morning peak on October 9, 1968, was 2.3 min, whereas the average queuing delay per bus was only 1.5 min. The maximum queuing delay for automobiles was 4.3 min and for buses only 2.75 min.

The average delay on the bridge (running delay) for automobiles is approximately the same as that for buses during the peak periods. The running delay is the difference between low-volume speeds (43 mph for buses and 45 to 50 mph for automobiles) and capacity-volume speeds (37 mph).

Because of the difficulty in obtaining data on the many San Francisco approaches, the queuing delay for automobiles was determined only for the westbound direction. The total queuing delay is much greater in the eastbound direction.

CONDITIONS WITH AN EXCLUSIVE BUS LANE

Only the westbound direction is considered in the analysis of the reservation of 1 of the existing 5 lanes for exclusive use by buses, leaving only 4 lanes available for vehicles other than buses. Details of the analysis are given in the Appendix; the results are discussed in the following.

If one lane is used exclusively by buses, the current travel demand and the number of people using each of the 2 modes will increase the total queuing delay in the westbound direction from 65,600 to 289,000 person-minutes even though the bus passengers suffer no delay. The total running delay will decrease from 36,900 to 36,800 person-minutes, and the total delay will increase from 102,500 to 325,800 person-minutes. The maximum queuing delay per vehicle will increase from 4.3 to 16 minutes, and the maximum number of vehicles in queue will increase from 620 to 1,950. A queue of this magnitude will cause congestion in the East Bay Distribution Structure, which is the interchange among I-80, I-580, and Cal-17 (Fig. 3), and will adversely affect 3 major freeways and many motorists on these freeways who are destined for locations other than San Fran-



Figure 2. San Francisco-Oakland Bay Bridge Toll Plaza (looking west).



Figure 3. San Francisco-Oakland Bay Bridge Toll Plaza (looking east).

cisco. Because of the excessive queuing that will occur in the distribution structure, the buses themselves will incur additional delay upstream from the toll plaza exceeding the savings that they gained by having an exclusive bus lane on the bridge. Congestion will be present for approximately 3 hours from 6:30 to 9:30 a. m.

Although not calculated, the delay caused by an exclusive bus lane in the eastbound direction will be much more severe because of lack of storage space for the queued vehicles. This in turn will create delays for large numbers of persons not destined for the bridge. Congestion will extend well beyond the Central Skyway connection to the James Lick Freeway. The bus traffic will save very little (1 to 2 min on the bridge) because even under the present conditions the buses are experiencing only 30 sec queuing delay.

In the event of accidents or stalls, the exclusive bus lane will be quite advantageous to the buses, unless the lane blockage occurs in the exclusive bus lane, and the savings to bus passengers may seem quite large when compared with the delay

suffered by motorists. The result, however, would be to greatly increase total person-delay above the calculated figures that are based on incident-free operation.

EFFECT OF CHANGE IN MODAL SPLIT

Figure 4 shows person-minutes of delay for various percentages of people riding buses with and without an exclusive bus lane and the current demand that stays constant regardless of the modal split. The present peak-hour demand is 25,200 persons. The figure is based on the following stipulations:

1. Five lanes are available, either 5 mixed or 4 for automobiles and trucks and 1 for buses.
2. Delay is incurred when queuing is present.
3. Automobile occupancy is 1.49 persons.
4. One bus is equivalent to 2 automobiles in mixed traffic stream.
5. Capacity of bridge with 5 lanes available for mixed traffic is 8,800 vehicles per hour.
6. Capacity of bridge for other vehicles with only 4 lanes available is 7,280 per hour.

The current modal split during the peak period is 44 percent for buses and 54 percent for automobiles; buses are much lower both before and after this period. Composition of demand for other modal splits, which caused the peak period to vary, was determined by allocating the changes made in proportion to the present demand.

As Figure 4 shows, if an exclusive bus lane is in effect, the delay will not be eliminated until nearly 57 percent of the people ride buses. However, if 57 percent rode buses and mixed traffic were allowed on all 5 lanes, there would be no delay for anybody and there would be considerably more freedom of movement in the traffic stream. In other words, the bridge would be operating at approximately 82 percent of capacity with mixed traffic in all 5 lanes, but at 100 percent of capacity with 1 lane for buses and 4 lanes for automobiles and trucks. It can be concluded that at the present demand no modal split would make an exclusive bus lane feasible.

Based on Current (1968) arrival rates (demand) from 6:00 to 11:00 a.m.,
 including { 6:55 to 8:25 a.m. : 33,444 persons (peak period under present conditions)
 7:00 to 8:00 a.m. : 25,200 persons (peak hour)

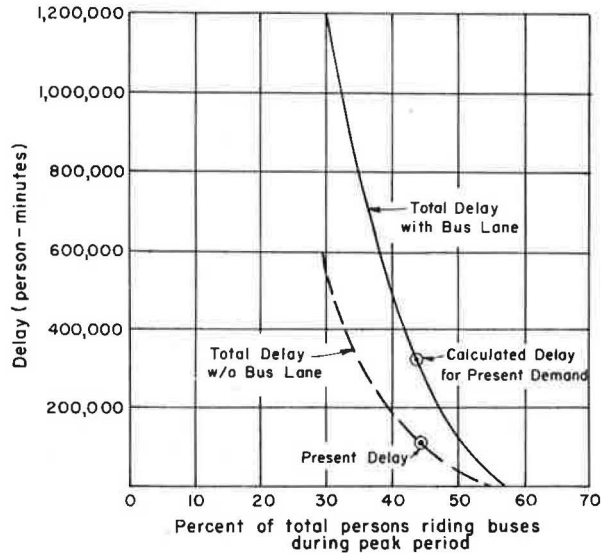


Figure 4. Delay during peak period westbound with and without an exclusive bus lane as a function of modal split.

BUS BYPASS AT BRIDGE APPROACH

Currently buses traveling eastbound are bypassing the major congested area, the approaches to the bridge. They are able to do this because they have an exclusive bus ramp to the bridge.

A study was made of the possibility of allowing buses to bypass the congestion westbound between the toll booths and the bridge. This could easily be done by reserving the far right toll booth exclusively for buses and striping an exclusive bus lane from this booth to near the beginning of the bridge where the buses would then merge with other traffic in the right lane. This would bring the buses in at the head of the line. The only delay the buses would incur would be the nominal running delay crossing the bridge.

The merits of this plan were determined by a detailed bus travel-time study to determine how long the buses are being delayed in this merging section between the toll booths and the bridge. It was discovered that the average delay to buses during the peak period is only 1.5 min compared to 2.3 min for all other vehicles. The bus drivers have discovered the advantage of using the right lanes even though they are not mandatory. Automobiles avoid the right lanes because of the high bus density. Therefore, the buses have an unofficial bypass. The major queuing delay being suffered is near the bridge where the buses merge into the mainstream of traffic. This problem would be present even if an exclusive bus bypass were in effect.

The bypass lane could not extend onto the bridge for this would mean eliminating a lane for use by other traffic. This would have the same effect on queuing delay as an exclusive bus lane across the entire bridge. At the present time an exclusive bus bypass between the toll booths and the bridge will result in a 1- to 1.5-min savings per bus. Some additional delay will be suffered by other vehicles at the merging section.

The savings to buses become quite insignificant when considered as part of the total trip time from point of origin to point of destination.

The eastbound approach is currently not a problem for the buses. Observations in this area revealed 3 detriments to vehicle traffic flow:

1. The exclusive bus entrance has an inadequate merging lane and is controlled by a yield sign;
2. The buses are starting from almost a complete stop on an approximately 3 per cent grade; and
3. The buses begin changing lanes at slow speeds as soon as they enter the bridge.

In addition to helping buses, improvements in the merging geometrics so that the buses can begin the grade at a higher speed will improve the flow of traffic. After entering the bridge, the buses should stay in the same lane until reaching the summit of the west bay crossing.

NONRECURRENT CONGESTION

This analysis is based principally on recurrent congestion, congestion caused from inadequate capacity. Nonrecurrent congestion, such as that caused from breakdowns and accidents, has not been discussed.

Traffic surveys by the University of California of stalls and accidents on the bridge during peak periods on October 18 and 19, 1967, and April 16 and 17, 1968, show that an average of $1\frac{1}{3}$ incidents occur daily in the westbound direction and last for an average of 19 min each. In the eastbound direction, approximately 5 incidents occur daily and last for an average of 12 min each. If nonrecurrent congestion had been included in the analysis, the exclusive bus lane alternative would have been even more detrimental.

CONCLUSION

An exclusive bus lane on the San Francisco-Oakland Bay Bridge is not feasible. The average delay currently on the bridge is only 1 to 2 min over a 5-mile section. The delay incurred in the merging section downstream from the toll plaza amounts to an average queuing delay of 1.5 min for buses and 2.3 min for other vehicles. An exclusive bus lane will increase delays to the automobiles, and the resulting loss will far exceed the savings to the bus traffic because available capacity in the bus lane will be wasted. The net increase in delay, if an exclusive bus lane is in effect, will be approximately 223,300 person-minutes daily during the westbound morning peak and a much larger amount during the eastbound evening peak.

Even though the exclusive bus lane will result in savings to buses while on the bridge, losses will be incurred upstream from the bridge. Other vehicles being delayed because of the reduced number of lanes for their use on the bridge will be backed up far upstream. The buses will suffer additional delay from this upstream congestion unless an exclusive bus lane extends back to the point where they enter the freeway. Because the buses now enter the freeway from many different ramps on 3 separate freeways, the provision of exclusive lanes from all of those origins is not feasible. None of this bus delay or delay to vehicles not destined for the bridge is included in the 223,300 person-minutes of increased delay per morning.

An exclusive bus bypass through the toll plaza area definitely has merit. Increased demands in the future may prove this to be a valuable alternative. At the present demand, no modal split would make an exclusive bus lane feasible.

These conclusions have been based principally on recurrent congestion. If nonrecurrent congestion, such as that caused from breakdowns and accidents, were included in the delay, the exclusive bus lane alternative would be even more detrimental.

ACKNOWLEDGMENTS

The author wishes to acknowledge the support given by the Division of Bay Toll Crossings, the Traffic Department of District 04 of the Division of Highways, and the Institute of Transportation and Traffic Engineering of the University of California. This research

project was undertaken in cooperation with the U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads. The opinions, findings, and conclusions expressed in this paper are those of the Division of Highways and not necessarily those of the Bureau of Public Roads.

Appendix

BUS TRAVEL TIME AND DELAY

Present Conditions

Tabulations of travel times for buses between the San Francisco Terminal Building and the Bay Bridge Toll Plaza were made by the Division of Bay Toll Crossings for both the eastbound and westbound directions. Table 2 gives information for conditions with no nonrecurrent congestion extracted from these tabulations.

Figure 5 shows the total travel time for buses between the toll booths and the terminal in the westbound direction on October 9, 1968, during the morning peak period. No nonrecurrent congestion existed, and the weather was clear; conditions were very close to perfect. In the eastbound direction, the travel time on the bridge during the peak period is approximately 2 min greater than that during the off-peak period.

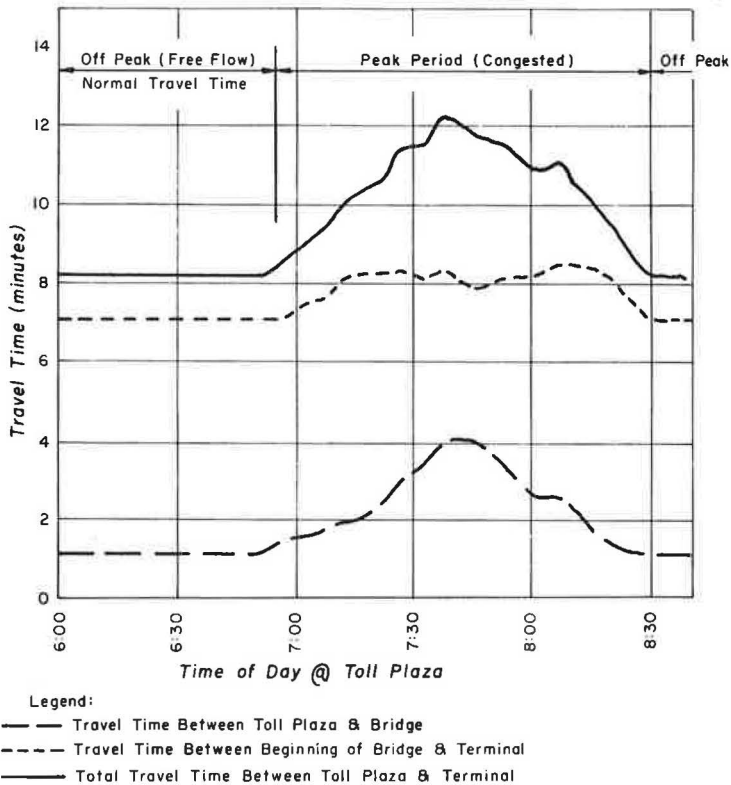


Figure 5. Bus travel times during peak period westbound from Toll Plaza to San Francisco Terminal.

TABLE 2
BUS TRAVEL TIMES IN MINUTES

Peak or Delay	Westbound	Eastbound
Average peak	10.85	11.30
Average off peak	8.25	8.75 ^a
Average delay	2.60	2.55
Maximum peak	13.00	13.00
Maximum delay	4.75	4.25

^aEastbound off peak takes 30 sec longer than westbound off peak because buses are starting from almost a complete stop on an approximately 3 percent grade.

Demand and Capacity Westbound

Flow through the toll booths in the westbound direction was used as the demand. Table 3 gives the demand on October 9, 1968, from the hours of 6:00 to 9:30 a. m. The capacity of the toll booths is greater than the capacity of the bridge; therefore, queuing occurs between the bridge and the toll booths. The capacity of the bridge was determined from mainline counts taken near the tunnel on Yerba Buena Island on 4 different days. These counts were taken in both directions far downstream from

TABLE 3
DEMAND OF MIXED TRAFFIC WESTBOUND ON
OCTOBER 9, 1968

Time (a. m.)	Vehicles	Cumulative Vehicles	15-Min Flow Rate
6:00		0	
6:15	837		3,348
6:30	1,308	837	5,232
6:45	1,871	2,145	7,484
7:00	1,993	4,016	7,972
7:15	2,435	6,009	9,740
7:30	2,455	8,444	9,820
7:45	2,350	10,899	9,400
8:00	1,943	13,249	7,772
8:15	2,048	15,192	8,192
8:30	1,748	17,240	6,992
8:45	1,599	18,988	6,396
9:00	1,466	20,587	5,864
9:15	1,350	22,053	5,400
9:30	1,250	23,403	5,000
		24,653	

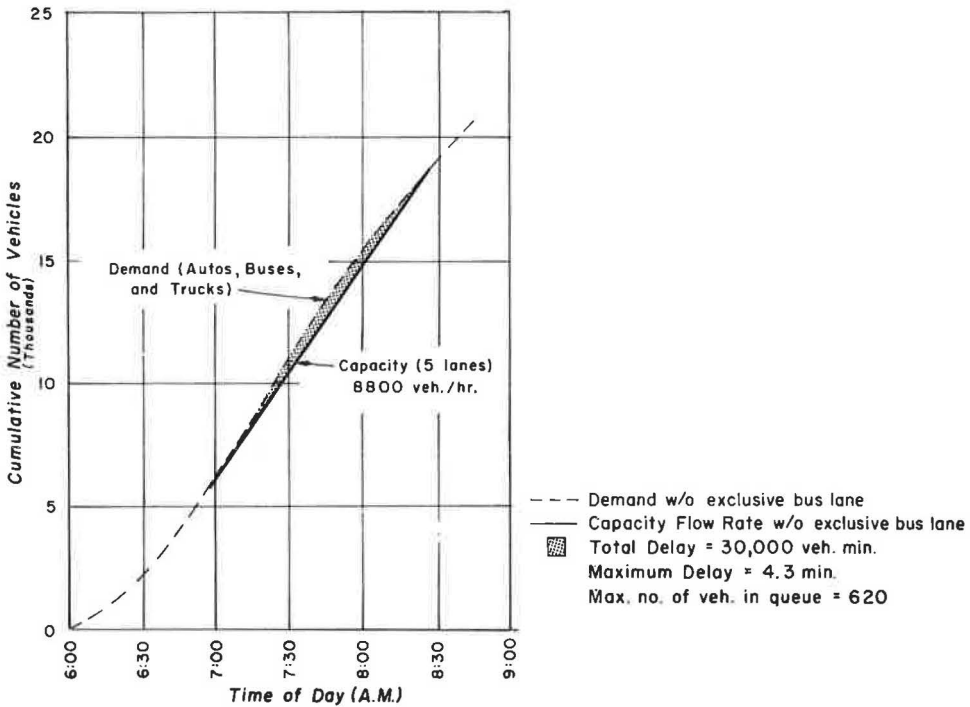


Figure 6. Relationship between demand and capacity during peak period on east approach of bridge without exclusive bus lane.

bottleneck sections. From these counts, the capacity for both directions was determined to be 8,800 vehicles per hour.

A reduced-scale representation of a cumulative plot of demand and capacity with respect to time is shown in Figure 6. The area within the demand and capacity curves is the total delay incurred in vehicle-minutes. The difference between the 2 curves on the vertical axis gives the number of vehicles in queue at any time. The difference between the 2 curves on the horizontal axis gives the delay per vehicle at any time.

The following information was obtained from Figure 6. In the bus queuing delay, 1.5 min is the average delay per bus during the morning peak period as determined from the bus travel-time survey.

Period of delay	6:55 to 8:25 a. m.
Total queuing delay, vehicle-minutes	30,000
Bus queuing delay (433 buses × 1.5 min), vehicle-minutes	650
Total automobile delay, vehicle-minutes	29,350
Maximum delay, min	4.3
Maximum number of vehicles in queue	620
Total number of automobiles delayed (13,200 - 433)	12,767
Mean automobile delay (29,350/12,767), min per automobile	2.3

TABLE 4
DELAY INCURRED BY BUSES DURING PEAK PERIOD WESTBOUND ON OCTOBER 9, 1968

Time (a. m.)	Number of Buses	Occupancy per Bus	Queuing Delay Toll Plaza to Bridge		Running Delay East End of Bridge to S. F. Terminal	
			Avg. Delay per Bus (min)	Total Delay (person-min)	Avg. Delay per Bus (min)	Total Delay (person-min)
6:55	14	32.2	0.25	117	0	0
7:00	11	42.4	0.25	117	0.50	234
7:05	18	42.4	0.25	191	0.50	382
7:10	25	42.4	0.50	531	1.25	1,330
7:15	21	42.4	0.75	667	1.25	1,110
7:20	30	42.4	1.25	1,590	1.25	1,590
7:25	23	42.4	1.75	1,705	1.25	1,219
7:30	28	35.3	2.00	1,975	1.25	1,235
7:35	38	35.3	2.75	3,680	1.25	1,675
7:40	39	35.3	2.75	3,780	1.00	1,375
7:45	26	35.3	2.25	2,065	0.75	687
7:50	29	35.3	2.00	2,050	1.00	1,025
7:55	26	35.3	1.75	1,610	1.25	1,150
8:00	34	22.3	1.25	760	1.25	760
8:05	21	22.3	1.25	588	1.50	705
8:10	25	22.3	0.75	420	1.50	840
8:15	13	22.3	0.25	73	1.25	364
8:20	<u>12</u>	22.3	0	<u>0</u>	1.00	<u>269</u>
8:25						
Total	433		—	21,919	—	15,950
Avg.	—		1.51	—	1.10	—

According to the University of California traffic survey, the average vehicle occupancy is 1.49 persons between the hours of 6:55 and 8:25 a. m. Therefore, queuing delays in person-minutes are

Automobile queuing delay (29,350 × 1.49)	43,700
Bus queuing delay (from Table 4)	21,900
Total queuing delay	65,600

The running delay incurred on the bridge was caused by high volumes. This delay extended from 6:55 to 8:25 a. m., the approximate time period of the queuing delay.

The average running delay is 1.1 min per vehicle during the congested period as determined by the bus travel-time survey on October 9, 1968. It is assumed that during the peak periods, running delay per automobile is the same as the running delay per bus. Total running delays in person-minutes between 6:55 and 8:25 a. m. were determined as follows:

Total people in 433 buses (from Table 4)	14,797
Total people in automobiles (12,767 × 1.49)	19,023
Running delay for buses (from Table 4)	15,950
Running delay for automobiles (19,023 × 1.1 min)	20,925
Total running delay	36,875
Total queuing delay	65,600
Total delay	102,475

Conditions With an Exclusive Bus Lane

There were 300 buses crossing the bridge during the peak hour. For the purpose of these calculations, we have assumed that in the traffic stream a bus is equivalent to

TABLE 5
DEMAND OF TRAFFIC WITHOUT BUSES WESTBOUND
ON OCTOBER 9, 1968

Time (a. m.)	Total Vehicles	Minus Buses	Vehicles Without Buses	
			Number	Cumulative
6:00				0
	837	10	827	827
6:15	1,308	10	1,298	2,125
6:30	1,871	15	1,856	3,981
6:45	1,993	29	1,964	5,945
7:00	2,435	54	2,381	8,326
7:15	2,455	74	2,381	10,707
7:30	2,350	105	2,245	12,952
7:45	1,943	81	1,862	14,814
8:00	2,048	80	1,968	16,782
8:15	1,748	38	1,710	18,492
8:30	1,599	33	1,566	20,058
8:45	1,466	26	1,440	21,498
9:00	1,350	25	1,325	22,823
9:15	1,250	15	1,235	24,058
9:30				

2 automobiles. If 1 lane is used exclusively by buses, the capacity of the remaining 4 lanes is as follows:

5-lane hourly capacity with no buses (8,800 vehicles - 300 buses + 600 automobiles that replace buses)	9,100
4-lane hourly capacity ($\frac{4}{5} \times 9,100$)	7,280

Table 5 gives the demand at the toll plaza without buses. This demand and a constant rate of 7,280 other vehicles for capacity were used to determine demand and capacity with respect to time as shown in Figure 7, which is a reduced-scale representation of the original plot. The area within the curve shows the total delay incurred by automobiles and trucks. Because the buses will use the fifth lane and suffer no delay, they are not shown in this plot. From Figure 7, the following information was obtained:

Period of delay	6:30 to 9:35 a. m.
Total queuing delay, vehicle-minutes	194,400
Maximum delay, min	16
Maximum number of vehicles in queue	1,950
Total queuing delay ($194,400 \times 1.49$), person-minutes	289,000
Total running delay ($22,447 \times 1.49 \times 1.1$), person-minutes	36,790
Total delay, person-minutes	325,790

If an exclusive bus lane is in effect, the increased delay in crossing the bridge will be 223,300 person-minutes, computed as follows:

Delay with bus lane, person-minutes	325,800
Delay without bus lane, person-minutes	102,500
Increased delay, person-minutes	223,300

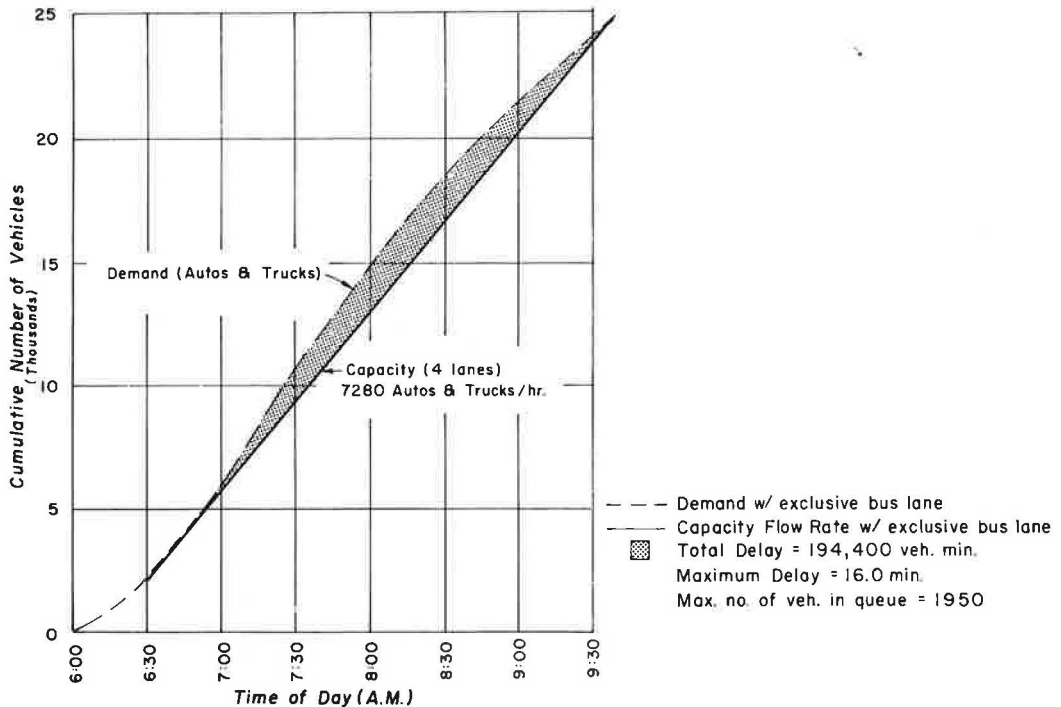


Figure 7. Relationship between demand and capacity during peak period on east approach of bridge with exclusive bus lane.

Discussion

FRANK S. KOPPELMAN, Tri-State Transportation Commission, New York—The establishment of exclusive bus lanes in heavy travel corridors has received considerable attention by researchers, planners, and administrators seeking ways to improve peak-hour access to central business districts. The study reported by Martin is useful in quantifying some of the effects of providing an exclusive bus lane in one specifically defined circumstance. Unfortunately, the limits of the study as reported by him may lead to erroneous conclusions.

The following comments stress the importance of considering the underlying demand functions that provide the basis for explaining or predicting (or both) changes in observed crossing volumes that will result from changes in the supply function. In general, observed travel volumes represent an equilibrium between travel supply (or cost) functions and travel demand functions. Any change in either of these functions will result in a change in the equilibrium point and, therefore, in the observed travel volume.

The study reported is based on the assumption that a fixed volume of vehicles and persons cross the bridge. In fact, the underlying economic rationale for travel demand indicates that changes in absolute or relative costs—in this case time costs—will lead to changes in observed volume. In the case under consideration, the probable result would be some increase in the bus-automobile modal split (probably also a shift in the time distribution of vehicle arrivals at the bridge, especially automobiles). Rather than make the fixed demand assumption, it is useful to determine the extent of the modal-split shift necessary to reach a break-even point in terms of the established objectives—in this case, reduction in person-minutes of delay—and then to consider the likelihood of such a shift. Such a break-even point occurs at a modal split of 50 to 51 percent on buses. (The author's assertion that person-minutes of delay is always less without the exclusive bus lane, shown in Figure 4, is misleading in that the modal split is itself dependent on the decision of whether or not to provide an exclusive bus lane.) It is interesting to note that an alternative policy of providing an exclusive bus lane during the period from 7:15 to 8:15 only with priority to buses during the remaining rush period would result in a time delay considerably smaller than that of providing an exclusive bus lane during the entire rush period and would have a break-even result at a modal split of approximately 47 percent (Table 6). The shift to 50 to 51 percent may be unlikely, but a shift in excess of 47 percent is quite reasonable. Although there would be obvious operational difficulties, the benefits might be considerable.

Once the modal-split shift question is introduced, it becomes possible to consider broadening the objectives of the study to consider potential peripheral benefits such as the reduction in CBD congestion. Such peripheral effects may be equally as significant as, or more significant than, the effects of direct time saving.

This discussion indicates some of the reasons for considering the characteristics of vehicle crossing or person crossing demand rather than a fixed-volume condition. An even more comprehensive analysis would consider the shifts in arrival time that might occur as a result of the changes under study. The necessary demand analysis is com-

TABLE 6
COMPARISON OF EXPECTED DELAY TIME

Condition	Delay, person-min		
	Queuing	Running	Total
Present conditions	66,000	37,000	103,000
Plan A ^a —44 percent on buses	289,000	37,000	326,000
Plan A ^a —51 percent on buses	53,000	37,000	90,000 ^c
Plan B ^b —44 percent on buses	134,000	37,000	171,000
Plan B ^b —47 percent on buses	72,000	37,000	109,000 ^d

^aBased on exclusive bus lane from 6:00 to 9:30 a.m.

^bBased on exclusive bus lane from 7:15 to 8:15 a.m. and priority treatment for buses during balance of high-volume period.

^cFor Plan A, breakeven occurs at a modal split of between 50 and 51 percent.

^dFor Plan B, breakeven occurs at a modal split of slightly over 47 percent.

plex but will be ultimately rewarding if it improves the decisions that are made. As part of this analysis, it would be appropriate to consider the development of a probabilistic demand function that could be used to estimate the relative benefits of each alternative over a range of possible conditions. Such a model could be designed to incorporate the generation of nonrecurrent incidents so that their effect on the overall results could be considered.

Although the quantitative approach taken by the author is a significant step forward, an even more rigorous approach is required before discarding a potentially valuable transportation alternative. In any case, the results obtained cannot be readily generalized to other conditions that might lead to quite contrary conclusions.

The author brings up a significant point concerning the relatively limited effect that may be achieved through the establishment of an exclusive bus lane over a short segment of the travel corridor. Future efforts should consider more general possibilities for establishing exclusive or priority lanes over extended portions of the commuter network.

An Empirical Analysis of Lane Changing on Multilane Highways

R.D. WORRALL, Peat, Marwick, Mitchell and Company; and
A.G.R. BULLEN, University of Illinois

The paper describes a macroscopic analysis of lane-changing behavior on multilane highways. It includes descriptions of the pattern and frequency of lane-changing maneuvers observed under varying road and traffic conditions, the distribution of maneuver lengths and times, and the acceptance and rejection of gaps by lane-changing vehicles. Data for the study were collected at a sample of 30 freeway-locations in Chicago.

•LANE-CHANGING BEHAVIOR may be described in terms of 2 measures: (a) frequency, the number of lane changes occurring among all lanes along a given length of road, L, and over a given time span, t; and (b) pattern, the distribution of lane changes between specific lane-lane pairs along a given road length, L, and over a given time span, t.

More formally, if $N_{ij}(t_m)$ lane changes occur between lanes i and j and within a length L of an n-lane roadway during the mth time interval of length t, then the average frequency of lane changing per unit length per unit time, λ , over a series of M such time intervals may be defined as

$$\lambda = \sum_{m=1}^M \sum_{i=1}^n \sum_{j=1}^n \left\{ \frac{N_{ij}(t_m)}{L \cdot M \cdot t} \right\}; \quad N_{ij} \begin{cases} \geq 0 & \text{if } i \neq j \\ = 0 & \text{if } i = j \end{cases}$$

In this paper the frequency of lane changing is expressed generally as an average maneuver rate per 500 ft of roadway per minute.

Similarly, the pattern of lane changing may be expressed as an $n \times n$ transition matrix $\{P_{ij}\}$, such that

$$P_{ij} = \frac{\left[\sum_{m=1}^M N_{ij}(t_m) \right]}{\left[\sum_{m=1}^M V_i(t_m) \right]}$$

where

$$\begin{aligned} V_i(t_m) &= \text{volume in lane } i \text{ during } m \text{ th period } t \text{ and} \\ P_{ij} &= 0 \text{ for } i = j. \end{aligned}$$

If the constraint $P_{ij} = 0$ for $i = j$ is relaxed (i.e., the values on the main diagonal may take on nonzero values) and the value of $\sum_j P_{ij}$ is in turn constrained equal to unity,

then the matrix $\{P_{ij}\}$ represents a stochastic matrix, the elements of which represent the probability that a vehicle entering section L in lane i leaves that section in lane j (i.e., the vehicle changes lanes from lane i to lane j over length L).

Data on the frequency of lane changing were collected at 30 locations by means of time-lapse ground photography. The study sample included sections of 2-, 3-, and 4-lane, 1-directional roadways, situated at varying distances from entrance and exit



Figure 1. Location of study sites in the Chicago area.

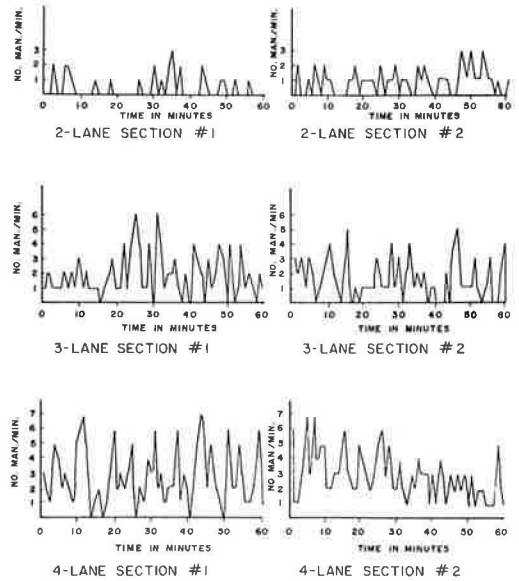


Figure 2. Typical time-series of 1-minute lane-change frequency.

ramp terminals. Mainstream volumes during the study period varied from approximately 1,000 to 5,000 vehicles per hour. Figure 1 shows the location of study sites in the Chicago area.

FREQUENCY OF LANE CHANGING

Figure 2 shows a set of typical time-series plots of lane-changing frequency for sections of 2-, 3-, and 4-lane roadway. Equivalent frequency plots for the same locations are shown in Figure 3. In each case, the pattern is essentially a random, apparently stationary variation about a stable mean value. This value—denoted as the average lane-changing frequency λ for the section—varies both with location and with flow rate.

For 2-lane roadways, the mean lane-changing frequency varies from 0.7 to 1.4 lane changes per 500 ft per minute. For 3- and 4-lane roadways, the equivalent figures are 1.2 to 3.0 and 2.9 to 3.4 lane changes per 500 ft per minute. These figures are based on data collected at points at least 500 ft from a ramp terminal. The equivalent 1-minute variances follow a similar pattern: 0.6 to 0.9 for 2-lane roadways, 1.8 to 2.9 for 3-lane roadways, and 2.9 to 3.8 for 4-lane roadways.

Figure 4 shows a simple plot of the value of λ versus total volume. In this case, the value of λ is expressed as the average number of lane changes per 200 ft per minute. The plot suggests that the intensity of lane changing tends to peak at medium-volume levels and to fall off at both higher

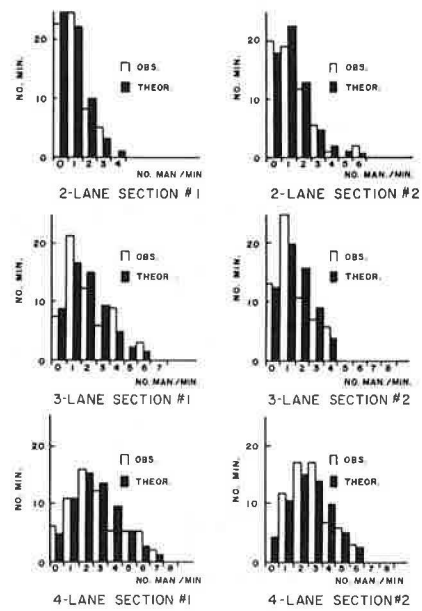


Figure 3. Typical lane-changing frequency distributions.

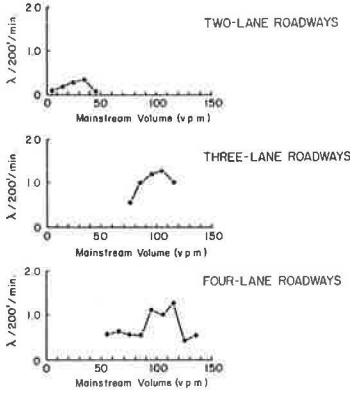


Figure 4. Variation of average lane-changing frequency, λ , with total volume.

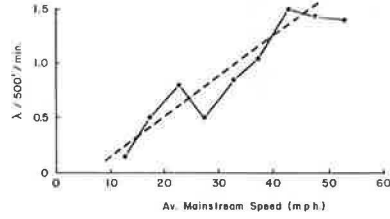


Figure 5. Variation of average lane-changing frequency, λ , with average speed on 3-lane roadway.

shown in Figure 5. The speed data are average minute speeds, analogous to the volume measurements shown in Figure 4, computed for the total sample of 3-lane locations. The lane-change data are expressed as the average number of lane changes per 500 ft per minute. The relationship is approximately linear in form over the 10-to-40-mph range, and lane changing tends to decrease in intensity as speeds increase above 40 mph.

Figures 6 and 7 show the variation in lane-changing frequency between specific lane-lane pairs as a function of their average minute volume and speed differentials. In each case, λ_{ij} is expressed as an average maneuver frequency per 500 ft per minute. The data are again based on the composite sample of 3-lane roadways. Equations based on least squares fits to these data are given in Table 1. As might be expected, lane

and lower flow rates. However, to talk of fitting a formal function to the data is clearly inappropriate.

An essentially similar relationship may be discerned between lane-changing intensity and average mainline speed. This is

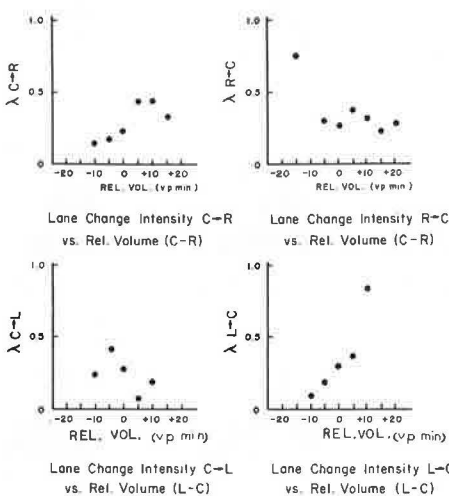


Figure 6. Variation of average lane-changing frequency, λ_{ij} , with lane-lane volume differential on 3-lane roadway.

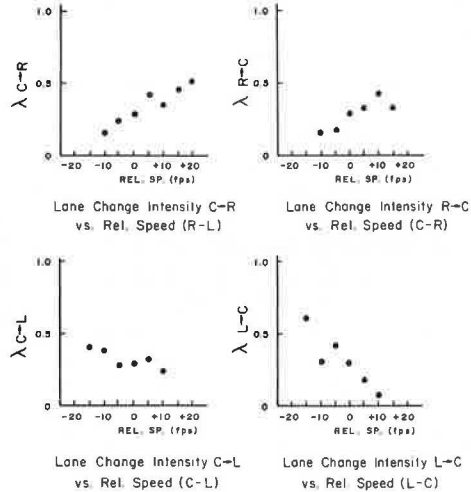


Figure 7. Variation of average lane-changing frequency, λ_{ij} , with lane-lane speed differential on 3-lane roadway.

TABLE 1
RELATIONSHIPS BETWEEN LANE-CHANGING FREQUENCY AND
LANE-LANE VOLUME AND SPEED DIFFERENTIALS ON
3-LANE, 1-DIRECTIONAL ROADWAYS

Lane-Lane Movement	Equations
Center to right	$\lambda_{CR} = 0.276 + 0.011 V_{CR}$ ($R^2 = 0.66$, SE = 0.0048, $\bar{\lambda} = 0.308$)
	$\lambda_{CR} = 0.074 + 0.012 S_{RC}$ ($R^2 = 0.93$, SE = 0.0050, $\bar{\lambda} = 0.342$)
Right to center	$\lambda_{RC} = 0.517 - 0.008 V_{CR}$ ($R^2 = 0.36$, SE = 0.0043, $\bar{\lambda} = 0.355$)
	$\lambda_{RC} = 0.045 + 0.017 S_{CR}$ ($R^2 = 0.68$, SE = 0.0051, $\bar{\lambda} = 0.347$)
Center to left	$\lambda_{CL} = 0.364 - 0.007 V_{LC}$ ($R^2 = 0.24$, SE = 0.0073, $\bar{\lambda} = 0.347$)
	$\lambda_{CL} = 0.468 - 0.011 S_{CL}$ ($R^2 = 0.77$, SE = 0.0027, $\bar{\lambda} = 0.27$)
Left to center	$\lambda_{LC} = -0.136 + 0.026 V_{LC}$ ($R^2 = 0.94$, SE = 0.0037, $\bar{\lambda} = 0.328$)
	$\lambda_{LC} = 0.580 - 0.017 S_{LC}$ ($R^2 = 0.81$, SE = 0.0041, $\bar{\lambda} = 0.315$)

Note: λ_{CR} = average number lane changes per 500 ft per minute between center and right lanes; similarly for λ_{RC} , λ_{CL} , and λ_{LC} .
 V_{CR} = average minute volume differential between center and right lanes; similarly for V_{LC} .
 S_{CR} = average minute speed differential between center and right lanes; similarly for S_{RC} , S_{LC} , and S_{CL} .

changing tends to be to the lane carrying the lower flow and to the lane with the higher speed.

PATTERNS OF LANE CHANGING

The pattern of lane changing along a portion of multilane road may be represented conveniently by a transition matrix, P_{ij} , the elements of which represent the probability of a single vehicle making a transition, i.e., changing lanes, from lane i to lane j , within a given distance and a given time span. Figure 8 shows a set of typical transition matrices, calculated for the 6 study locations used in the discussion of lane-changing frequency.

In each case, the individual values in the cells of the matrix represent average transition probabilities computed from a sequence of successive, 15-minute observations of lane-changing behavior at each location. The variance of these successive 15-minute observations is given in parentheses in each cell.

Similar transition matrices were developed for each of the other 26 study locations included in the sample of ground photography. The total set was classified according to study locations and to the average rates of flow during the observation periods and then subjected to a statistical grouping analysis. The results of this analysis are shown for one study location in Figure 9. Similar analyses, with similar results, were performed for each of the other sections of 2-, 3-, and 4-lane roadways studied.

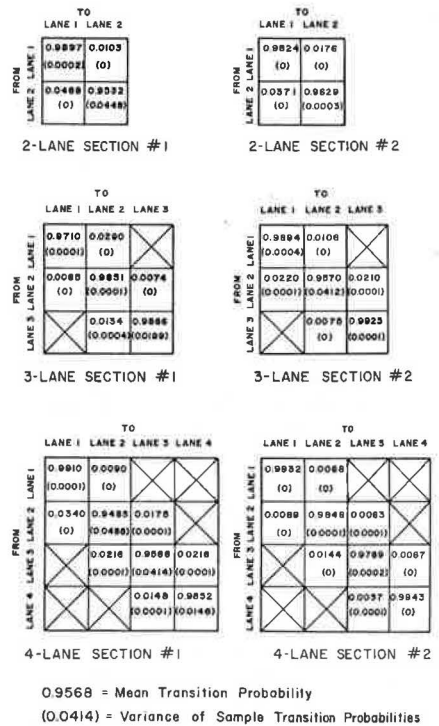


Figure 8. Typical lane-changing transition matrices.

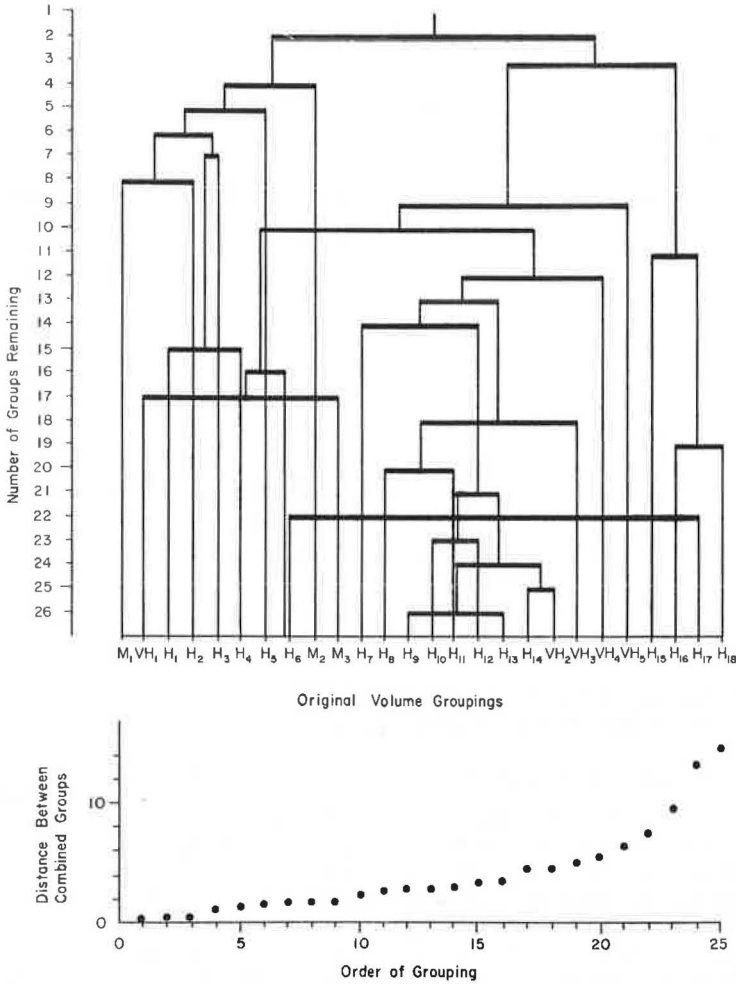


Figure 9. Results of grouping analysis.

Though the data sample is small (and hence some caution is suggested), results suggest that there is little systematic difference between the lane-changing patterns observed at different flow levels for a given location or geometric configuration.

In the 3-lane example in Figure 9, for instance, medium-volume and very high-volume observations or patterns tend to occur at similar flow levels. The differences that do exist appear to be in terms of the intensity rather than the pattern of maneuvers and to be within rather than between volume groupings. This is particularly noticeable if the values of P_{ij} are readjusted to remove the effects of volume and yield simple estimates of the conditional probability of a lane change occurring between lanes i and j , given that a lane change of some sort occurs. Similar results were obtained at each of the other locations studied. In no case did the pattern of lane changing appear to vary systematically with variation in total traffic flow.

TIME AND DISTANCE REQUIRED TO CHANGE LANES

The discussion so far has focused on a simple, macroscopic description of lane-changing behavior. Attention is directed here, and in the next section, to a more



Figure 10. Location of study sites where data were obtained by aerial photography.

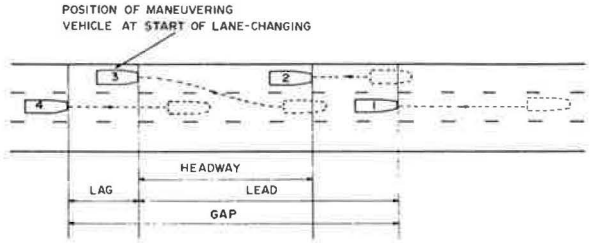


Figure 11. Lane-changing maneuvers and terms.

detailed examination of the actual mechanics of lane changing, expressed, first, in terms of the time and distance required to complete the lane-changing maneuver and, second, in terms of gap acceptance behavior.

Data Collection

Data for the study were derived from 70-mm aerial photography taken at the points shown in Figure 10. A lane change was considered to commence (Fig. 11) when a vehicle first en-

croached on the lane line separating the lane in which it was currently traveling from that into which the lane-change maneuver was to be made. The maneuver was considered to be completed once the vehicle had completely crossed that line. The remaining head and tail portions of the maneuver were then analyzed separately. The head portion of the maneuver is the time and distance required for a vehicle to move from a straight-ahead path in its origin lane to first intercept the dividing lane line, and the tail portion of the maneuver is the equivalent time and distance required for a vehicle to return to a straight-ahead path after crossing that line.

Numerical Results

Figure 12 shows the relationships between volume, speed, and maneuver time; Figure 13 shows the relationships between volume, speed, and maneuver distance.

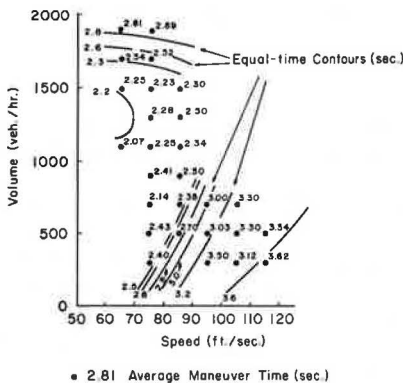


Figure 12. Variation in average maneuver time with traffic volume and speed.

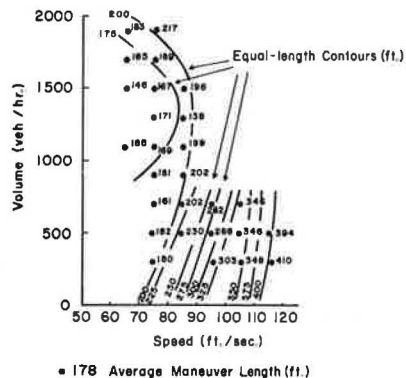


Figure 13. Variation in average maneuver distance with traffic volume and speed.

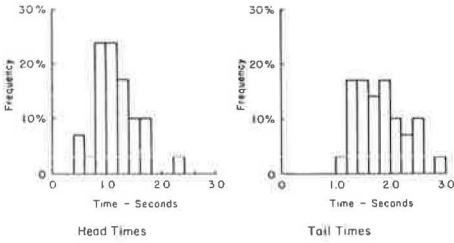


Figure 14. Distributions of head and tail maneuver times.

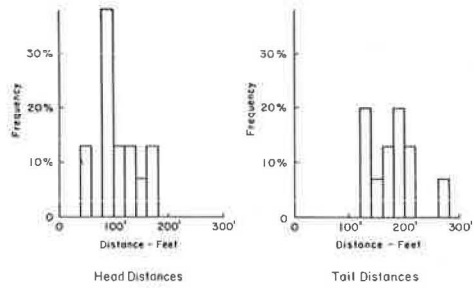


Figure 15. Distributions of head and tail maneuver distances.

The functions $T = h(q, v)$ and $L = g(q, v)$ (where T = maneuver time, L = maneuver length, q = volume, and v = speed) are represented on the figures by families of parabolic curves, each solid "contour line" representing the locus of points of either constant maneuver time or maneuver distance. The range of the data set, i.e., the range of traffic conditions studied, precluded development of a complete contour map for either maneuver length or maneuver time for all traffic conditions. The existence of a parabolic relationship between maneuver length and time and between traffic volume and speed is, however, strongly suggested by the data shown in Figures 12 and 13.

Figure 14 shows the results of an analysis of the head and tail portions of the lane-changing maneuver. The average time required for the head of the maneuver was 1.25 seconds with a standard deviation of 0.4 second and for the tail, 1.95 seconds with a standard deviation of 0.5 second. Figure 15 shows the equivalent distances. The mean distance for the head was 110 ft and for the tail, 160 ft.

The distribution of lateral placement for the changing vehicle prior to and after the lane-changing maneuver is shown in Figure 16. Vehicle placements are relatively widely distributed with respect to the centerline both before and after the maneuver. The average vehicle moved laterally 27 ft before encountering the lane line and 30 ft after crossing the line before regaining a straight travel path. The average lateral speed of the changing vehicle during the head of the maneuver was thus 2.2 ft per sec, and during the tail, 1.5 ft per sec. For the total maneuver, including the head and the tail, the average lateral speed of the changing vehicle was 3.1 ft per sec, reflecting the higher average lateral speed achieved during the core of the maneuver as the vehicle was actually crossing the lane line.

AN ANALYSIS OF MINIMUM ACCEPTABLE GAPS FOR LANE CHANGING

The following 3 definitions are used in this analysis:

1. Accepted Gap—the time (or distance) headway between the leading and lagging vehicles that defines the gap accepted by the maneuvering vehicle and is measured at the start of the maneuver;
2. Accepted Lead—the time (or distance) headway between the maneuvering vehicle and the leading vehicle in the destination lane measured at the start of the maneuver; and
3. Accepted Lag—the time (or distance) headway between the maneuvering vehicle and the lagging vehicle in the destination lane measured at the start of the maneuver.

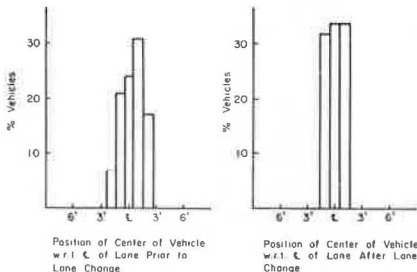


Figure 16. Lateral placement of maneuvering vehicle before and after lane changing.

TABLE 2
MINIMUM DISTANCE AND TIME LAGS, LEADS, AND GAPS
FOR SELECTED DENSITY AND RELATIVE SPEED GROUPINGS

Traffic Density in Destination Lane (vpm)	Relative Maneuver Speed (fps)	Distance (ft)			Time (sec)		
		Lag	Lead	Gap	Lag	Lead	Gap
0 to 3	<10	35	— ^a	126	0.3	— ^a	1.1
4 to 7	<10	48	27	300	0.6	0.2	2.5
8 to 11	<10	30	42	341	0.4	0.4	2.8
17 to 22	<10	20	18	110	0.4	0.3	1.5
23 to 28	<10	17	18	92	0.4	0.3	1.3
29 to 34	<10	17	13	86	0.4	0.3	1.3
35 to 40	<10	28	18	89	0.5	0.4	1.3
0 to 3	>10	76	16	166	0.6	0.4	1.5
4 to 7	>10	20	80	296	0.4	0.7	2.4
8 to 11	>10	61	— ^b	— ^b	0.6	— ^b	— ^b
17 to 22	>10	15	54	215	0.3	0.7	2.9
23 to 28	>10	27	— ^b	238	0.5	— ^b	2.8
29 to 34	>10	14	58	87	0.3	0.7	1.1
35 to 40	>10	— ^a	— ^a	— ^a	— ^a	— ^a	— ^a
Average					0.44	0.44	

^aInsufficient data.

^bOnly large values were observed.

Table 2 gives the values of the 95 percentile minimum gap, lead, and lag values observed in the field. The data are based on a total of 1,706 lane changes observed under varying traffic conditions and are stratified by lane density for the destination lane and relative maneuver speed measured between the maneuvering vehicle and the leading-lagging vehicles defining the destination gap. The 95 percentile value was selected somewhat arbitrarily as representative of the average minimum value for a given traffic condition. The observed values of minimum accepted time gap, lead, and lag vary respectively from 1.1 to 2.9 sec, 0.2 to 0.7 sec, and 0.3 to 0.6 sec. The equivalent distance gap, lead, and lag headway measures are respectively 86 to 341 ft, 13 to 80 ft, and 14 to 76 ft.

No stable relationship is discernible between the observed minimum accepted gap, lead, or lag values and the traffic density in the destination lane. Minimum accepted gap, lead, and lag values in both time and space, however, tend to increase as the relative maneuver speed increases. This increase is most marked in lead rather than lag values. As might be expected, the minimum accepted gap size is greater for a given traffic condition than the sum of the equivalent minimum accepted lead and lag values, suggesting that either the lead or the lag value may be critical in any given maneuver but probably not both.

FORMULATION OF AN ACCEPTANCE FUNCTION FOR LANE CHANGING

Unlike most other gap acceptance and rejection situations, it is not possible in the case of a lane change to identify a rejected gap directly. Some mild computational gymnastics must be resorted to, therefore, in order to generate an empirical acceptance function.

Assume that the size of the gap immediately available to a driver in an adjacent lane in no way influences his initial desire to change lanes, and that the probability of his wishing to change lanes remains constant for a given set of environmental conditions. These conditions might include roadway geometrics, overall volume of traffic, or the relative speeds between given lanes of traffic. Assume also that gap acceptance behavior as observed in the field is an unbiased sample of the behavior of all drivers under a given set of environmental conditions, and that an unbiased estimate may be made of the frequency distribution of available gaps in each adjacent lane.

Then one may write

$$r(t) = a(t) \cdot p \cdot F(t)$$

where

$r(t)$ = the percentage of gaps accepted that are of size t ,

$a(t)$ = the percentage of size t gaps available to all drivers,

p = the percentage of available gaps of all sizes into which a lane change is desired, and

$F(t)$ = the percentage of drivers who will accept a size t gap.

The formulation can be transposed and $F(t)$ can be expressed as

$$F(t) = r(t)/a(t) \cdot p$$

The parameters $r(t)$ and $a(t)$ can be directly measured through sampling techniques applied to actual freeway operations. The parameter p , however, remains unknown. However, because we assume that p is constant, we may consider 2 gap sizes, t_1 and t_2 , and compute

$$F(t_1) = \frac{r(t_1)}{a(t_1) \cdot p} \text{ and } F(t_2) = \frac{r(t_2)}{a(t_2) \cdot p}$$

The ratio of $F(t_1)$ to $F(t_2)$ can be written

$$\frac{F(t_1)}{F(t_2)} = \frac{r(t_1) \cdot a(t_2)}{r(t_2) \cdot a(t_1)}$$

Let

$$N(t) = \frac{r(t)}{a(t)}$$

Then

$$\frac{F(t_1)}{F(t_2)} = \frac{N(t_1)}{N(t_2)}$$

If there exists some gap size t_0 , which we can assume to be acceptable to all drivers, then at this value ($t = t_0$) the acceptance function $F(t_0)$ will take on the value of 1.0, i. e., $F(t_0)$ represents the horizontal asymptote for the acceptance function. Therefore

$$\frac{F(t)}{F(t_0)} = F(t) = \frac{N(t)}{N(t_0)}$$

Determination of the acceptance function $F(t)$ thus requires estimation of the values of $r(t)$ and $a(t)$. Given these values, one may determine, at least conceptually, a functional form for the gap acceptance curves and, hence, compute its mean and variance. Comparison of these values for functions developed for different highway and traffic conditions then provides a convenient basis for evaluating gap acceptance behavior within the lane-changing process.

The time and distance measurements of headway, lag, lead, and gap sizes used in this analysis are shown in Figure 11. All distance measurements are made at the outset of the maneuver. The time measures are then calculated as follows:

Time headway for changing vehicle in origin lane
 = distance headway/speed of changing vehicle
 Time lag for changing vehicle in destination lane
 = distance lag/speed of lagging vehicle
 Time lead for changing vehicle in destination lane
 = distance lead/speed of changing vehicle
 Time gap for changing vehicle in destination lane
 = distance gap/speed of lagging vehicle

Comparison of the speeds of the leading and lagging vehicles in the destination lane indicated that the values did not differ significantly for any of the traffic conditions studied.

The data set was first stratified according to the traffic density in the destination lane and the direction of the maneuver; i. e., lane changes to the left or to the right were separated. The availability of gaps in the adjacent lane was initially estimated on the assumption that the positioning of vehicles in adjacent lanes was independent (i. e., that it was equally likely for a vehicle in the lane adjacent to the maneuvering vehicle to be positioned at any point within the available gap). The availability function was calculated as

$$P(\text{gap} \leq TK) = \frac{\sum_{i=1}^K t_i n_i}{\sum_{\text{all } j} t_j n_j}$$

and

$$P(\text{lag or lead} \leq TK) = \frac{\sum_{i=1}^K t_i n_i}{\sum_{\text{all } j} t_j n_j} + \frac{t_K (\text{number of gaps} > t_K)}{\sum_{\text{all } j} t_j n_j}$$

where

t_i = gap of size t_i and
 n_i = number of times t_i is observed.

The assumption of independence between lanes, however, is only weakly supported by empirical analysis. In view of this, a second method was developed to estimate the availability distribution, $a(t)$, based on the simplistic stratification of the data set according to the distance headways and relative speeds of the vehicles involved in the lane-changing maneuver.

The asymptote to the presumed acceptance function t_0 was estimated by first discounting all but the lower 95 percent of the 2 distributions $r(t)$ and $a(t)$. The values of $r(t)$ and $a(t)$ were then aggregated successively for each interval in the distribution from n to $n-K$ until the following inequality was satisfied:

$$\frac{\sum_{n-K}^n r(t_j)}{n} \geq r(t_j)/a(t_j) \quad \text{for all } j \leq (n - K)$$

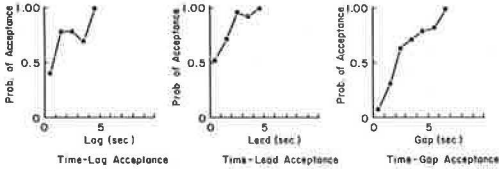


Figure 17. Gap acceptance functions for total data sample.

and

$$r(t_j)/a(t_j) = N(t_j) \text{ for all } j \leq (n - K)$$

$F(t_j)$ can now be written as the ratio

$$F(t_j) = N(t_j)/N(t_0)$$

A set of gap acceptance functions based on the entire data set considered as a whole is shown in Figure 17. Figure 18 shows an equivalent set of acceptance functions based on a 5-level density stratification. The results of a probit analysis applied to the entire data set are shown in Figure 19. The estimated equation is

$$Y = 1.523 + 1.513 X$$

where

- X = \log_e [gap size] and
- Y = probit of $F(t)$.

The mean accepted gap size derived from this analysis is 1.985 seconds. The equivalent lead and lag values are 0.527 second and 0.676 second.

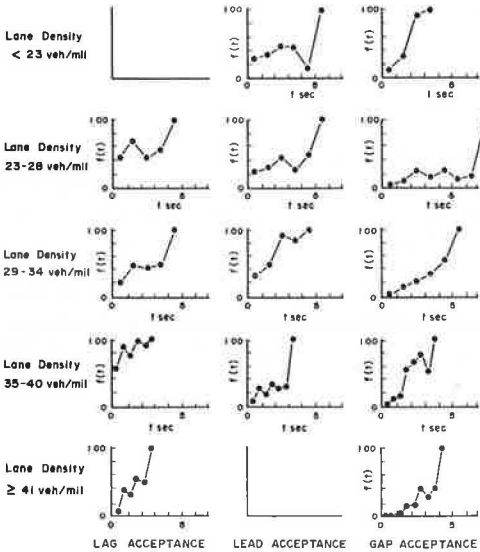


Figure 18. Gap acceptance functions based on empirical availability function and stratified by density.

where

- $t_{95} = t_n$ and
- n = the interval corresponding to the 95 percentile gap.

Let

$$\frac{\sum_{n-K}^n r(t_i)}{\sum_{n-K}^n a(t_i)} = N(t_0)$$

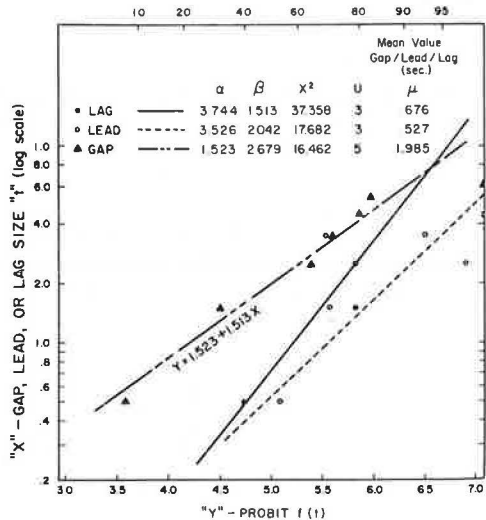


Figure 19. Gap acceptance analysis for total data sample.

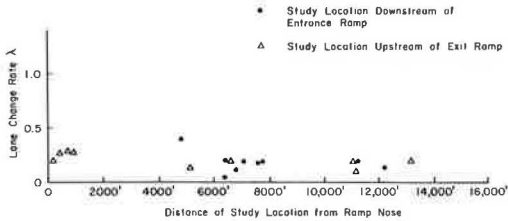


Figure 20. Influence of ramps on lane changing for 2-lane, 1-directional roadway.

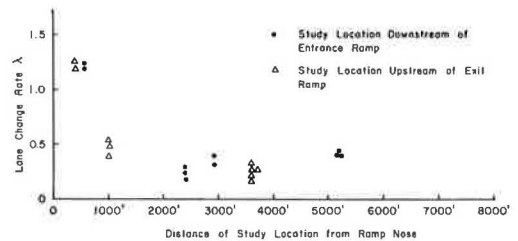


Figure 21. Influence of ramps on lane changing for 3-lane, 1-directional roadway.

THE INFLUENCE OF RAMP TERMINALS AND HIGHWAY DESIGN ON LANE CHANGING

In an attempt to assess the influence of ramp terminals on lane-changing behavior we made estimates of the average lane-changing rate, λ , at varying distances from a sample of exit and entrance ramp terminals. Separate data were collected for 2-, 3-, and 4-lane, 1-directional roadways and for varying volume levels. For each location, the data were tested for any systematic bias or nonrandom characteristic or both that could be related to the design of the highway or its environs. Similar analyses were also performed systematically for lane-changing patterns by the use of the transition matrix notation described earlier. Figures 20, 21, and 22 show the results of these analyses respectively for 2-lane, 3-lane, 4-lane roadways.

The sparseness of the data for a given location and geometric configuration and the absence of any significant experimental control over potentially confounding influences makes it difficult, if not impossible, to draw very meaningful conclusions from these figures. In each case there is a tendency, albeit weak and nonsystematic, for the intensity of lane changing to increase in the vicinity of the ramp terminal. This is most marked on 3-lane roadway sections and much less marked on 2- and 4-lane sections. The variation about this trend, however, is extremely high. In no case are there grounds for developing a formal relationship between average lane-changing frequency and distance to and from ramp terminals. Variations in mainstream volume do not appear to exert any systematic effect on the spatial distribution of maneuver intensity.

Examination of the distribution of minute-by-minute maneuver counts at each location confirms the general results reported earlier. The distribution of maneuvers was again essentially random at all except 3 locations. These 3 were each immediately downstream from an entrance ramp. Examination of the distributions for these locations indicated that the major discrepancy between the observed and the theoretical random distributions could in each case be attributed to the effect of intermittent platoons of vehicles entering the freeway via the upstream on-ramp.

The general, if not very conclusive, finding is confirmed by the variation in λ along a section of approximately 1,500 ft of the Edens Expressway in Chicago (Fig. 23). The figure shows the profile of a gradual increase in the lane-change rate on the approach to the interchange, a decrease within the interchange area, and a sudden increase immediately downstream of the entrance ramp.

Examination of the equivalent patterns of lane changing, expressed as transition matrices P_{ij} , yielded equally inconclusive results. Again, the pattern of maneuver tended to reflect the proximity of the ramp

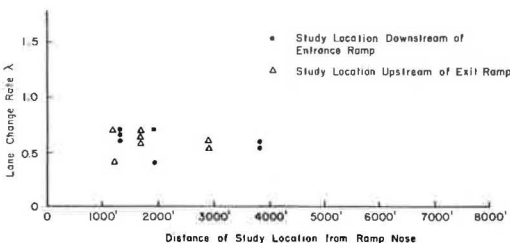


Figure 22. Influence of ramps on lane changing for 4-lane, 1-directional roadway.

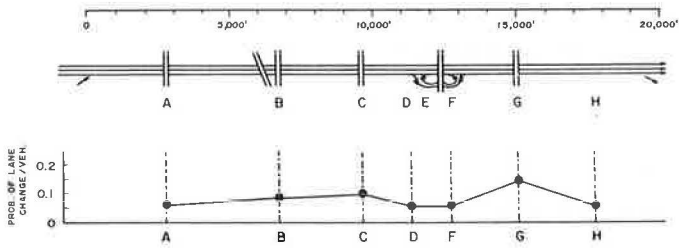


Figure 23. Variation in lane-changing frequency with distance from cloverleaf interchange on 3-lane roadway.

terminal—indicated in this case by an increase in the values of P_{ij} associated with movements into and out of the lane adjacent to the ramp terminal. Again, however, this trend was not sufficiently marked to enable formal distinction to be drawn among the patterns observed at varying distances from entrance and exit terminals.

In interpreting these results, one should recognize that lane-changing maneuvers are generated for a wide variety of reasons and that their patterns and intensity are influenced by many different factors. Ramp location is only one such factor, albeit an important one. Of at least equal importance are the quality and location of directional signing, the location of adjacent ramp terminals, the volume of truck traffic, the relative speeds in the adjacent traffic lanes, and the aggressiveness of individual drivers. All these and more contribute significantly to the pattern and intensity of maneuvers observed at any one point in either time or space. Given the systemic, and rather crude, viewpoint adopted here and the absence of any significant degree of experimental control, it is not surprising that the results obtained were relatively bland and inconclusive.

Equally important, however, is the fact that no generally dominant influence of ramp terminals on lane-changing behavior was detected at the level of analysis pursued here. Given that most, if not all, of the locations studied represented good rather than bad examples of geometric design, this suggests that lane-changing behavior is not seriously influenced by the proximity of ramp terminals, and particularly that the intensity of maneuvers is not increased by ramps spaced at the intervals (roughly $\frac{1}{2}$ to $1\frac{1}{2}$ miles) observed in the field.

SUMMARY AND CONCLUSIONS

This paper has presented a selection of elementary empirical results concerning lane-changing behavior on 1-directional, multilane roadways. The discussion has ranged from considerations of the frequency and pattern of lane changing to an analysis of the mechanics of the actual lane-changing maneuver and the effects of ramp location on lane-changing behavior. All of the analyses have been based on field data collected in the Chicago area.

Lane changing is shown to be essentially a random event in the traffic stream, subject to considerable variation at any point in either time or space. Average frequencies and patterns of maneuver exhibit a weak, but systematic, relationship to traffic and roadway conditions. Maneuver times and lengths similarly tend to vary as a function of the volume and speed of traffic. Gap acceptance behavior displays only a very weak, and as yet largely undefined, relationship to traffic flow. Although the average rate of lane changing varied considerably across the set of traffic conditions and field locations included in the study, the distribution of maneuvers over time remained random in virtually every case. The major exceptions to this finding were study locations situated immediately downstream from high-volume entrance ramps. There was no indication that the overall pattern or intensity of lane changing changed from random to nonrandom at a given distance from an entrance or exit terminal.

A final word of caution is in order. Although these findings are based on the analysis of a relatively large and well-defined data base, the amount of substantive information underlying some of the conclusions and the range of traffic and design conditions for which these conclusions are valid without extrapolation are relatively small. A truly definitive analysis here should be based on a much larger sample of study locations. In all cases, it should be borne in mind that only a very crude experimental control could be exercised over the collection and analysis of the data and that all of the results pertain to data collected in the Chicago area; no information was obtained, for example, on lane changing on grades where the results may be very different from those obtained here for level terrain. Equally, no attempt was made to explain why lane-change maneuvers occurred but only to describe some of their characteristics. Clearly, this analytical perspective is reflected in and may partially bias the results discussed here.

ACKNOWLEDGMENT

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An Evaluation of Ramp Control on the Harbor Freeway in Los Angeles

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This paper describes a ramp control project on the 8-lane Harbor Freeway in Los Angeles. Congestion occurred each weekday in the 4 outbound lanes during the evening peak period, while the adjacent parallel streets were relatively uncrowded. The control project is designed to reduce total travel time through the freeway corridor by metering 5 on-ramps and closing 1 ramp in a 5-mile section during the peak period.

Some unusual techniques are being employed: Buses are allowed to bypass ramp queues and to make left turns where other traffic cannot; storage of queued vehicles on a frontage road is carried back across an intervening major thoroughfare by timing the intersection signal so that the intersection itself remains clear; and, at one location, storage room is increased and high metering rates are made possible by the release of vehicles 2-abreast at the ramp signal.

Results show that freeway users are saving about 1,000 vehicle-hours per day against a loss, or increased travel time for diverted or delayed ramp traffic, of about 130 vehicle-hours.

●IN SEPTEMBER 1968, the California Division of Highways, with the cooperation of the Los Angeles City Traffic and Police Departments, began a freeway ramp control project on the southbound Harbor Freeway in south Los Angeles. The Harbor Freeway is a major 8-lane facility extending 22 miles from downtown Los Angeles to San Pedro. Basically, the project involves limiting on-ramp traffic in a 5-mile section of the 4 southbound lanes during evening peak hours (Fig. 1). The purpose is to reduce delay to freeway users and gain optimum use of the freeway and street system during the rush hours.

The science of controlling ramp traffic to reduce total travel time (freeway and surface streets) is comparatively new. However, the Harbor Freeway project is not the first project of this type. A similar, although smaller, system has been in effect in Los Angeles on the Hollywood Freeway for more than a year; other projects are in operation in San Diego, Chicago, Detroit, and Houston. The Harbor Freeway was selected as the first major effort in Los Angeles because of the high probability of success. The availability of relatively uncrowded parallel streets in the Harbor Freeway corridor and excess capacity on the downstream freeway sections were promising indicators that more good could be done more quickly here than anywhere else in the Los Angeles area. Also, several on-ramps in this area adapted themselves rather easily to metering (i.e., available frontage road and storage area). Control of 6 on-ramps to the southbound Harbor Freeway is accomplished by various means that were determined by an in-depth study requiring application of basic traffic engineering principles at each ramp. One

- Metered Ramps
 ⊗ Closed Ramp

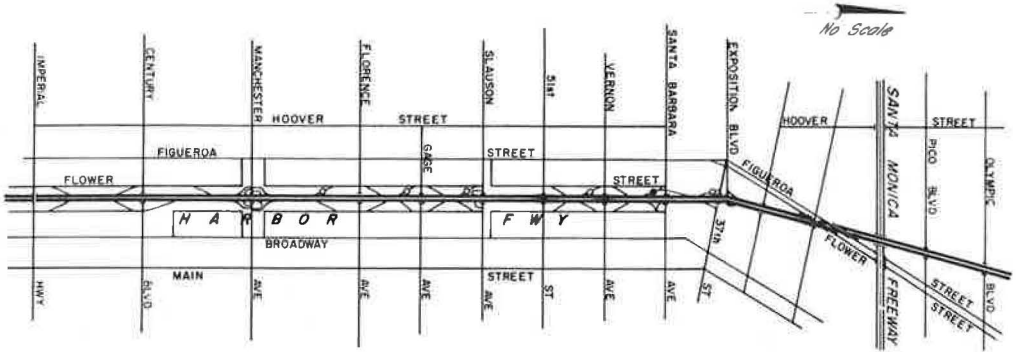


Figure 1. Location of ramp control project.

ramp is closed during the control period by the use of barricades, 2 ramps are controlled by platoon metering, and 3 ramps are controlled by single-car metering. Generally, the control period is from 3:45 to 5:45 p. m. each weekday.

This paper is primarily an evaluation of the ramp control project, but it also briefly describes the planning prior to implementation. The conclusions drawn are based on extensive data collected before and after the project was initiated.

CONDITIONS BEFORE CONTROL

Freeway

Prior to the ramp control, congestion was caused by simply too much traffic trying to use the southbound Harbor Freeway south of Exposition Boulevard. Typical weekday operation of the freeway before control is shown in Figures 2, 4, and 5, representing density, speed, and volume. (Figure 3 shows density after control.) Congestion normally started about 3:45 and lasted until after 6:00 and extended from Manchester Avenue to just north of Adams Boulevard.

The density contours (Fig. 2) indicate 2 distinct congested areas—1 south and 1 north of Slauson Avenue. The congestion was the result of excess demand of approximately 500 vehicles per hour (vph) affecting 2 key sections. The most southerly bottleneck was where the Florence Avenue on-ramp traffic joined an almost full freeway and caused excess demand for the section. This caused congestion that extended to and upstream of Slauson Avenue. The second major bottleneck was caused by Vernon Avenue on-ramp traffic joining a near-capacity flow to create excess demand for an upgrade (800 ft at approximately 5 percent) approaching Slauson Avenue.

Total delay on a typical weekday was about 60,000 vehicle-minutes from 3:45 to 6:15 p. m. Maximum delay to individual vehicles was more than 6 min upstream of Slauson Avenue and about 3 min between Slauson and Manchester Avenues.

Figure 4 shows travel times and speeds throughout the evening period; travel times before control were frequently greater than those shown. Average speeds for the entire trip from 30th Street to Century Boulevard were as low as 15 mph, with much stop-and-go driving. A freeway trip from 30th Street to Century Boulevard frequently took about 17 min during the peak period.

Typical peak-period flow rates, shown in Figure 5, represent conditions from about 5:00 to 5:30 p. m. At this time, the entire length upstream is controlled by the capacity at the most southerly bottleneck. For this reason, throughput upstream of Manchester Avenue was considerably below capacity of the various sections. Before 5:00 p. m.,

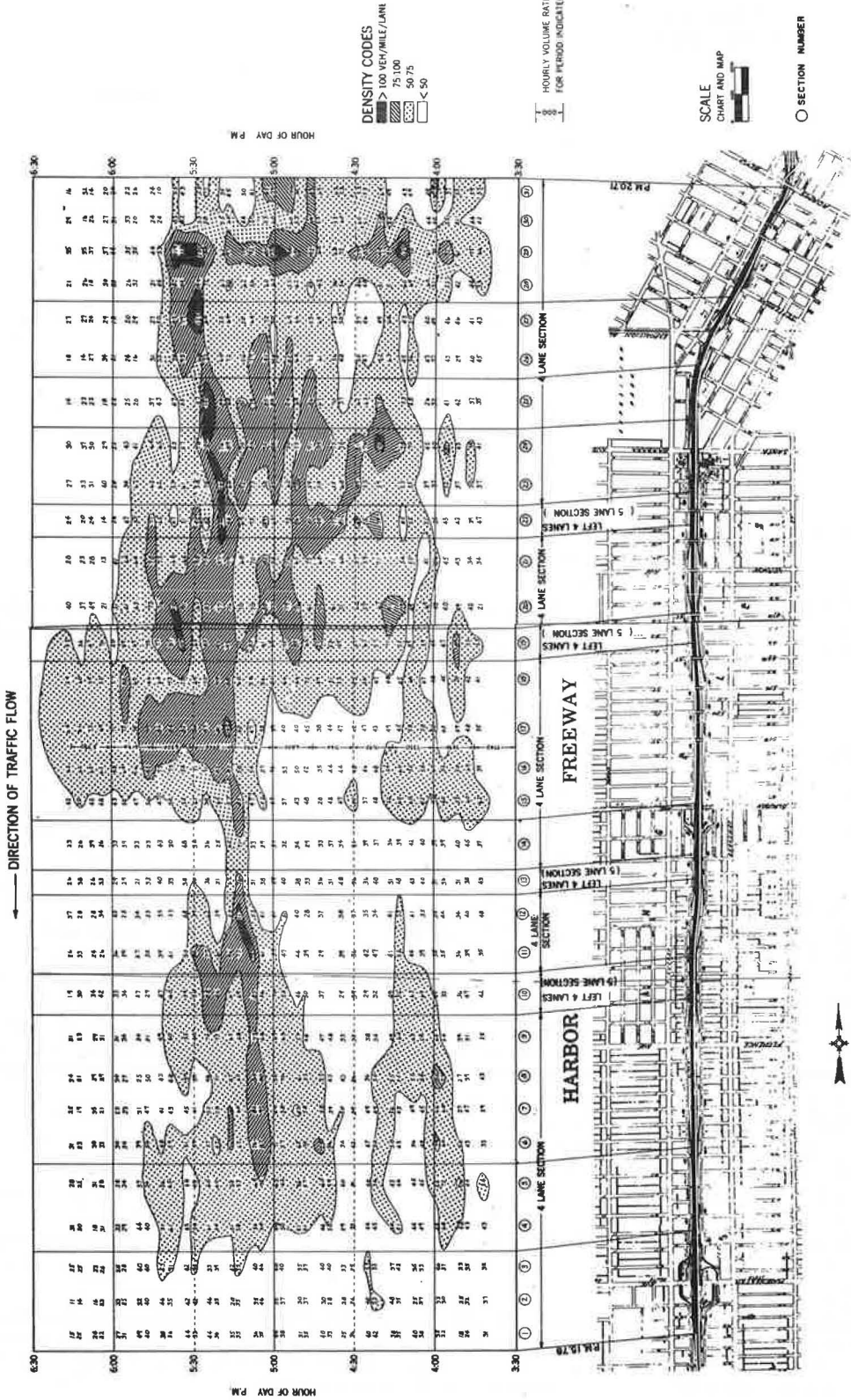


Figure 2. Density before controls on the southbound Harbor Freeway from 3:30 to 6:30 p.m., Tuesday, October 25, 1966.

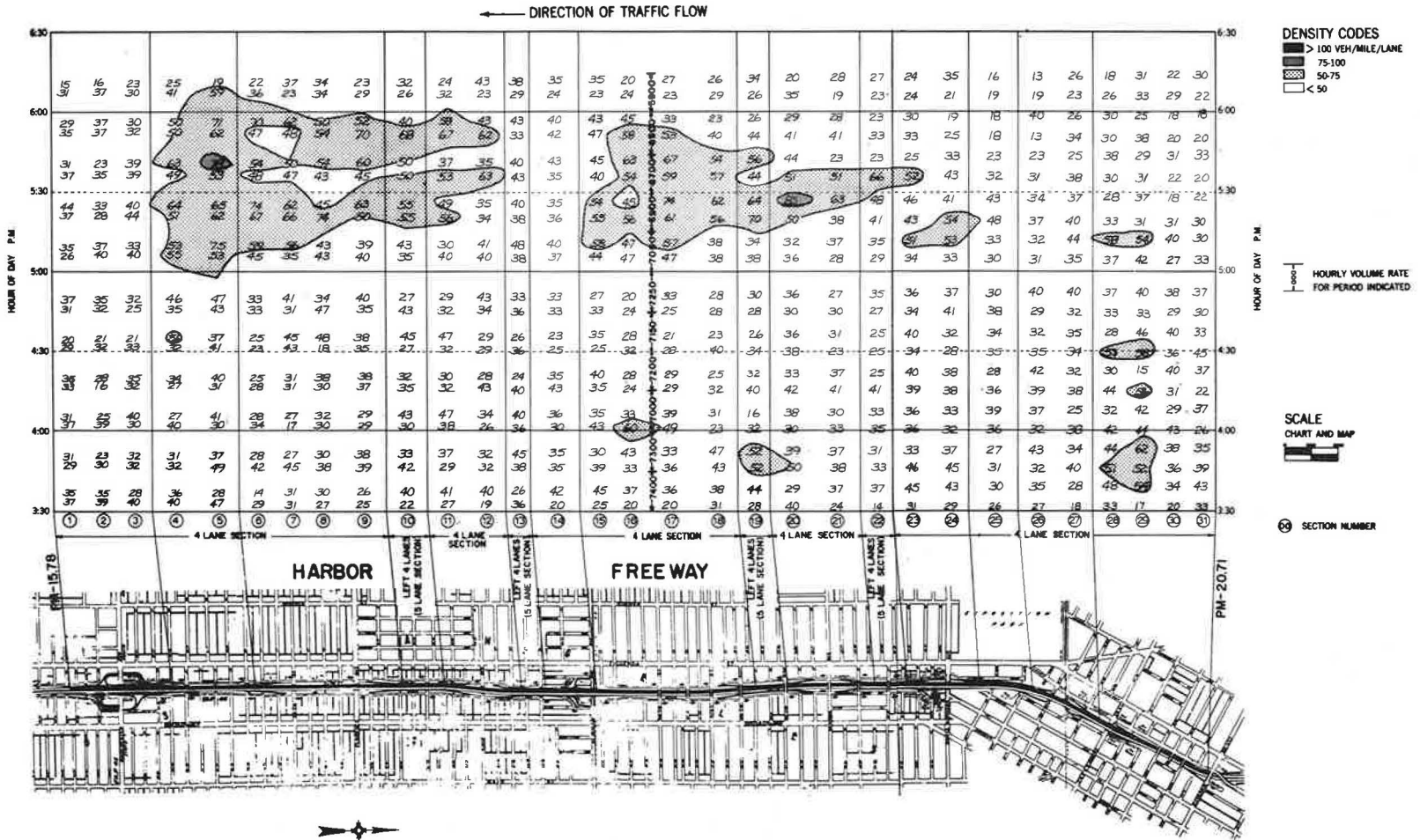


Figure 3. Density after controls on the southbound Harbor Freeway from 3:30 to 6:30 p.m., Wednesday, October 16, 1968.

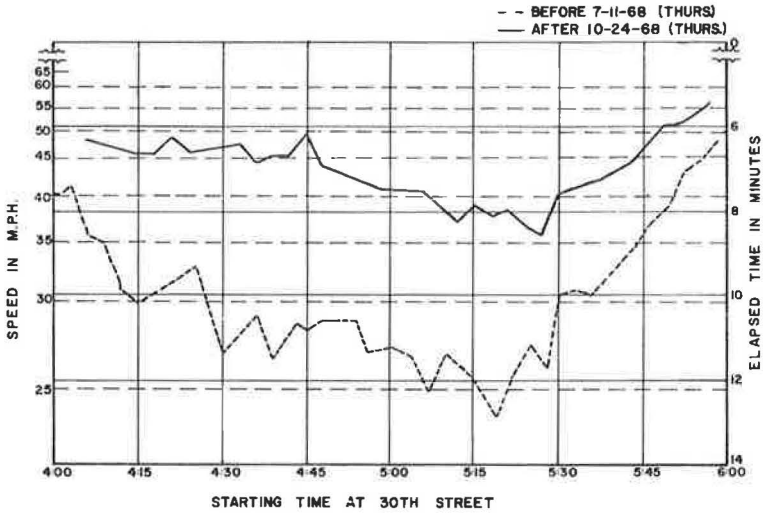


Figure 4. Travel time and speed on the southbound Harbor Freeway between 30th Street and Century Boulevard (5.08 miles).

conditions were somewhat similar, except that the Slauson Avenue grade was an independent bottleneck with flow rates of about 7,100 vph.

Surface Streets

Preliminary data indicated that the parallel streets, Broadway and Figueroa, would be the best alternate routes for diverted traffic. Both of these are major thoroughfares. Broadway has 4 lanes, a painted median, and left-turn pockets. Figueroa has the same and, in addition, a parking lane on each side with restricted parking during peak hours.

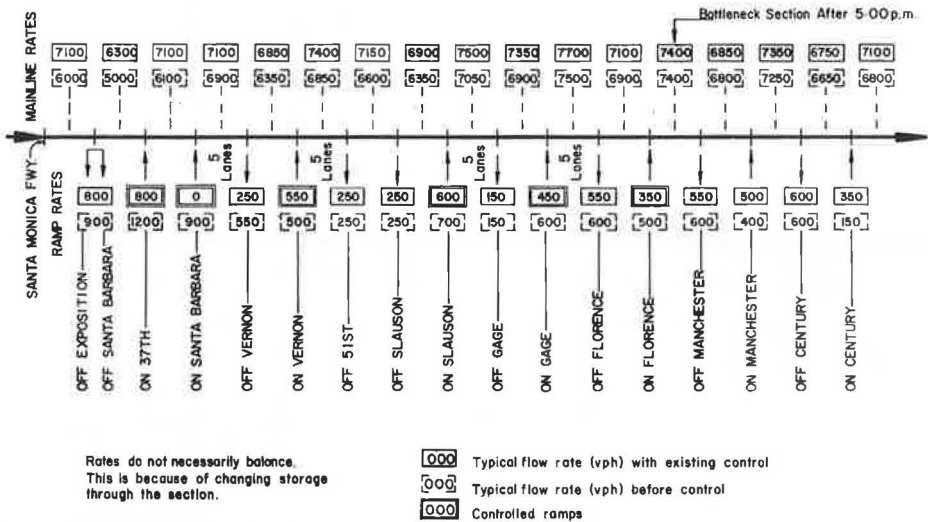


Figure 5. Typical peak-period flow rates from approximately 5:00 to 5:30 p.m.

westbound 37th Street because these turning movements would block eastbound 37th Street traffic. The metering rate of 650 to 850 vph is too great for single-car metering. Platoon metering was the answer, and it has worked well. The 37th Street on-ramp was particularly adaptable to platoon metering because of a long collector road. This collector road also handles off-ramp traffic that tends to break up platoons and approximate single-car metering.

Santa Barbara Avenue

The Santa Barbara Avenue on-ramp (Fig. 7) had to be closed to all traffic, except buses, because of inadequate storage for metering.

Vernon Avenue

The Vernon Avenue on-ramp (Fig. 8) has a minimum storage area, so normal metering at the ramp would cause interference with Vernon Avenue traffic. Closure of the ramp would be too restrictive. Therefore, it was necessary to devise a method for control that allowed approximately the existing volume to enter without an extensive queue forming at the ramp. Metering signals were placed on the west frontage road just upstream of the Vernon Avenue off-ramp. These signals intercept traffic coming from Santa Barbara Avenue and release an amount slightly below the metering rate at the Vernon Avenue ramp signal. This allows the right turn from eastbound Vernon Avenue to reach the on-ramp. Initially, the left turn from westbound Vernon Avenue was prohibited because the ramp queue might have kept this traffic from clearing the intersection. Now the left turn has a separate green phase and pocket lane. Traffic on the west frontage road approaches in 2 lanes and is released in 2-lane platoons from the front-



Figure 7. Santa Barbara Avenue on-ramp that is closed because of lack of storage space. Buses have always had a separate entrance to the ramp and can be easily allowed special access. Ramp is closed by hand-placed barricades that will be eventually replaced by automatic "pop-ups". Queue on Figueroa Street is the result of preferential signal timing for Santa Barbara Avenue traffic. Although not related to the project, green time on Santa Barbara Avenue has been increased since the start of controls. This is the key intersection in the entire project area.



Figure 8. Vernon Avenue on-ramp where single-car metering from 2 lanes is used because of limited storage space. Metering signals are used on the 1-way frontage road to keep stored traffic from blocking the Vernon Avenue intersection. Green phases of the frontage road and Vernon Avenue signals coincide. Queue on the northbound off-ramp is unrelated to project; queues were a frequent occurrence before controls.

Average speeds on both Broadway and Figueroa were 15 to 30 mph. A trip from 30th Street to Century Boulevard via Figueroa took an average of about 15 min and a peak of 19 min. Hourly volumes on southbound Broadway ranged from 1,200 vehicles at 57th Street to 850 vehicles at 85th Street. On Figueroa, volumes were as high as 1,400 vehicles at 57th Street and diminished to about 1,100 vehicles at 85th Street.

PLANNING

The basic theory used in planning was a demand-capacity analysis on the freeway. For 2 bottlenecks that were located, capacity was determined, demand was estimated, and a ramp control plan was developed that would reduce demand to what each section could handle. A more comprehensive discussion of the theory of ramp control is presented in another paper (1).

An origin and destination survey of traffic using the ramps within the critical area was necessary for an accurate prediction of the pattern of traffic diversion caused by control. We were able to predict that a high percentage of the traffic subject to control would enter at upstream locations and that parallel facilities would not become overloaded.

During the planning phase, meetings were held with interested individuals and local agencies to inform them of our efforts. The ramp users were informed by handouts the week before control began. Alternate route signs were placed on the city streets to aid the diverted traffic.

One of the most important phases of the project was planning the method for control at each ramp location to determine the amount of traffic that should be let on and the means for minimizing the effect of control on "innocent" traffic. When severe metering was necessary, we knew that time and not distance would determine the queue length. In other words, the driver is concerned with the amount of time it takes him to enter the freeway, rather than with the distance he must travel. Therefore, if a short queue were desired, then a more restrictive metering rate had to be used. Increasing the delay to the individual motorist causes him to seek an alternate route.

The metering signals operate on a fixed-time basis with up to 3 different rates operated by a time clock; a 3-dial controller is used. The rates are set based on "typical" conditions or traffic patterns. The system is not traffic-responsive. In the following is a discussion of some of the major points considered in planning the controls at each ramp.

37th Street

The 37th Street on-ramp (Fig. 6) was the most critical on the project. It is the first on-ramp south of the Santa Monica Freeway and as such has a very high demand (1,200 to 1,300 vph). Limiting this traffic to 650 to 850 vph could have very seriously affected nonfreeway traffic on 37th Street and Flower Street, and possibly Exposition Boulevard and Figueroa Street. This problem was minimized by the prohibition of left turns into the ramp from



Figure 6. 37th Street on-ramp where platoon metering is used because of high metering rates used. Most traffic comes from southbound Flower Street and is stored in the left-turn lane upstream of Exposition Boulevard. Left turns to the ramp from westbound 37th Street are prohibited because they would interfere with eastbound 37th Street traffic. Right lane of the ramp is striped and signed for buses and emergency parking only. Congestion in northbound lanes is the result of a previous incident in the northbound lanes and is not affecting southbound traffic at this time.

age road metering signals. These metering signals are interconnected with the signals at the Vernon Avenue and west frontage road intersection. The green phases of the two sets of signals coincide so that metered traffic does not have to stop at Vernon Avenue.

In order to handle these two lanes of traffic, the Vernon Avenue on-ramp entrance was widened to 2 lanes. The ramp metering signals release 1 car at a time from each lane. The ramp signals are located near the entrance from the frontage road leaving most of the ramp available for the 2 cars to merge into 1 lane before they reach the freeway acceleration lane. Initially, it was difficult to maintain single-car operation, but this was partially resolved through the use of signs that read METERED—ONE CAR PER GREEN THIS LANE. With these signs, violations are around 10 percent; without the signs, they were 15 to 20 percent.

Slauson, Gage, and Florence Avenues

On the basis of traffic volumes before control at Slauson, Gage, and Florence Avenues, metering appeared to be necessary at these ramps only after 5:00 p. m. However, diversion from upstream ramps before 5:00 p. m. was anticipated (which in fact happened), so the metering periods at these ramps were set to approximately coincide with those for upstream ramps (3:45 to 5:45 p. m.). This holds the ramp volumes to about the same as they were before control.

Manchester Avenue and Century Boulevard

On-ramp volumes at Manchester Avenue and Century Boulevard were not so high that they had to be metered; however, signals were placed at each in anticipation of an increase from diverted traffic. Volumes have increased, but not to the point that metering is necessary.

Buses

Alterations were made to provide special access for buses. At 37th Street (Fig. 6), the right lane of the ramp was striped and signed for buses only. This allows them to bypass the ramp queue. The ramp configuration at Santa Barbara Avenue, where buses have always had a separate entrance to the ramp (Fig. 7), made the access of buses and prohibition of other vehicles quite easy.

CONDITIONS AFTER CONTROL

Freeway

The before-and-after density data (Figs. 2 and 3) were used to estimate freeway travel times. These data give a good picture of the amount of congestion between Adams Boulevard and Manchester Avenue on typical days. There is considerable congestion upstream of Adams Boulevard, and this has not changed much with control.

Total freeway travel time before control from 3:45 to 6:15 p. m. between Adams Boulevard and Manchester Avenue was 188,000 vehicle-minutes per day. Now during the same period, freeway travel time is approximately 128,000 vehicle-minutes, a reduction of 60,000 vehicle-minutes in spite of an increase in the total vehicle-miles of travel.

During the period before control, there were 81,000 vehicle-miles of travel on the freeway. This means that during the total 2½-hour period average speed between Adams Boulevard and Manchester Avenue was 2.32 min per mile (188,000 divided by 81,000) or 25.8 mph. During the period after control, there were approximately 86,000 vehicle-miles of travel, and average speed was 1.49 min per mile or 40.3 mph. Without the extra vehicle-miles of travel, the after travel time would have been 81,000 times 1.49 or 121,000 vehicle-minutes. Savings on the freeway, therefore, were actually 67,000 vehicle-minutes for the base or original amount of travel instead of the calculated difference of 60,000 vehicle-minutes.

The extra travel on the freeway is caused by about 900 vehicles that now enter the freeway upstream of the control section instead of at the controlled ramps as they for-

merly did. These trips should add to the savings, although to determine their actual savings is virtually impossible. Probably, where there is enough backtracking, some of these trips actually lose time compared to the time they took before control. However, we are sure the net result is added savings because (a) the great majority of these extra trips originated north of the control section and (b) their responses to a survey questionnaire indicate they save time. Net savings are not as great as noted, however, because there is delay to diverted traffic and to vehicles at the ramp metering signals.

Approximately 500 vehicles no longer use controlled ramps where delay may be experienced. They are staying on surface streets to Manchester Avenue or Century Boulevard, or staying off the freeway completely. These trips will average about 2 minutes longer than they formerly did on the freeway for a total delay of 1,000 vehicle-minutes. This is a conservative estimate as it assumes that all this traffic starts upstream of 37th Street and goes all the way to Century Boulevard, but, actually, some of it is on streets a lesser distance and would not suffer as much delay.

Queue lengths versus time were plotted for each controlled ramp on a typical day. With these, the delay caused by the signals was estimated for each on-ramp. Total delay at the ramps is about 9,000 vehicle-minutes per day. The net savings then to traffic in the freeway corridor is at least 50,000 vehicle-minutes per day from the ramp control.

Generally speaking, individual motorists who enter the freeway at 37th Street or Vernon Avenue save time even though they must wait at the metering signals. They more than make up their lost time at the signal by increased speeds on the freeway. Those who enter at Slauson or Gage Avenue break even; their wait at the ramp is about the same as their gain on the freeway. Those entering at Florence Avenue are the only ones who actually lose time because they were entering at the head of the line before control.

Figure 4 shows travel time and speed for individual trips through the control section. These were obtained by photographic methods (taking pictures of traffic at points along the freeway and matching cars in the pictures to get travel time) and floating car runs. The before data were collected during the summer of 1968 and represent somewhat less congestion than that indicated by the before densities, which are for a day in October 1966. After control, average speed is frequently more than 40 mph. Before control, a trip from 30th Street to Century Boulevard off-ramp could have taken as long as 16 to 17 min (even though the densities do not show this). This same trip now takes a maximum of 7 to 9 min. Conditions upstream of 30th Street have remained about the same.

Before control, about 6,300 vehicles entered the freeway from the 6 controlled ramps between 3:45 and 5:45 p.m. With the Santa Barbara Avenue ramp closed and the others metered, now about 4,900 vehicles are allowed to enter.

Increased input to the control section from the freeway upstream indicates about 900 of the diverted vehicles now enter the freeway upstream of the control. These motorists are probably not backtracking to enter the freeway, that is, their trips originate upstream of the control area. An origin-destination survey made prior to control showed that at least 32 percent of the Vernon Avenue on-ramp traffic would probably have entered upstream had it not been for the freeway congestion. These motorists obviously feel they are saving time if they have changed their routes. It is estimated that now 500 vehicles either stay off the freeway altogether or use a ramp downstream of the controls.

As expected, off-ramp volumes have decreased somewhat since control. Before control, 45 percent of the traffic exiting at Vernon Avenue alone continued south along the freeway corridor. Other off-ramps were somewhat similar. These motorists were either re-entering the freeway downstream or staying off completely to avoid freeway congestion until their desired destination.

Figure 5 shows typical peak-period flow rates on the freeway. These volumes are representative of the period from 5:00 to 5:30 p.m. when the greatest throughput increase is realized. During this period, the downstream bottleneck at the Florence Avenue on-ramp controlled the before period throughput of all upstream sections including the Slauson Avenue grade.

Capacity of a bottleneck cannot change, except in very special cases and then only to a very limited degree. Although there is no conclusive evidence, capacity of the Slauson Avenue grade may have increased slightly because of increased approach speeds, but the rate increase from 6,600 to 7,150 vph shown in Figure 5 does not represent this increase. The low before rate was a result of the downstream bottleneck controlling output during this period.

No doubt, the best overall picture of the changed traffic conditions is the before-and-after densities shown in Figures 2 and 3. The after densities indicate that, before 5:00 p. m., there are only isolated points of densities over 50 vehicles per mile per lane (vml) with the major portion of operation below capacity. This slack allows a natural recovery capacity to dissipate congestion resulting from incidents. After 5:00 p. m., there are still pockets of congestion upstream of both bottleneck sections. Initial metering rates did not eliminate this congestion because they were based on a precontrol condition where input to the section dropped significantly after 5:00 p. m., and off-ramp volumes increased. This condition changed once control began. Input now stays at a high level until 5:20 p. m., and off-ramp volumes actually decreased during this part of the control period.

Surface Streets

Subjectively, traffic conditions on the city streets have not changed appreciably. Trips through the control area on either Broadway or Figueroa Street average about the same time as they did before. Speeds are between 15 and 30 mph. Speeds on the surface streets were obtained by a license plate study that involved recording time and license plate number at key locations. These were then matched to determine elapsed time. Floating car runs were used to supplement these data.

Not all the diverted traffic could be located with the techniques available. No doubt some motorists are using routes other than Broadway or Figueroa Street. However, based on origin-destination data, street patterns, and travel times, Broadway and Figueroa Streets are the best alternate routes for most of the diverted traffic. And the effect on these routes was barely measurable with travel times and volume counts.

An estimated 500 vehicles have been diverted from the freeway throughout the control period. Traffic volumes, obtained at various locations on Figueroa Street and Broadway, show that the volume has not significantly increased at any one location. Five hundred vehicles spread over a 2-hour period and throughout the adjacent streets have easily been absorbed.

RESULTS AND CONCLUSIONS

The project is very successful. Delay to the freeway motorist is significantly reduced, delays to controlled traffic are relatively minor, and travel time on adjacent streets has shown little or no increase. Basically, the ramp control prevents about 1,400 vehicles, which used the freeway during the before period, from entering the on-ramps within the control area from 3:45 to 5:45 p. m. Of these, approximately 900 vehicles now enter the freeway at some ramp upstream, and 500 vehicles are being diverted to surface streets.

Congestion on the southbound Harbor Freeway is almost entirely eliminated south of Exposition Boulevard, except on days when incidents occur and reduce capacity. A minimum average speed of 40 mph is now maintained throughout the control period—with occasional shock waves. This compares with 15 to 25 mph under stop-and-go driving conditions before control. Individual motorists save a maximum of 9 min per through trip on some days, and the average motorist saves 4 to 5 min per trip. The controls benefit not only through traffic, but also drivers that get off the freeway in the control area, depending on where they exit.

Traffic conditions on the parallel city streets, principally Broadway and Figueroa, are practically unaffected by the ramp control. Total travel times and volumes through the control area on either Broadway or Figueroa Street have remained nearly the same.

The net effect of the control project is estimated to be a time reduction on the freeway of about 60,000 vehicle-minutes and increased travel time of roughly 10,000 vehicle-

minutes per day to diverted traffic and traffic waiting at the metering signals. This assumes that travel time has not significantly increased for motorists who have always used city streets. Drivers who formerly entered the freeway in the control area and now enter upstream also save time, but the amount is unknown.

Solicited opinions of users at the Manchester Avenue and Century Boulevard off-ramps show the motorists to be very much aware of, and in favor of, the system. Ninety-two percent of the responses indicated approval. Those entering at the controlled ramps were not as enthusiastic as those entering upstream, but the majority still approved the project. Figure 9 shows a typical response.

The project shows that a relatively small number of vehicles in excess of capacity at key bottlenecks can cause severe and recurrent congestion. In this case, the excess was about 500 vehicles over a 2-hour period.

Significant knowledge has been gained about ramp control from this project. The following are the important points:

1. Planning of control methods is very important and, in this project, was directly responsible for minimum disruption to surface street traffic. The slight sacrifices in the optimum freeway control that sometimes must be made to ensure good street operation are not critical.
2. Platoon merging or random merging of single vehicles has caused no problems.
3. Incidents are very frequent. The entire 5-mile section at times is operating virtually at capacity and any incident in this reach has a drastic effect. When operating at capacity, the storage built up by each incident cannot be dissipated for the rest of the control period. The sharp reductions in metering rates that would be required to dissipate congestion are usually not possible because of the severe congestion that would be caused on surface streets. In fact, unless good information can be given to drivers approaching metered ramps, we do not feel a sharp and nonpredictable (to the driver) fluctuation of metering rates is advisable. Because of the frequency of incidents and difficulty in dissipating resultant congestion, operating at volumes slightly below capacity, if possible, is probably justifiable. This slack allows a natural recovery capacity. In

Where did you enter the freeway:

SANTA MONICA FREEWAY

15:16

(Give name of ramp, street or other freeway. For example: 5th St., 37th St., Pasadena Freeway.)

If you have been a frequent user of the freeway at about this time, please answer the following questions:

Prior to the start of the control project, did you usually enter the freeway at the same location noted above? Yes No

If "No", where did you usually enter:

VERNON

Do you think freeway driving conditions, on an average day without accidents, have been improved since the project started: Yes No

If "Yes", how much time do you believe you save: 10 to 15 minutes

Comments: Prior to the control project, I entered the freeway at Vernon because of the congestion from th Santa Monica & Harbor interchange. Now it is a constant flow from the interchange.

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Figure 9. Survey response of driver that exited at Century Boulevard about 5:20 p.m.

our project this is possible from 3:45 (start of control) to 5:00 p. m. We now operate with some slack during this period. The adjacent streets are at or near capacity only from 5:00 to about 5:30 p. m. After 5:00 p. m. the metering would have to be too restrictive to operate with any slack.

4. This is not a traffic-responsive system. Such a system would provide some additional benefits. However, for reasons we have mentioned, more refined metering rates is not one of them. The primary benefit would be to take advantage of unused capacity downstream of incidents and to allow for major changes in input demands. In other words, we could start and end control at different times more in keeping with actual freeway conditions.

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The Analysis and Design of Freeway Entrance Ramp Control Systems

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This paper describes some of the results obtained from an analytic study undertaken as part of the Gulf Freeway Surveillance and Control Project in which the on-line, dynamic control of individual entrance ramps is investigated. For this study 2 alternate control philosophies are considered. The first philosophy concerns the control of ramps on which a string of one or more vehicles is released when a suitable merge opportunity arises and when all previously released vehicles have successfully merged into the freeway stream. The second control philosophy concerns ramps on which the requirement that the ramp be cleared of all previously released vehicles is relaxed to increase the merge capacity of that ramp. In this second case, the controller may release an additional vehicle whenever the expected delay associated with attempting a merge into a detected gap is less than the expected delay associated with waiting for the ramp to clear. Here the controller is a sequential decision-maker that evaluates the expected delay associated with all previously released vehicles that have not yet merged and reflects this information into the control process. Fixed-time and demand-capacity metering controllers are special cases of the control system analysis presented.

•SEVERAL FREEWAY CONTROL projects are presently under way in the United States, and each is attempting to improve the operational characteristics of a freeway by use of an entrance ramp control system. However, the operational modes for the respective control systems and the rationale that underlies individual control system designs appear to be quite different. As a result of this diversity, there appears at first to be no clear-cut design philosophy with which to approach a new access ramp control system design problem. Because of this deficiency, a study group at the Polytechnic Institute of Brooklyn undertook the development of a more unified design theory for dynamic control systems for freeway entrance ramps. The study, initiated in August 1967, was part of the Gulf Freeway Surveillance and Control Project conducted by Texas A&M University in Houston, Texas, for the U.S. Bureau of Public Roads. The portion of that study described in this paper concerns the design of a dynamic control algorithm for an arbitrary entrance ramp configuration, subject to the assumption that control is exercised by use of a green-amber-red traffic signal located at a fixed position on the ramp. Control systems designs that utilize other controller configurations, such as speed signs or multiple signal stations, are not included in this presentation.

For the type of systems considered, 2 identifiable primary functions are (a) to improve the merge service offered to vehicles that enter the freeway and (b) to improve the operation of the freeway. The first function is performed by any system that better coordinates the arrival of the merge vehicle at the merge zone with the availability of a high-quality merge opportunity, and the second function is performed by any system that improves the freeway operational characteristics such as volume, speed, density,

accident rate, and occupancy. These 2 functions are not always compatible, but there is evidence to indicate that compatible solutions do exist. In particular, observation of the Gulf Freeway reveals that operation under conditions of ramp control (6) yields the following:

1. The rush period volume increased by about 10 percent;
2. The speed on the test section increased by about 30 percent;
3. The average travel time over a 5-mile section decreased from 16 to 11 minutes; and
4. The number of accidents during the 2-hour morning peak decreased from 145 to 75 per year for the 3 inbound lanes over the 6.5-mile controlled section.

DYNAMIC RAMP CONTROL

To understand the nature of the control system design problem for a freeway entrance ramp required that initial consideration be focused on the Gap Acceptance Control System. In this system any intervehicular headway in the outside freeway lane in excess of T_1 seconds is defined as a gap into which a vehicle that seeks to enter the freeway may be placed. To achieve the placement of entering-vehicles into detected gaps, however, the gap detection process must be carried on sufficiently upstream from the merge point to allow both for synchronization after suitable travel time of vehicles on the ramp and for survival of detected gaps during the passage from the detection point to the merge point. In addition, the gap threshold T_1 must be sufficiently large so that an acceptable portion of the drivers execute a synchronized moving merge. Within this framework the design problem is then to select locations for the traffic control signal and the gap detector equipment and to specify the gap threshold T_1 .

Location of the control signal is restricted by the need for sufficient room to accelerate prior to the merge maneuver and by the desire to provide sufficient room for the queue that develops behind the signal. Coordinated with this is the need to place the gap detector as close to the merge point as possible to obtain the best possible gap survival conditions. The choice at any particular ramp is thus limited. The specification of T_1 to provide an acceptable rate of gap acceptance for merging drivers, however, is arbitrary and a large range for choice exists. When T_1 is chosen to be small, there are many gaps and the smaller of these are likely to be rejected by the drivers. For larger values of T_1 there are fewer gaps, but the smaller of the gaps are deleted and the likelihood that a driver will reject the offered gap is reduced.

If the design concept is to be properly established, first consideration must be given to the rate at which gaps (i.e., headways in excess of T_1) appear in a traffic stream of volume q . Figure 1 shows that the gap detection rate, $\mu(T_1, q)$, is a function of both the threshold limit T_1 and the lane volume q . Then, it is necessary to note that for very large gaps the probability that a driver accepts the gap for a moving merge is approximately 1, and for sequentially smaller gaps the probability of acceptance by the driver is reduced toward a finite limit, K , between 0 and 1 (Fig. 2). This lower limit corresponds to the probability that a driver can force entrance into a stream at a point at which no headway was detected. Based on these 2 concepts, subdivision of all gaps into 2 groups is then possible; the 2 groups include those that are accepted for a moving merge and those that are not accepted for a moving

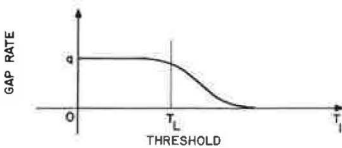


Figure 1. Gap detection rate as a function of threshold limit and lane volume.

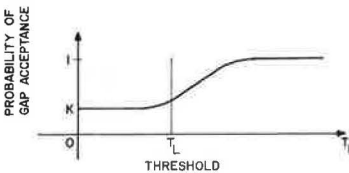


Figure 2. Gap acceptance by driver as a function of gap size.

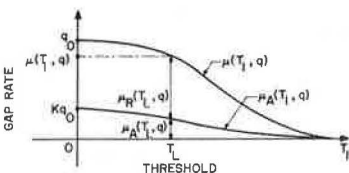


Figure 3. Gaps accepted and not accepted by drivers for moving merge.

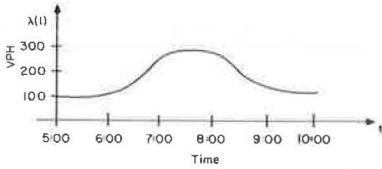


Figure 4. Mean of time-dependent Poisson process used to characterize the ramp arrival process.

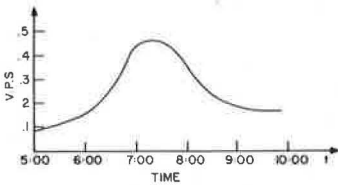


Figure 5. Time-dependent freeway volume.

merge (Fig. 3). Corresponding to this division, $\mu_A(T_1, q)$ is the rate at which accepted gaps are detected, and $\mu_R(T_1, q)$ is the rate at which rejected gaps are detected; the sum of these quantities, $\mu(T_1, q)$ is the rate at which all gaps in excess of T_1 are detected, that is,

$$\mu(T_1, q) = \mu_R(T_1, q) + \mu_A(T_1, q) \tag{1}$$

Single-Vehicle-Release-After-Completion Mode

For any specified value of gap threshold, $T_1 = T$, the rate at which gaps are detected is simply $\mu(T, q)$. Thus a driver who randomly arrives at the controller suffers an expected delay of $1/\mu(T, q)$ seconds before a gap is detected. Then, the driver must proceed down the ramp for which the expected travel time is denoted by t_r . In addition, the probability that the offered gap is unacceptable is equal to the ratio of unacceptable gaps to total gaps, $\mu_R(T, q)/\mu(T, q)$. Hence, an extra expected delay associated with the rejection of an offered gap is

$$\frac{\mu_R(T, q)}{\mu(T, q)} \cdot \frac{1}{\mu_S(q)}$$

where $\mu_S(q)$ is the occurrence rate of gaps that are acceptable for merge by drivers who have previously rejected the gap offered to them and are stopped in the merge zone. These quantities can be used to evaluate the total expected service time for a driver who arrives at the ramp control signal for merge service onto the freeway as

$$t_e = \frac{1}{\mu(T, q)} + t_r + \frac{\mu_R(T, q)}{\mu(T, q)} \cdot \frac{1}{\mu_S(q)} \tag{2}$$

As a result, when operation is restricted to a one-at-a-time mode and release is predicated on the completion of the merge maneuver by all previously released vehicles, a single-vehicle-release-after-completion system exists for which the ramp service rate μ' is a function of the threshold limit T and the volume q in the outside freeway lane. This service rate is

$$\mu' = \frac{1}{t_e} = \frac{\mu_S(q) \mu(T, q)}{\mu_R(T, q) + \mu_S(q) [1 + t_r \mu(T, q)]} \tag{3}$$

when t_e is large compared with $1/q$ so that the time interval between the instants at which sequential gaps are sought is large compared with average headway in the stream. (Note that the condition probability of a gap at the instant $t = T_0$ given a gap at the instant $t = 0$ approaches the unconditioned probability of a gap at the instant $t = T$ when T_0 is made very large. For an Erlang headway process on the freeway, this is equivalent to $t_e \gg 1/q$.)

The Design of a Controller for the Single-Vehicle-Release Mode

One useful and fairly common model for an urban freeway employs stochastic processes with slowly varying parameters to account for both ramp arrivals and highway flows. In particular, the peak-period ramp-arrival process is often described as a time-dependent Poisson process (1) with $\lambda(t)$ as shown in Figure 4. In addition, the

intervehicular spacings for vehicles on the highway are described as independent samples from an Erlang distribution (2, ch. 9):

$$f(t) = \frac{(aq)^a t^{a-1} e^{-aqt}}{(a-1)!} \quad (4)$$

where a is an integer and q varies slowly as a function of time corresponding to average volume (Fig. 5). Based on these descriptions and subject to the assumption that the process parameters vary slowly, one controller design philosophy is to maximize the service rate μ' subject to a limitation of downstream freeway capacity. The service rate μ' can be rewritten with the arguments omitted as

$$\mu' = \frac{1}{t_R + (\mu_R + \mu_S)/\mu\mu_S} = \frac{1}{t_R + 1/\mu''}$$

From this form it is seen that, for a given t_R , the maxima of μ' correspond to the maxima of μ'' . Hence, setting $d\mu''/dT = 0$ located the values of T corresponding to the relative minima of μ' . In particular,

$$d\mu''/dT = \frac{\mu_S(\mu_R + \mu_S)(d\mu/dT) - \mu_S\mu(d\mu_R/dT)}{(\mu_R + \mu_S)^2} = 0 \quad (5)$$

reduces to

$$(\mu_R + \mu_S)(d\mu/dT) = (d\mu_R/dT)\mu \quad (6)$$

because $(\mu_R + \mu_S)^2$ is positive and finite, and $\mu_S \neq 0$. From Eq. 6,

$$(d/dT) [\ln(\mu) - \ln(\mu_R + \mu_S)] = 0 \quad (7)$$

Because T corresponds to the threshold that is set on the minimum spacing between vehicles on the freeway into which a moving merge will be attempted, all spacings in excess of this threshold are gaps. For any given threshold limit, the rate at which gaps appear in a stream with volume equal to q vehicles per second is

$$\begin{aligned} \mu &= q \int_T^\infty \frac{(aq)^a t^{a-1} e^{-aqt} dt}{(a-1)!} \\ &= q e^{-aqT} \sum_{i=0}^{a-1} \frac{(aqT)^i}{i!} \end{aligned} \quad (8)$$

When the probability that a presented gap of t is accepted by a driver for a moving merge is described (2, ch. 9) by

$$P_a(t) = 1 - e^{-Kt} \quad (9)$$

then the rate of successful moving merges is

$$\begin{aligned} \mu_A &= q \int_T^\infty (1 - e^{-Kt}) \left[\frac{(aq)^a t^{a-1} e^{-aqt}}{(a-1)!} \right] dt \\ &= q \left\{ e^{-aqT} \sum_{i=0}^{a-1} \frac{(aqT)^i}{i!} - \frac{(aq)^a}{(aq+K)^a} e^{-(aq+K)T} \sum_{i=0}^{a-1} \frac{[(aq+K)T]^i}{i!} \right\} \end{aligned} \quad (10)$$

and the rate of unsuccessful moving merges is

$$\mu_R = q \frac{(aq)^a}{(aq + K)^a} e^{-(aq+K)T} \sum_{n=0}^{a-1} \frac{[(aq + K)T]^n}{n!} \quad (11)$$

When the indicated derivatives are evaluated and employed in Eq. 6, then algebraic manipulations yield

$$\mu_S = \frac{q e^{-(aq+K)T}}{(aq + K)^a} \left\{ \sum_{n=0}^{a-1} \frac{[(aq + K)^a (aq)^n - (aq + K)^n (aq)^a] T^n}{n!} \right\} \quad (12)$$

as the equation from which the optimum threshold settings are obtained, subject to the constraint $T \geq 0$. Because this equation is of the form $\mu_S = f(T)$ where

$$f(T) = A e^{-BT} \sum_{n=0}^{a-1} C_n T^n \quad (13)$$

and because

$$C_n > 0 \text{ for every } n \quad (14)$$

the sum

$$\sum_{n=0}^{a-1} C_n T^n > 0 \text{ for } T > 0 \quad (15)$$

Thus, $f(T) > 0$ for $T > 0$. In addition, the derivative of the expression $f(T)$ is

$$f'(T) = (d/dT) \left[A e^{-BT} \sum_{n=0}^{a-1} C_n T^n \right] \quad (16)$$

where

$$f'(T) = -KA(aq + K)^a \left[e^{-BT} \sum_{n=0}^{a-1} \frac{(aqT)^n}{n!} \right] \quad (17)$$

Inspection of this quantity reveals that $f'(T)$ is negative for all positive T . Therefore, $f(T)$ is monotonic decreasing for $T \geq 0$. On this basis, the conclusion is that a unique optimum solution for the controller threshold T exists. That value is 0 when

$$\mu_S \geq q \frac{1}{(aq + K)^a} \left[(aq + K)^a - (aq)^a \right] \quad (18)$$

Otherwise, the value is the positive number T_{opt} , obtained as the solution to Eq. 12.

After T_{opt} is evaluated as described, the control policy is specified next. In particular, when the sum of the volume on the freeway and the demand on the ramp does not exceed the volume limitation for the freeway, a threshold of T_{opt} is used for the ramp. This ensures the highest possible service rate for demand on the ramp and results in minimum expected delay and minimum expected queue length. When the sum of the freeway volume and ramp demand exceeds the volume limitation, a threshold of T_{opt} is not acceptable. Instead a threshold T_c must be employed such that the sum of the freeway volume and the served portion of the demand is equal to or less than the volume limitation.

At this point, the heretofore undefined quantities of freeway volume, ramp service, and volume limitation must be considered more exactly. For this purpose it is noted that experience and traffic flow theory (3) indicate that the short-term production of any given point on a freeway cannot exceed some upper bound Q , approximately 2,000 vehicles per hour (vph), without significant risk of breakdown in the flow of traffic. Thus the T_A minute running average of flow past a critical point must be limited to approximately QT_A vehicles, depending on the facility. When this short-term average volume is below the specified capacity limit, additional vehicles from the ramp can be admitted into the stream, provided that the running average of the sum remains below the critical value. Thus the average ramp service is limited at most to the difference between the limiting and actual average volumes.

Many solutions are possible to the controller design problem that satisfy this restriction. One such possibility allows the threshold to be set at T_{opt} , provided that the restriction is met, and inhibits all merging otherwise. By this process all available roadway capacity is used as quickly as possible, and additional waiting vehicles are inserted into gaps as additional room for single vehicles arises. This solution, when associated with the limit condition in which T_{opt} equals 0, becomes the capacity adjusted metering system (4, 5).

A second approach to the controller design problem involves the gradual adjustment of the threshold as a function of freeway volume. This technique has the usual advantages associated with smooth variation in controller policy; however, it also has the added limitation, extra delay, associated with smoothing. In particular, the first policy carries the risk of inserting an acceptable averaged number of vehicles into the stream too quickly thus causing breakdown, and the second policy includes the risk of adjusting too slowly thus overloading the stream.

The Multiple-Vehicle-Per-Gap Merge Mode

In the generalized release-after-completion merge mode, a string of vehicles is released to attempt to merge into a suitable single gap after all previously released vehicles have completed the merge operation. To design a controller for this action requires only the specification of the gap threshold T_n that corresponds to the smallest gap into which a string of n vehicles may attempt a merge. When this is done, the sequence of numbers $\{T_n\}$ determines the number of vehicles that may attempt to merge into any given gap.

The subsequent analysis and controller design is simplified by the assumption that the gap threshold is chosen so that the probability that the last vehicle in a released string of n fails to merge is substantially larger than the probability that 2 or more vehicles in the string balk. Thus, the expected service time t_{en} for a string of n vehicles that is released to attempt to merge into a gap in excess of T_n but less than T_{n+1} is

$$\begin{aligned}
 t_{en} &= \text{the expected travel time for the first vehicle to reach the} \\
 &\quad \text{merge zone + the sum of the expected intervehicular head-} \\
 &\quad \text{ways between vehicles in the string + the expected extra} \\
 &\quad \text{delay due to gap rejection by the } n \text{th vehicle} \\
 &= t_r + (n - 1) h + (\text{Prob } n \text{ are released and the } n \text{th balks}) (1/\mu_S) \\
 &= t_r + (n - 1) h + (1/\mu_S) \frac{[\mu_{Rn}(T_n) - \mu_{Rn}(T_{n+1})]}{[\mu(T_n) - \mu(T_{n+1})]} \quad (19)
 \end{aligned}$$

where

t_r = the expected travel time for 1 vehicle on the ramp,

h = the expected intervehicular headway,

μ_S = the rate at which vehicles stopped at the merge zone execute merges into the freeway stream,

$\mu(T)$ = the rate at which gaps in excess of the threshold setting of T appear in the stream, and

$\mu_{Rn}(T)$ = the merge rejection rate for the n th vehicle in a string when the release threshold is set at T .

Next, it is noted that the rate at which merge opportunities appear in the stream, μ_{Total} , is

$$\begin{aligned}\mu_{\text{Total}} &= \sum_{n=1}^{\infty} n \text{ (the rate of merge opportunities for strings of exactly } n \text{ vehicles)} \\ &= \sum_{n=1}^{\infty} n[\mu(T_n) - \mu(T_{n+1})] = \sum_{n=1}^{\infty} \mu(T_n)\end{aligned}\quad (20)$$

Based on this, the expected service time associated with the merge opportunities offered by the stream T_e is

$$\begin{aligned}T_e &= \sum_{n=1}^{\infty} \left[\frac{\text{(merge opportunity rate for strings of } n \text{ vehicles)} \times \text{(total expected delay)}}{\text{(total merger opportunity rate)}} \right] \\ &= \sum_{n=1}^{\infty} \frac{\mu(T_n) - \mu(T_{n+1})}{\mu_{\text{Total}}} \left\{ t_r + (n-1)h + (1/\mu_S) \left[\frac{\mu_{Rn}(T_n) - \mu_{Rn}(T_{n+1})}{\mu(T_n) - \mu(T_{n+1})} \right] + \frac{1}{\mu(T_1)} \right\} \\ &= \frac{1}{\mu_{\text{Total}}} \left\{ (t_r - h)\mu(T_1) + h\mu_{\text{Total}} + (1/\mu_S) \sum_{n=1}^{\infty} [\mu_{Rn}(T_n) - \mu_{Rn}(T_{n+1})] + 1 \right\} \\ &= h + \frac{\sum_{n=1}^{\infty} [\mu_{Rn}(T_n) - \mu_{Rn}(T_{n+1})] + [1 + (t_r - h)\mu(T_1)] \mu_S}{\mu_S \mu_{\text{Total}}}\end{aligned}\quad (21)$$

Finally, $\mu_{Rn}(T_n) \gg \mu_{Rn}(T_{n+1})$ is implied by the assumption that the probability of gap rejection by the last released vehicle in a string is much greater than the probability of a balk by any other vehicle in that string. Thus,

$$T_e = h + \frac{\sum_{n=1}^{\infty} \mu_{Rn}(T_n) + \mu_S [1 + (t_r - h)\mu(T_1)]}{\mu_S \mu_{\text{Total}}}\quad (22)$$

and the merge capacity limit for the stream is

$$\begin{aligned}\mu_{\text{Stream}} &= \frac{1}{T_e} = \frac{1}{h + \frac{\sum_{n=1}^{\infty} \mu_{Rn}(T_n) + \mu_S [1 + (t_r - h)\mu(T_1)]}{\mu_S \mu_{\text{Total}}}} \\ &= \frac{1}{(h + t')}\end{aligned}\quad (23)$$

where

$$t' = \frac{\mu_S \mu_{\text{Total}}}{\sum_{n=1}^{\infty} \mu_{Rn}(T_n) + \mu_S [1 + (t_r - h)\mu(T_1)]}\quad (24)$$

Hence, the merge service capacity is maximized when t' is minimized. A controller design based on this is considered next.

The Design of a Controller for String-Release Mode

The extension of controller action to include the release of strings of vehicles for insertion into adequately large gaps offers several advantages at the expense of separate, individualized merge service. In particular, the safety level associated with the single-vehicle-release-after-merge mode is exchanged for a higher merge capacity when demand indicates that this is necessary. The result is a control law that offers individual service when demand is low and that allows for extended merge capability when demand is high, provided that adequate freeway capacity exists.

Inspection of the expression for the capacity of the ramp controller, μ_{Stream} , reveals this quantity depends on the set of threshold settings $\{T_n\}$. In general, this expression need not be unimodal, and the coordinates of the local maxima are not always identified by a sequential search. There is a good basis, however, for setting the thresholds by a sequential optimization. In particular, because there is no information included in the model to indicate the actual demand for service, an increase in the ramp capacity by the increase of the 2-vehicle merge rate at the expense of the 1-vehicle merge rate is detrimental to system performance when the actual demand requires only 1-vehicle merges. This situation is contrasted by the following: When the value of T_1 is selected to maximize the 1-vehicle merge process and then T_2 is selected to maximize the 2-vehicle merge process given T_1 , the resultant system always provides the highest 1-vehicle merge rate and yields extra capacity through 2-vehicle merges as required by demand. Although this controller may not provide as large a total ramp capacity as is available by the simultaneous selection of T_1 and T_2 , only the actual demand will determine under which controller the ramp provides better service.

When the intervehicular headways on the freeway are described as independent samples from the Erlang distribution

$$f(t) = \frac{(aq)^a t^{a-1}}{(a-1)!} e^{-aqt} \quad (25)$$

and the probability that the last vehicle in a string of n vehicles accepts an offered gap of t seconds or less is described by the cumulative Erlang

$$P_{an}(t) = 1 - e^{-S_n b_n t} \sum_{m=0}^{S_n-1} \frac{(S_n b_n t)^m}{m!} \quad (26)$$

the values of $\mu_{Rn}(T_n)$, $\mu(T_n)$, and μ_S can be evaluated. Here

$$\mu(T_n) = q \int_{T_n}^{\infty} f(t) dt = q e^{-aqT_n} \sum_{\ell=0}^{a-1} \frac{(aqT_n)^\ell}{\ell!} \quad (27)$$

and

$$\begin{aligned} \mu_{Rn}(T_n) = q \int_{T_n}^{\infty} [1 - P_{an}(t)] f(t) dt = q \sum_{m=0}^{s_n-1} \left[\frac{(s_n b_n)^m}{m!} \frac{(aq)^a}{(a-1)!} \frac{(a+m-1)!}{(aq + s_n b_n)^{a+m}} \right] \\ \cdot \left[e^{-(aq+s_n b_n)T_n} \sum_{\ell=0}^{m+\ell-1} \frac{(aq + s_n b_n)^\ell T_n^\ell}{\ell!} \right] \quad (28) \end{aligned}$$

Likewise, when $P_s(t)$ is the standing merge headway acceptance probability,

$$\mu_S = q \int_0^{\infty} P_S(t) f(t) dt$$

and

$$\mu_S = q \sum_{m=0}^{r-1} \frac{(a+m-1)! (rc/aq)^m}{m! (a-1)! (1+rc/aq)} a + m \quad (29)$$

when $P_S(t)$ is cumulative Erlang type r with mean equal to $1/C$.

The expression for ramp capacity for this case has been evaluated for some examples subject to the philosophy of sequential optimization. Capacity versus threshold is shown for outside lane volumes between 800 and 2,000 vehicles per hour in Figures 6 through 13 to indicate the system sensitivity to adjustment of the threshold T_1 . Figures 14 through 21 show capacity versus string length for similar volumes. In all cases the lower order thresholds are set at their optimum value before the threshold of interest is varied. In addition, it is assumed that the gap acceptance probability that corresponds to the end vehicle in a string is type 3 Erlang with a mean that varies linearly with string length.

Finally, it is noted that when the maximum string length is restricted to 1, and the gap acceptance probability for single vehicles is Erlang type 1, the string merge capacity simplifies into the 1-vehicle merge capacity previously developed.

CAPACITY AND SENSITIVITY OF A STRING-RELEASE-AFTER-COMPLETION MERGE-CONTROLLER—AN EXAMPLE

At this point the capacity of a ramp control system that releases a string of vehicles to attempt a merge into the freeway stream is considered. For this operational mode the number of vehicles in the string is dependent on the size of the detected gap, and release is conditional on the completion of successful merges by all previously released vehicles. Hence, the ramp is clear before an additional string is released.

Several cases considered for the controller design presented in the previous section included the following:

1. Two alternate length entrance ramps were measured in "seconds of time between the instant the control signal is turned to green and the instant the first merge-vehicle arrives at the merge zone"; thus $t_r = 5$ seconds and $t_r = 10$ seconds are used to investigate the effect of ramp length.

2. Four alternate gap acceptance criteria were employed to describe drivers that balk at the moving-merge attempt. Here $1/c$ equal to 1.5, 3.0, 4.5, and 6.0 seconds are respectively used for the mean gap sizes that drivers who stop at the merge zone find acceptable for a standing merge.

3. Five freeway volumes that correspond to q equal to 800, 1,100, 1,400, 1,700, and 2,000 vph were investigated to study the effects of variation in volume.

In addition, the following parameter values were employed as reasonable or typical or both:

1. The standing-merge gap acceptance probability is described as Erlang type 3, thus $r = 3$;

2. The moving-merge gap acceptance probability for the n th vehicle in a string is Erlang type 3, thus $s_n = 3$ for all n ;

3. The mean acceptable gap for the n th vehicle in a string is $1/b_n = 3n - 1.5$, thus the first, second, third, and n th vehicles accept a gap of 1.5, 4.5, 7.5, and $3n - 1.5$ seconds respectively on one-half of the merge attempts; and

4. The freeway intervehicular headway process is described by an Erlang type a distribution with mean equal to $1/q$ (where q is the volume), and here, a is the nearest integer value of the expression $a = 0.92 e^{3.6q}$ and is based on the experimental results obtained on the Gulf Freeway (2).

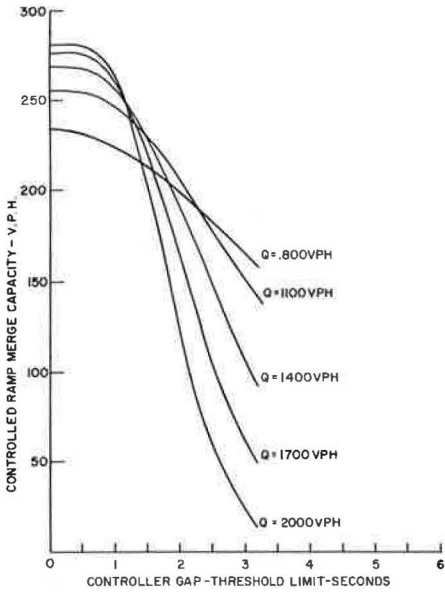


Figure 6. Single-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 1.5 seconds.

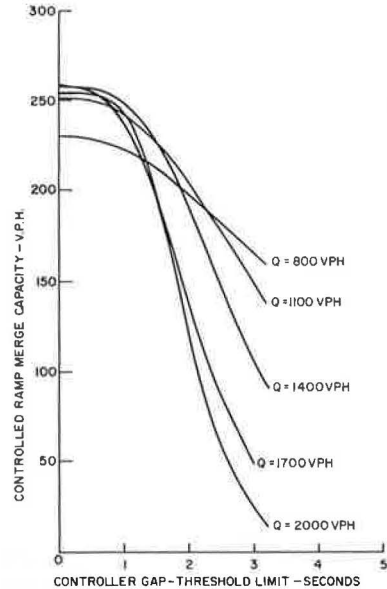


Figure 7. Single-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 3.0 seconds.

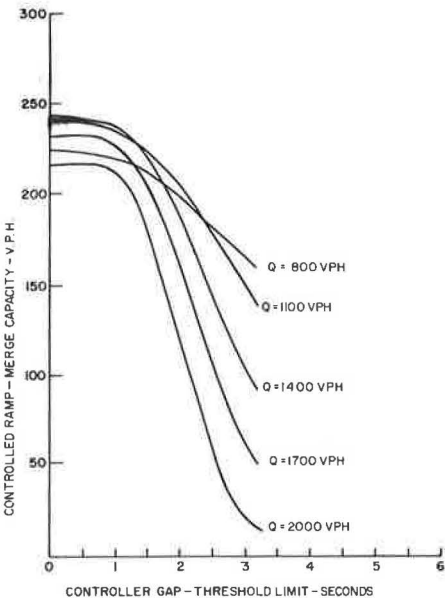


Figure 8. Single-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 4.5 seconds.

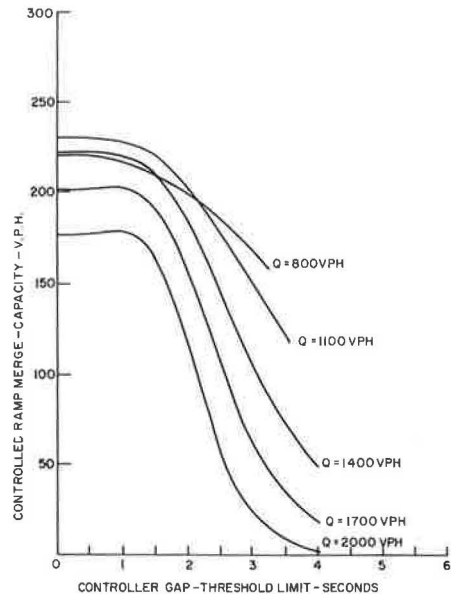


Figure 9. Single-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 6.0 seconds.

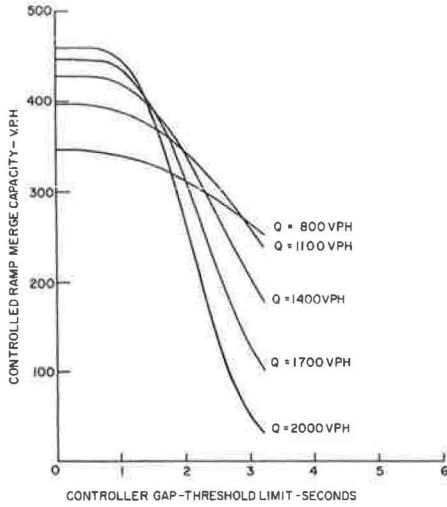


Figure 10. Single-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 1.5 seconds.

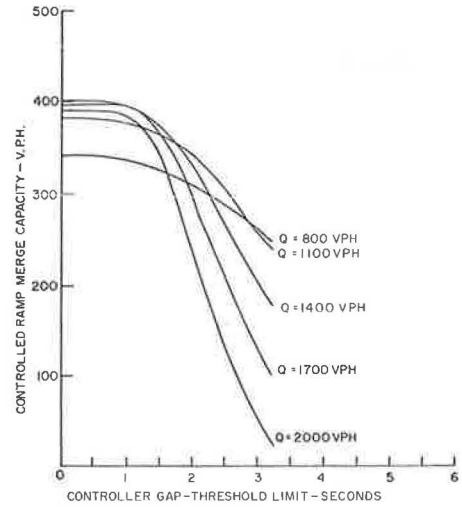


Figure 11. Single-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 3.0 seconds.

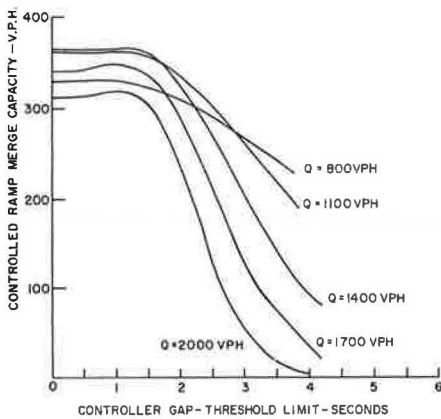


Figure 12. Single-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 4.5 seconds.

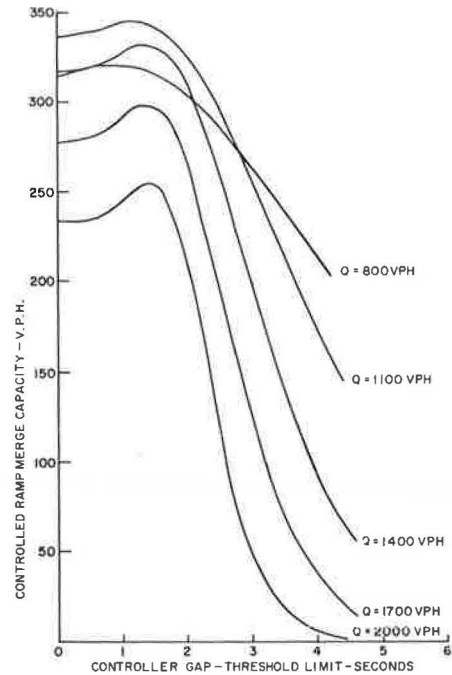


Figure 13. Single-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 6.0 seconds.

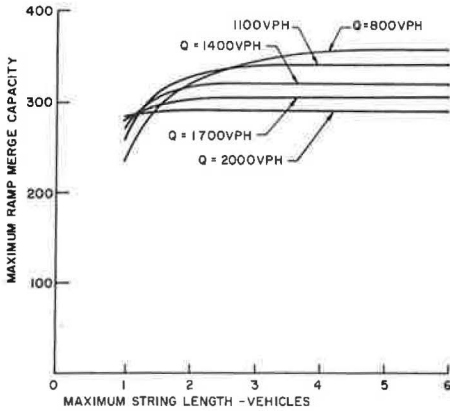


Figure 14. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge for n th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 1.5 seconds.

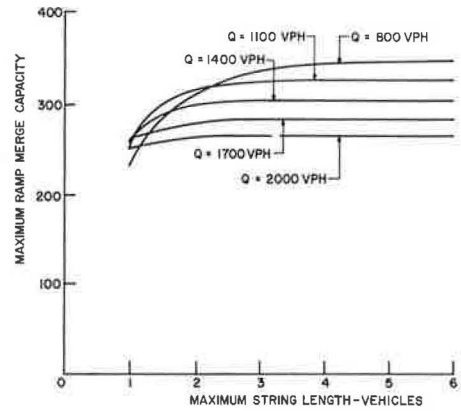


Figure 15. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge for n th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 3.0 seconds.

Before the results of the numerical analysis are discussed, 2 points are worth noting

1. When the time required by a vehicle to reach the merge zone is t_r seconds (after the signal is changed to green), then the maximum ramp volume in the single-vehicle-release-after-completion mode is $1/t_r$. This quantity equals 720 and 360 vph when the respective values of t_r are 5 and 10 seconds. When strings of n vehicles each are released in the release-after-completion mode and the assumption is used that intervehicular headway is n equal to 3 seconds, the corresponding merge rate is $1/(t_r + 3n - 3)$.

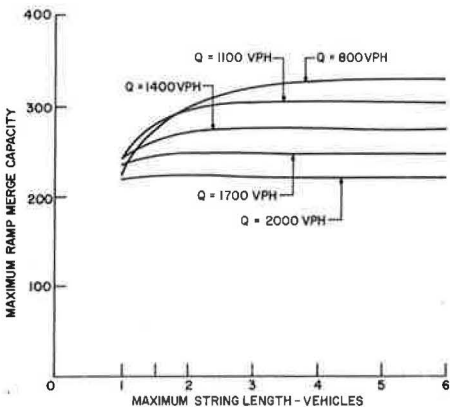


Figure 16. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge for n th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 4.5 seconds.

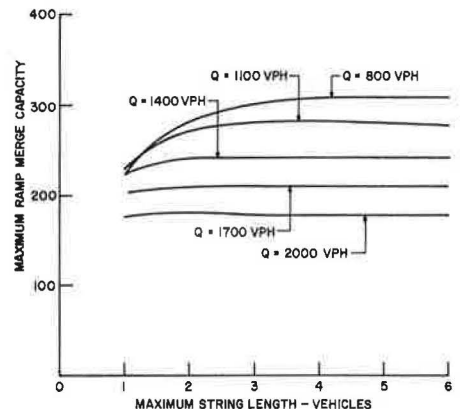


Figure 17. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge for n th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 6.0 seconds.

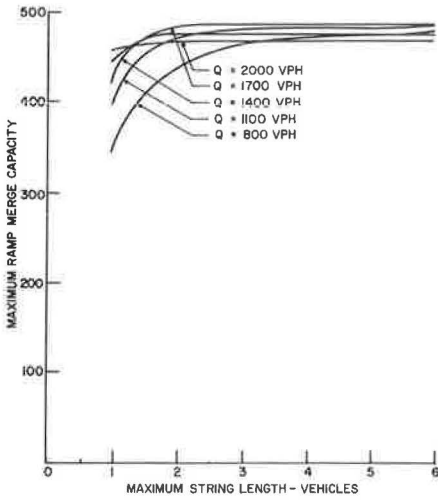


Figure 18. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge for n th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 1.5 seconds.

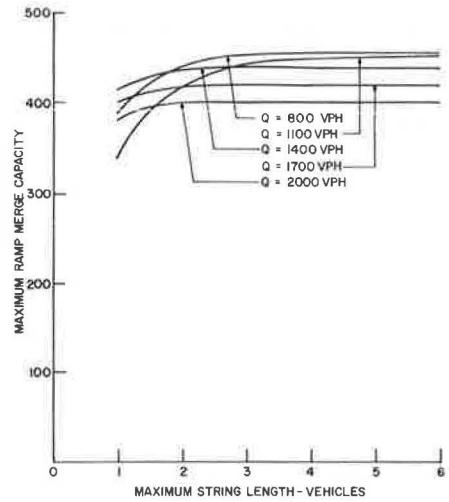


Figure 19. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge for n th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 3.0 seconds.

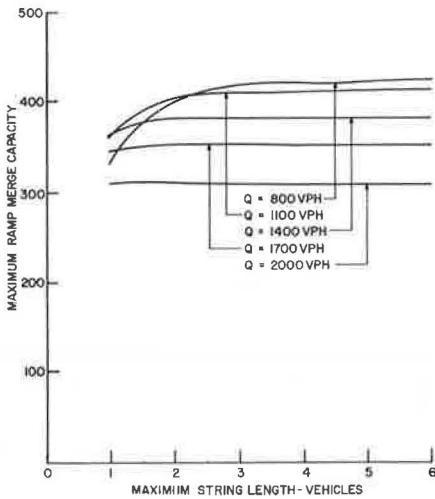


Figure 20. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge for n th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 4.5 seconds.

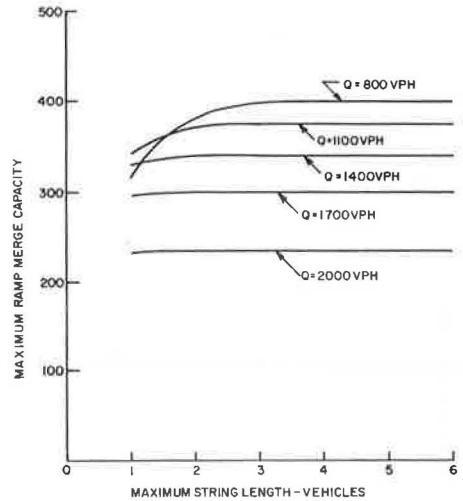


Figure 21. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge for n th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 6.0 seconds.

As n approaches infinity, this quantity approaches 1,200 vph for all values of t_r . This limit corresponds to the uncontrolled capacity limit of the entrance ramp.

2. The capacity of an entrance ramp in the release-after-completion mode depends on both the expected length and the expected frequency of the strings of vehicles that may be released to attempt a moving-merge into the freeway stream. For such operation, an increase in the freeway volume always results in a decrease in the number of vehicles in a released string. The expected number of released strings (i.e., the expected frequency), however, may either increase or decrease with an increase in freeway volume because this quantity depends on both the number and the suitability of freeway gaps. In particular, for low freeway volumes almost all gaps are suitable for a merge attempt by 1 vehicle, and an increase in the number of gaps implies an increase in the allowable number of released strings; however, for high freeway volumes only a portion of the gaps are suitable for merge attempts, and an increase in the number of gaps implies both that the average gap becomes smaller and that the number of gaps suitable for merge attempts decreases.

The use of parameter values previously listed in the models that were developed in the prior section makes it possible to numerically evaluate representative entrance ramp capacities. The results of such numerical evaluation with a string-release-after-completion merge controller are presented graphically for ease of interpretation. In Figures 6 through 13, the ramp capacity is shown as a function of threshold setting for the single-vehicle-release-after-completion mode, with volume as a parameter. These curves show that a threshold of approximately 1 second yields a generally near-maximum capacity for all cases. However, the sensitivity of capacity to threshold setting varies substantially. The short ramps (i.e., $t_r = 5$ seconds) tend to have higher capacities and greater sensitivity; the $t_r = 10$ second cases have lower rates and are less sensitive to variations in threshold. Likewise, there is a functional dependence of ramp capacity on the nature of the merge zone. For short ramps, the ease of execution of standing merges substantially affects the ramp capacity, but on long ramps this quantity is less significant.

More exactly, the numerically evaluated results indicate the following:

1. As the threshold is increased the occurrence of balks becomes less significant, and the capacities are asymptotically identical (i.e., independent of c for a given q and t_r).

2. The quality of service is improved at the expense of added delay when the threshold is increased, and the expected service time for a given threshold T_e is simply the reciprocal of the associated ramp capacity.

3. For all cases investigated, the largest ramp capacity for a single-vehicle-release-after-completion controller corresponds to a short ramp with an excellent merge zone (i.e., $t_r = 5$ and $1/c = 1.5$ seconds). Here, a ramp capacity of approximately 450 vph is indicated for a heavy freeway flow condition (i.e., q of 1,700 to 2,000 vph), but actual service must be suitably restricted below this value to prevent a breakdown of the stream.

4. The differences between the ramp capacities shown by the curves in Figures 6 through 13 and the maximum capacities of 360 and 720 vph that correspond to $t_r = 10$ and 5 seconds respectively are attributed to the combined effects of limiting by the threshold setting and by gap rejection (i.e., by both moving and stopped vehicles).

The maximum ramp capacity available in the string-release-after-completion mode has been evaluated by a sequential search, subject to the restriction that a second vehicle is not released unless a gap of 3 seconds or larger is detected (i.e., because the headway $h = 3$ seconds). Similarly, the release of a third vehicle required a gap of at least 6 seconds, the release of a fourth vehicle required a minimum gap of 9 seconds, and so on. The resultant ramp capacities are shown as a function of maximum string lengths in Figures 14 through 21 with freeway volume as a parameter and for various values of t_r and c . These curves show that the largest benefit of multiple vehicle release occurs at low freeway volumes; at high volumes, little extra capacity is gained. Hence, the largest volume available for the cases investigated is approximately 480 vph when $t_r = 5$ seconds, $1/c = 1.5$ seconds, and $q = 800$ vph. This is substantially below the limit of 1,200 vph that corresponds to $q = 0$. However, when the case of $t_r = 10$ seconds and $1/c = 3$ seconds is considered as an example, it is observed that, when the freeway volume gradually drops from 2,000 to 800 vph, the single-vehicle-merge ramp

capacity remains almost constant at approximately 250 vph and the string-release mode yields an automatic increase in capacity up to 350 vph. Finally, it is noted that, when a ramp capacity of 400 vph is required for this ramp, a reduction in t_r from 10 seconds to 5 seconds yields a string-release capacity in excess of 400 vph. However, when a ramp capacity of 500 vph is needed, use of the string-release-after-completion mode is unacceptable and release-before-completion must be implemented. This mode of operation is considered in the next section.

One final point remaining with regard to the numerical results is that the parameter values were chosen on the basis of reasonableness in order to yield typical results for presentation. The models developed are quite general and may be applied to alternate situations by the proper selection of parameter values.

THE RELEASE-BEFORE-COMPLETION MODE

Although the release-after-completion mode exhibits many desirable attributes, such as high safety level and high quality of merge service, the question of whether to release an additional vehicle while previously released vehicles remain on the ramp requires consideration. In particular, when a possible merge opportunity arises for a vehicle awaiting service behind the ramp control signal, the expected service delay for that vehicle may be significantly reduced when that vehicle is released without awaiting merge-completion for the previously released vehicle. However, when this operational mode is to be employed, the question of what value of gap threshold to use must be reexamined.

When a possible merge opportunity is detected in the freeway traffic stream before all previously released vehicles have merged, the question of whether to release the next queued vehicle can be converted into a question of which action minimizes the expected delay. Here the expected delay subject to release of the vehicle must be compared with the expected delay subject to nonrelease to determine which decision to implement. The resultant analysis yields a controller that is a sequential decision-maker in which both the target gap sizes and present state of vehicles on the ramp affect the decision process.

Analysis of the operation of an entrance ramp in the release-before-completion mode can take several forms. One possibility is to assume that the actual ramp service rate is restricted to a value that does not cause a breakdown in the flow of vehicles on the freeway. Then, an investigation of expected delay for the next vehicle awaiting service simplifies to a question of which decision yields the earliest expected merge instant. For this case, the following points are noted:

1. There are n unmerged vehicles on the ramp;
2. The last previous release of a vehicle occurred at $t = t_2$;
3. The target gap sizes are $\tau_1, \tau_2, \dots, \tau_n$ respectively;
4. A target gap of τ_{n+1} is detected at $T = t_d$;
5. The expected delay between the completion of standing merges is $1/\mu_S$;
6. The probability that the next vehicle can complete a moving merge into the pro-

spective target is $\prod_{i=1}^{n+1} P_a(\tau_i)$; and

7. The expected delay before a successful merge, given the merge zone is cleared, is $t_r + 1/\mu''$ (i. e., μ'' is the merge rate when $t_r = 0$).

Based on these definitions, the expected additional time required to complete the merge of the $(n + 1)$ th vehicle (i. e., beyond the time required to complete the merge of the n th vehicle and provided that no balk has occurred) is

$$\Delta t_y = \left[1 - \prod_{i=1}^{n+1} P_a(\tau_i) \right] (1/\mu_S) + \left[\prod_{i=1}^n P_a(\tau_i) \right] [t_d - t_l] \quad (30)$$

when the vehicle is released to attempt entry into the τ_{n+1} gap, and

$$\Delta t_n = t_r + 1/\mu'' \quad (31)$$

when release is inhibited until the ramp is clear.

Comparison of these 2 quantities reveals that when the gap τ_{n+1} satisfies the inequality

$$\begin{aligned} P_a(\tau_{n+1}) &\geq [(t_d - t_l)\mu_S - 1] \prod_{i=1}^n P_a(\tau_i) - [\mu_S t_r + (\mu_S/\mu'') - 1] \\ &\geq f[\mu_S, \mu'', t_r, t_d, \tau_1, \tau_2, \dots, \tau_n] \end{aligned} \quad (32)$$

the $(n + 1)$ th vehicle suffers less delay in the merge process by not waiting for the ramp to clear. Thus for the operational condition in which the function $f(\cdot)$ on the right side of the inequality is negative, the inequality is always satisfied, and any value of τ_{n+1} is acceptable for a merge attempt. Similarly, when $f(\cdot)$ exceeds unity, no gap is acceptable because $P_a(\tau_{n+1})$ is a gap-acceptance probability and cannot exceed unity. Finally, when $f(\cdot)$ is between 0 and 1, a threshold T_{n+1} exists such that

$$T_{n+1} = P_a^{-1} [f(\cdot)] \quad (33)$$

where $P_a^{-1}(\cdot)$ is the inverse of the gap acceptance probability function and $P_a^{-1} [P_a(t)] = t$.

For a ramp that has experienced a stoppage and offers zero probability of a moving merge for the $(n + 1)$ th vehicle, the expected delays associated with the release or the nonrelease of that vehicle at t_d are respectively

$$\Delta t'_y = 1/\mu_S \quad (34)$$

and

$$\Delta t'_n = t_r + 1/\mu \quad (35)$$

Comparison of these quantities indicates that, for any ramp on which the expected travel time t_r exceeds the difference between the expected delay $1/\mu_S$ (associated with a standing merge from the merge zone) and the travel-free service time $1/\mu''$ (associated with the successful clear-ramp, single-vehicle-merge rate), the release of the next vehicle into any size gap is warranted. Setting the probability of a moving merge for the $(n + 1)$ th vehicle in the previous case equal to 0 reinforces this conclusion; this is exactly the same criterion as that for release emerges.

The Design of a Controller for the Release-Before-Completion Mode

Under those operational conditions in which an increase becomes necessary or desirable in the production of a particular entrance ramp above the level achieved in the release-after-completion mode, it can be achieved by operation in the release-before-completion mode. To do so, however, involves a trade-off between the quality of merge-service offered and the merge production.

For a long ramp (i. e., t_r is large) with a merge zone that is unobstructed so that the expected standing merge delay is comparable to (though more than) the expected delay in the attempt of a moving merge, analysis has shown that the release-before-completion mode offers an improved merge rate for the ramp, provided that the freeway stream does not break down. This property allows for a variation of the controller threshold (or merge-rate limit or both) to increase the ramp production as required within the limits necessary to preserve the stability of the stream. Such operation has been implemented on the Gulf Freeway when queue lengths beyond the available storage capacity have made action necessary. In addition, ramp operation with a fixed threshold in the release-before-completion mode has been more the rule than the exception on that Freeway.

By contrast, when a ramp is short (i. e., t_r is small) and the merge zone is marginal so that standing merge opportunities are rare compared with moving merge opportunities,

operation in the release-before-completion mode does not always yield an increase in ramp merge capacity. In particular, analysis reveals that when

$$t_r < (1/\mu_S) - (1/\mu''') \quad (36)$$

a stoppage of a released vehicle warrants that the release-before-completion operation be inhibited until the ramp is cleared. The existence of this restriction in the release-before-completion operation, however, is temporary and in no way negates the overall advantage of increased ramp capacity that results, provided that the controller selects only adequately large gaps that occur within an adequately short period after the last previous release.

SUMMARY

Under either control philosophy described, the threshold that yields the maximum ramp capacity must be identified first and, based on that value, a threshold setting is established that both provides an appropriate level of merge service and preserves the stability of the freeway stream against breakdown in flow caused by excess density. By this process, merge capacity will often be reduced below its maximum value to increase the level of service offered to drivers. Thus, the controller may attempt to operate in a mode in which a single vehicle is released whenever the ramp is clear, the demand is low, and the probability of a successful moving merge is adequately high. When it is necessary to increase the available ramp merge capacity, the operational mode is modified so as to reduce the level of service by the relaxation of control limits until adequate merge capacity is achieved. In this way the controller is continuously adjustable to both actual demand and actual freeway volume.

Based on analysis of the results obtained by use of the control modes developed in this paper, we believe a sound analytic theory exists for the design of freeway entrance ramp control systems. In particular, the relationship between merge capacity and system parameters is explicitly developed for several control modes. These results make possible the evaluation of both the capacity limitations inherent in the design of a particular ramp control system and the control system parameters necessary to provide a specified merge capacity for the ramp.

To aid in the design process, several typical ramp control configurations were analyzed in detail. These included the following conditions: (a) placement of a traffic control signal at between 5 and 10 seconds of travel time from the merge zone, (b) mean gap acceptance limits for vehicles stopped in merge zone of between 1.5 and 6.0 seconds, and (c) freeway volumes in the outside lane of between 800 and 2,000 vph. As a result of this analysis, the following observations are made:

1. The gap rejection phenomenon was found to increase the minimum expected ramp service time, in the single-vehicle-release-after-completion mode, to a level between 1.25 and 3.0 times the expected vehicular travel time on that ramp. For example, for a long ramp with an expected travel time of 10 seconds, the maximum ramp capacity is reduced to between 50 and 80 percent of the 360-vph limit that the 10-second travel time imposes on this mode; 3,600 seconds per hour divided by a minimum of 10 seconds per vehicle yields a maximum flow in the single-vehicle-release-after-completion mode of 360 vph. Here, the larger capacity corresponds to a volume of 800 vph in the outside freeway lane; the lower capacity occurred with a freeway volume of 2,000. In both cases, the assumption is that a relatively good merge zone exists because the mean acceptable gap for a standing merge from the merge zone was defined to equal 1.5 seconds. For a second example, with an expected travel time of 5 seconds and a short merge zone, the results indicate a reduction in maximum ramp capacity to between 35 and 65 percent of the limit of 720 vph that the 5-second travel time imposes on this mode. Again, capacity depended on freeway volume.

2. The release of more than 1 vehicle per string can increase the ramp capacity without forsaking the release-after-completion operational mode. For such operation, multiple-vehicle-release opportunities are rare when high freeway volumes exist and

little is gained in these instances. However, when freeway volumes are low (e.g., 800 vph per lane), improvements of approximately 50 percent over single-vehicle operation are possible.

3. Operation in the string-release-after-completion mode appears to be limited at most to between 15 and 40 percent of the uncontrolled ramp capacity, with the higher production generally corresponding to the lower freeway volumes (i.e., 800 vph per lane). To provide ramp capacities in excess of this limit requires that release-before-completion operation be employed.

4. When the difference between the expected service time required to complete a standing merge from the merge zone is less than the total expected service time required to complete a moving merge from behind the ramp control signal (i.e., including the expected travel time), then the release-before-completion mode exhibits a maximum ramp capacity that equals the uncontrolled ramp capacity. However, the actual service provided by this controller must be constrained by consideration of freeway stream stability and actual demand characteristics. For example, metering at any fixed rate below 1,200 vph would be possible for several of the cases investigated in this report, provided that stream stability was not a problem.

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Study of Rural Freeway Emergency Communications for Stranded Motorists

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This report presents further information on the operation of a motorist-aid telephone system on a rural freeway. Stranded motorists' needs and the ways in which these needs are met are examined. Data from observations made during the summer of 1968 and the winter of 1969 indicate about 50 percent of drivers who needed aid used the motorist-aid telephones. The rate for vehicle stops of 12 min or longer was 1 stop in 26 miles each 66 min in the summer survey and 1 stop each 99 min in the winter survey. This is consistent with other reported findings that show greater assistance needs in summer. Assistance rates may correlate directly to traffic volumes, but these data have not been fully analyzed. The percentage of trucks requiring assistance is consistently large compared to their percentage of the traffic stream. Of those that stopped 12 min or longer, 48 percent were serviced by the truck drivers or the drivers were assisted by passing drivers. Drivers of passenger cars requiring mechanical aid are most likely to phone for aid. A comparison of the data from the control section on US-23 to those from the study section on I-94 shows that trucks on US-23, at least, would have greatly reduced their stopped time had aid phones been available to them. The surveys of motorists passing through the motorist-aid phone area on I-94 showed approximately 87 percent favoring such a system. Seventy-five percent of the users of the aid phones travel the study section once a month or more often and, not surprisingly, 97.4 percent of those who have used the phones think the system should be expanded.

*THIS IS the second interim report of a study to determine the usefulness of a motorist-aid telephone system to motorists stranded on a rural freeway and to observe the needs of these motorists. Presented and reviewed are available data to May 1969 from a 2-month summer survey in July and August 1968 and a winter survey in January 1969, both of which included mobile and stationary observations of stranded motorists, interviews with stranded motorists, and a handout questionnaire survey. Included also is a summary of system operations and a partial review of maintenance activities.

DESCRIPTION OF MOTORIST-AID TELEPHONE SYSTEM

A 30-mile study section on I-94, between Jackson and Battle Creek, was selected for the experiment (Figs. 1 and 2). I-94 is a major east-west freeway connecting Detroit to Chicago. About 21 percent of its 14,000 average annual daily traffic is commercial.

The motorist-aid telephone system is entirely state-owned with the exception of the leased lines connecting the freeway circuits to the State Police posts. These lines are



Figure 1. Location of study section on I-94 and control section on US-23.

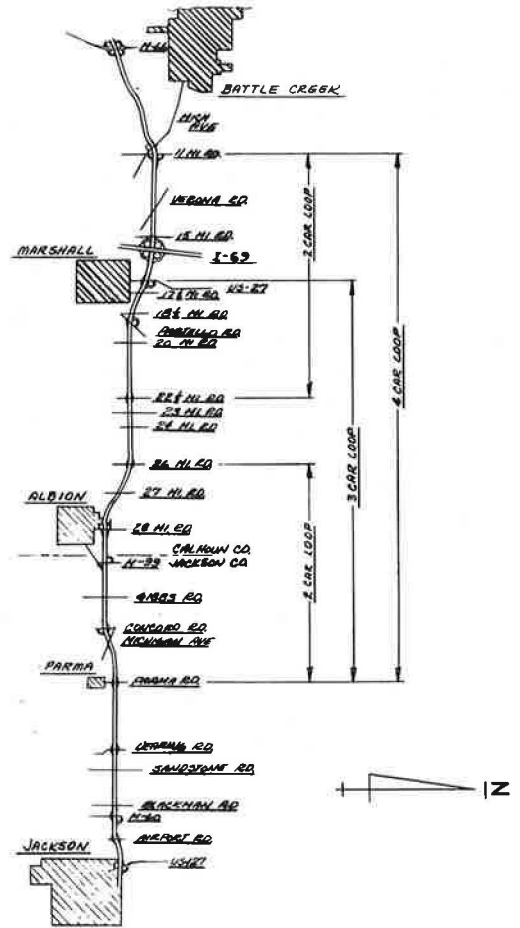


Figure 2. Location of motorist-aid phones on I-94 Study Section.

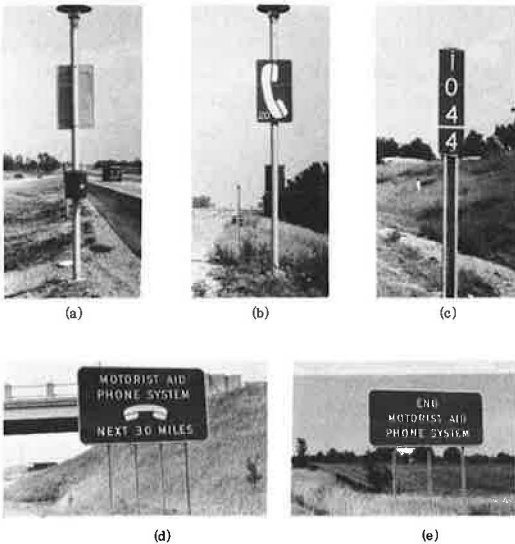


Figure 3. Installation and signing of motorist-aid phones.

nected to the Jackson State Police post, and Battle Creek State Police post.

Each motorist-aid telephone is 13.5 ft from the edge of the pavement. A red weather-proof cabinet containing the handset is attached on the downstream side of a 12-ft aluminum pole with a blue light on top. A blue sign displaying a white telephone symbol is also attached facing traffic (Fig. 3a and b).

All telephone and power cables within the right-of-way are underground. The 30-mile section is signed at the beginning, end, and midpoint (Fig. 3d and e). Mileage markers are placed every two-tenths mile along the study section to identify stopping location (Fig. 3c).

To use the telephones, the motorist opens the cabinet door and lifts the handset from the hook. The dispatcher at the police post is notified by a ring and a red light that

leased from the Michigan Bell Telephone Company. There are 31 pairs of phones over the 30 miles. The east 17 pairs of phones are spaced at approximately 3,400 ft and the remaining 14 pairs at approximately 5,400 ft. The phones are numbered 1 through 62; phones 1 through 28 are con-

phones 29 through 62 are connected to the

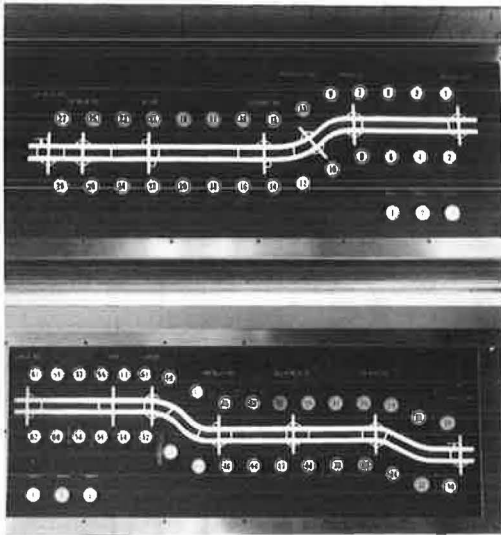


Figure 4. Call identification panels at State Police post.

down by a vehicle, 9 handsets or cord damages occurred, and 1 transformer was stolen. This repair activity, plus relamping and placement of a capacitor in each phone site, has kept one man busy full time troubleshooting and repairing. The new capacitor installations have eliminated more false ringing problems, many of which were apparently caused by power interruptions. Recently a building remodeling at the Battle Creek State Police post caused those circuits to be out of service for several days while the display panel was relocated.

Aside from these random problems, the system has been operating well. Weekly checks show that occasionally the State Police personnel are too busy with other duties to answer quickly; however, we have not heard recently that anyone tried to call and could not get an answer. A few returns of an earlier handout questionnaire contained comments of this type. It does appear that we are not receiving data on many of the calls for assistance when in fact assistance to the motorist is provided.

Plans have been made to interface a tape recorder with the system at each post. These recorders will be activated when the handsets at the posts are lifted for an incoming call. This addition should fill the gap in total information on call activity. Normal police post activity is sufficient to take most of the dispatchers' time and, thus, calls on the motorist-aid phones are usually handled as briefly as possible and perhaps at times are not recorded at all.

In any case, the system can still be of value even with some periods when a phone or circuit is temporarily inoperative. The forthcoming period of operation with the tape recorders should provide a broader base of information on the real use of the system.

MOBILE OBSERVATIONS OF STRANDED MOTORISTS

Mobile observations were made during the summer of 1968 and winter of 1969 on the study section and the control section on US-23. The 10-mile section on US-23 (Fig. 1) was chosen as a control to provide a comparison of stranded motorist activities on this section of roadway with those on I-94.

The mobile survey was designed to let the stopped motorist take some action before being interviewed. Four survey cars were equally spaced in a 26-mile loop from Parma Road to 11-Mile Road on I-94 (Fig. 2). The time interval between cars was 12 min; by definition, motorists stopped over 12 min were deemed stranded. The first survey car to spot a stopped vehicle reported to the next survey car in the loop by radio. If the

identifies the calling site (Fig. 4). The dispatcher answers the call and obtains information necessary to assist the motorist and completes a questionnaire (Fig. 21). The State Police usually supply gasoline to a motorist with this need. For other needs, the proper commercial agency is contacted to service the motorist.

SYSTEM OPERATION AND MAINTENANCE

Since the last interim report, the system has operated well, but there have been some periods when a phone or a circuit was inoperative. Operational problems still occur, ranging from lightning strikes, handsets or the roadside box being vandalized, leased line problems, vehicle damage to underground circuits, and short circuits in the terminal boxes caused by flooding.

Within a 6-month period, 6 to 8 phones or circuits were out of service because of lightning damage. One site was knocked

Date _____ Recorder _____	Route _____
TIME (00:00 - 2400)	REASON FOR STOP
_____ of Sighting Vehicle if Stopped	_____ Abandoned _____ Police Action
_____ of Stop	_____ Weather _____ Illness _____ Other _____
_____ of Call	CAR TROUBLE
_____ of Interview	_____ Tire _____ Gas _____ Oil
_____ of Police Arrival	_____ Mechanical _____ Stuck off Road
_____ of Other Aid _____	_____ Accident _____ Water _____ Fire
_____ of Stranded Vehicle Leaving _____	_____ Other _____
LOCATION: Milepost _____ W.D. _____ E.D. _____	DID YOU CALL FOR AID? _____ No _____ Yes Phone No. _____
VEHICLE:	NAME & ADDRESS
_____ Pass Car _____ With Trailer	_____ Mr. _____
_____ Truck Single Unit _____ Pickup or Panel	_____ Mrs. _____
_____ Truck Combination Units _____ Motorcycle	_____ Miss _____ Age _____
_____ Bus _____ Other _____	_____ Number & Street _____
REGISTRATION:	City _____ State _____ Zip _____
License No. _____	Where Are You Coming From? _____
_____ Local	Where Are You Going? _____
_____ Other in State	TRIP PURPOSE:
_____ Out of State	_____ Social and _____ to or from _____ Misc.
OCCUPANTS: Driver _____ Man _____ Woman	_____ Recreation _____ work _____
Passengers _____ Man _____ Woman _____ Child	_____ Shopping _____ Work _____ School
WEATHER:	HOW OFTEN DO YOU DRIVE THIS ROUTE?
_____ 75°+ _____ Sunny _____ Dry	Almost Every _____ Day _____ Week _____ Month
_____ 50°-75° _____ Cloudy _____ Wet	_____ Once or Twice a Year _____ Less than once a Year
_____ 30°-50° _____ Fog _____ Snow	WERE YOU AWARE OF THE PHONES BEFORE YOU STOPPED?
_____ 0°-30° _____ Rain _____ Icy	_____ No _____ Yes _____ How? _____
_____ Below 0° _____ Snow	COMMENTS: _____

Figure 5. Interview form used in stranded-vehicle summer and winter surveys.

stopped vehicle was still present when the next survey car approached, an interview was taken and recorded on the form shown in Figure 5. A fifth survey car took the stopped survey car's position in the loop. On the US-23 control section, there was 1 survey car whose driver stopped when he saw a stopped vehicle and waited at a distance for the stopped motorist to take action. When the motorist started remedial action, the survey car driver moved up and interviewed the stopped motorist. Obviously,

TABLE 1
REASONS FOR VEHICLE STOPS ON I-94 DURING 1968 SUMMER SURVEY

Reason	Passenger Cars			Other Vehicles ^a		
	Number	Percent	Used Aid Phones	Number	Percent	Used Aid Phones
Tire failure	57	48	10	19	36	7
Gas, water, or oil	21	18	4	6	11	1
Mechanical, tow	19	16	12	4	7	2
Mechanical, no tow	21	18	5	19	36	6
Accident	0	0	0	2	4	0
Fire	0	0	0	1	2	1
Miscellaneous	1	0	0	2	4	0
Total	119	100	31	53	100	17

^aIncludes 48 trucks, 3 buses, and 2 motorcycles.

TABLE 2
ACTIONS TAKEN BY STRANDED MOTORISTS TO
OBTAIN AID ON I-94 DURING 1968 SUMMER SURVEY

Action	Number	Percent
Off-freeway aid	90	52
Used aid phones ^a	48 ^b	28
Used public phones	4	2
Walked	14 ^c	8
Hitchhiked	18 ^d	11
Abandoned vehicle for over 10 hours	6	3
Miscellaneous	82	48
Self-help		
Tire	36 ^e	
Mechanical	9 ^f	
Used own radio	2	
Drove to service	8	
Others helped		
Survey group	7	
Passerby	11	
Police	1	
Unknown	8	
Total	172	100

^aOf those interviewed, 40 motorists or 23 percent were not aware of the aid phones.

^b52 percent of the 90 motorists needing off-freeway aid.

^c11 of these were aware of the aid phones.

^d15 of these were aware of the aid phones.

^e49 percent of the motorists with tire needs.

^f15 percent of the motorists with mechanical needs.

many stopped motorists were not interviewed because the survey car would occasionally be stopped for prolonged periods of time. Because of this, comparison section would be a better name for the control section.

On the I-94 study section, observations were made during the summer for 192 hours over 26 days during which 172 interviews were taken with motorists who were stopped for 12 min or longer. The results are given in Table 1. Passenger cars comprise 69 percent of the stranded vehicles; however, trucks at 31 percent have a disproportionate share of the breakdowns as they comprise only 21 percent of the traffic. Tire failure caused 48 percent of the passenger cars to stop. Mechanical aid was needed by 43 percent of the stopped trucks and other vehicles, and tire repairs by 36 percent.

Table 2 gives the action taken by the stranded motorists. Those in the others-helped category, under miscellaneous, are not combined with those needing off-freeway aid, as there will always be some

passing motorists willing to give aid. The 26 people who walked or hitchhiked, but were aware of the aid phones, apparently assumed that they could obtain aid either faster, or cheaper, themselves. It is rather puzzling, however, that 77 percent of the needy motorists were aware of the phones but only 52 percent made use of them.

Observations were made on I-94 during the winter for 71 hours over 10 days during which 36 interviews were taken with motorists stopped 12 min or longer. Results are given in Table 3. Although the total number of interviews is smaller, the percentage distribution of the reasons cars (69 percent) and trucks (31 percent) stopped during the winter survey is the same as that of the summer survey. Trucks had a disproportionate share of the breakdowns. The changes in the percentages in Table 3 can be attributed largely to the cooler weather and correspondingly fewer tire failures. Trucks obviously have more mechanical needs in winter; however, gas, water, and oil needs were not evident in winter for trucks. Some of these variations may also be the result of the smaller winter sample. Table 4 gives the actions taken by drivers during the winter survey to meet their various needs when their travel was interrupted. More stopped winter drivers (89 percent) than summer drivers were aware of the aid phones; however, only 50 percent chose to utilize them.

TABLE 3
REASONS FOR VEHICLE STOPS ON I-94 DURING 1969 WINTER SURVEY

Reason	Passenger Cars			Trucks		
	Number	Percent	Used Aid Phones	Number	Percent	Used Aid Phones
Tire failure	6	24	1	3	27	1
Gas, water, or oil	4	16	1	0	0	0
Mechanical, tow	4	16	3	7	64	3
Mechanical, no tow	3	12	1	1	9	1
Accident	3	12	3	0	0	0
Fire	1	4	1	0	0	0
Miscellaneous	4	16	0	0	0	0
Total	25	100	10	11	100	5

TABLE 4

ACTIONS TAKEN BY STRANDED MOTORISTS TO OBTAIN AID ON I-94 DURING 1969 WINTER SURVEY

Action	Number		Percent	
Off-freeway aid		30		83
Used aid phones ^a	12 ^b		33	
Other motorists used aid phones	3 ^b		8	
Other motorists used public phones	1		3	
Walked	4		11	
Hitchhiked	6		17	
Abandoned vehicle for over 10 hours	4		11	
Miscellaneous		6		17
Self-help				
Tire	3			
Others helped				
Passerby	1			
Police	1			
Unknown	1			
Total		36		100

^aOf those interviewed, 4 motorists or 11 percent were not aware of the aid phones.

^b50 percent of the 30 motorists needing off-freeway aid.

TABLE 5

REASONS FOR VEHICLE STOPS ON US-23 DURING SUMMER AND WINTER SURVEYS

Reason	Passenger Cars		Trucks	
	Number	Percent	Number	Percent
Summer survey				
Tire failure	17	38	3	17
Gas, water, or oil	12	26	3	17
Mechanical, tow	5	11	3	17
Mechanical, no tow	5	11	7	38
Accident	2	5	0	0
Stuck off road	0	0	0	0
Miscellaneous	4	9	2	11
Total	45	100	18	100
Winter survey				
Tire failure	6	35	5	28
Gas, water, or oil	0	0	4	22
Mechanical, tow	1	6	2	11
Mechanical, no tow	3	18	3	17
Accident	2	12	0	0
Stuck off road	0	0	2	11
Miscellaneous	5	29	2	11
Total	17	100	18	100

Data from the survey on US-23 are given in Table 5. There are no aid phones on this control section, and 63 interviews or observations of vehicles stopped for 12 min or longer were made during the summer and 35 during the winter. The number obtained on US-23 during the winter is close to the number (36) obtained during the winter survey on I-94 and is difficult to explain because of the method of survey. Although the total number of interviews is small, trucks had a larger share of the needs. They make up 17 percent of the traffic but 29 percent of the summer stops and 51 percent of the winter stops. The major problem for passenger cars in both winter and summer is tire failure, and that for trucks in the summer is mechanical and in the winter, tire failure. Tire problems in winter are rather surprising inasmuch as tire failures are usually reduced in cold weather.

Table 6 gives the actions taken by motorists to obtain aid. The lack of aid phones forced 48 percent of the motorists on US-23 to leave the freeway to obtain aid; only 24 percent of the motorists on I-94 left the freeway to obtain aid. The group leaving the

TABLE 6

ACTIONS TAKEN BY STRANDED MOTORISTS TO OBTAIN AID ON US-23 DURING SUMMER AND WINTER SURVEYS

Action	Summer Motorists		Winter Motorists	
	Number	Percent	Number	Percent
Used public phones	6	10	1	5
Walked	14	22	3	8
Hitchhiked	6	10	0	0
Abandoned vehicle for over 10 hours	4	6	4	12
Miscellaneous	33	52	24	75
Self-help				
Tire	10	30	4	17
Mechanical	8	24	4	17
Drove to service	0	0	2	8
Others helped				
Survey group	3	9	7	29
Passerby	9	27	4	17
Police	1	3	1	4
Unknown	2	7	2	8
Total	63	100	32	100

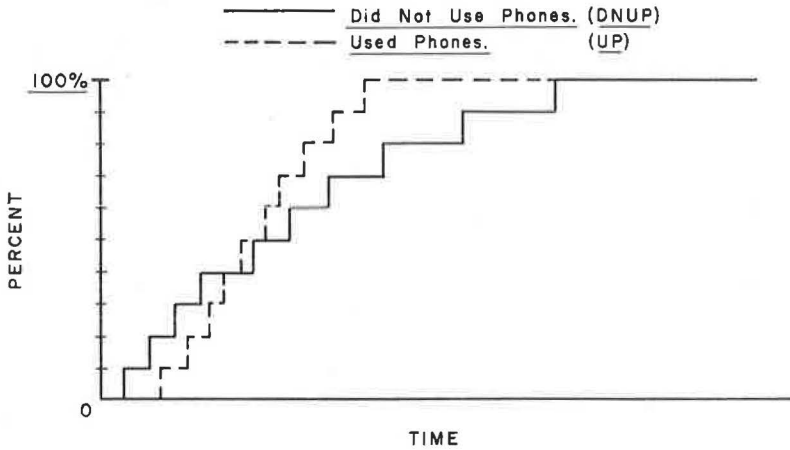


Figure 6. Ideal cumulative function.

freeway were considered to be those who used public phone, walked, hitchhiked, and abandoned vehicle. These varying percentages may not be entirely realistic because of the small numbers involved; however, clearly demonstrated is the forced reliance on self-help and help from passing motorists. One-third or more of the stranded motorists received aid from other motorists.

LEVELS OF SERVICE TO STRANDED MOTORISTS IN TERMS OF AID TIME

Figure 6 shows an ideal cumulative distribution function for levels of service in which causative factors are related to total time required for the stranded motorists to obtain aid. The first characteristic is that the did-not-use-phone (DNUP) line begins to increase to the left of the used-phone (UP) line. The justification for this is that motorists who are relatively fortunate to be disabled near an intersection with service, or otherwise are able to obtain aid immediately, will realize less delay than most phone users. Thus, there may well be this small privileged class when the system is in the ideal state. The second characteristic is that the UP increased at a rate greater than that of the DNUP. The justification is that the use of the phone should initiate a chain of communications and service links that is reasonably uniform in its capability to aid the stranded motorist. Thus, the spread of time over which aid is given should be less for the phone user than for the individual who does not use the phone. The third characteristic is that the UP obtains 100 percent to the left of the DNUP. The justification is that the use of the phone should ensure that a motorist is not stranded for an extreme length of time. The person who does not use the phone does not have this assurance. In summary, the greater percentages of the DNUP during the excellent level of service times might be anticipated and do not indicate a defect in the system. However, the UP should quickly overcome this advantage and reach 100 percent without long flat periods of time as might also be anticipated for the DNUP. The empirical cumulative distribution functions are shown in Figures 7 through 19. The differences in the number of vehicles shown in these figures and that given in earlier tables is due to the availability of aid-time data.

Summer Survey on Study Section

Tire aid was required by 56 stranded passenger cars. Of these, 36 (64 percent) helped themselves and 20 (36 percent) received help from others. Among these 20, 9 (45 percent) used the phones. Figure 7 shows that there is little difference in the aid time of those who used the phones and the aid time of those who did not. More than 80 percent of the group took less than 1 hour to fix their tires.

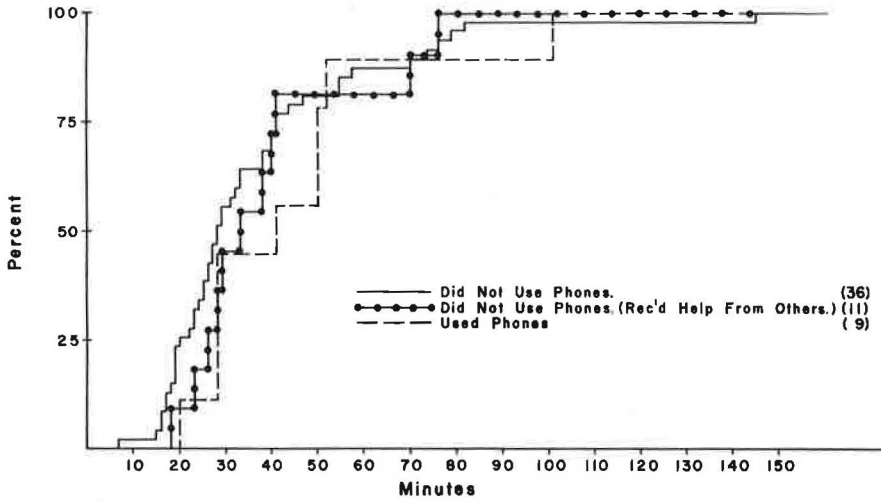


Figure 7. Aid time for passenger cars with tire failure on I-94 during 1968 summer survey.

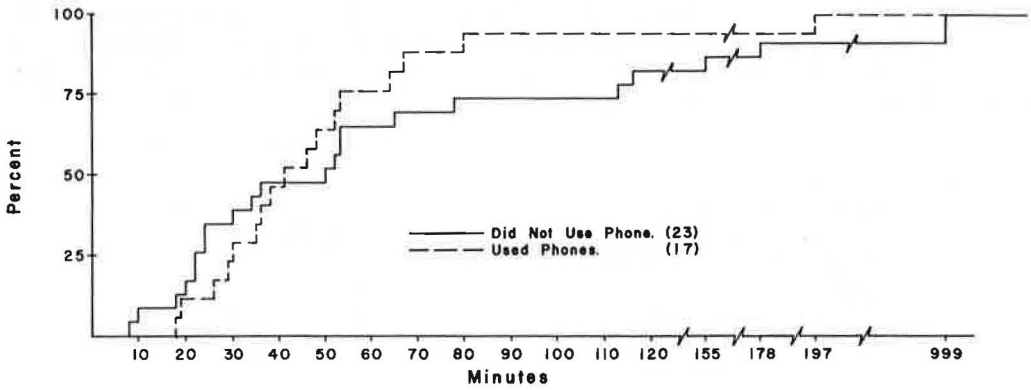


Figure 8. Aid time for passenger cars with mechanical failure on I-94 during 1968 summer survey.

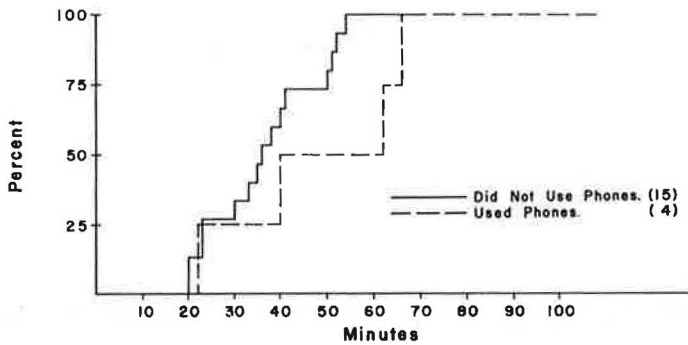


Figure 9. Aid time for passenger cars needing water, gas, or oil on I-94 during 1968 summer survey.

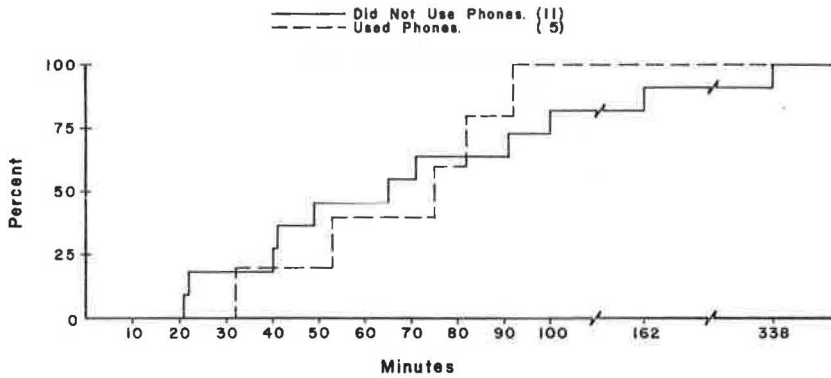


Figure 10. Aid time for trucks with tire failure on I-94 during 1968 summer survey.

Forty passenger cars required mechanical aid. Nine (23 percent) of the drivers helped themselves, and 31 (77 percent) received outside help. Of the 31, 17 (55 percent) used the phones. The cumulative curve shown in Figure 8 is very close to the ideal curve; it shows that a high percentage (77 percent) of the passenger cars with mechanical trouble needed outside help and the phones provided more than half of this assistance.

Nineteen passenger cars required gas, water, or oil (Fig. 9). Four of the drivers used the phones. One of these received help from the survey group in 22 min, but another hitchhiked to get gas and took 62 min. The other 2 who called waited 40 to 66 min to get help from the State Police. The 15 who did not call received help as follows:

	<u>Time Required, min</u>
Survey group	23, 25
Other motorists	23, 33, 38, 50
Walked to the nearest gas station	30, 40, 51
Walked to rest area for water	41
Self-help	36
Unknown assistance	20, 20, 52, 54

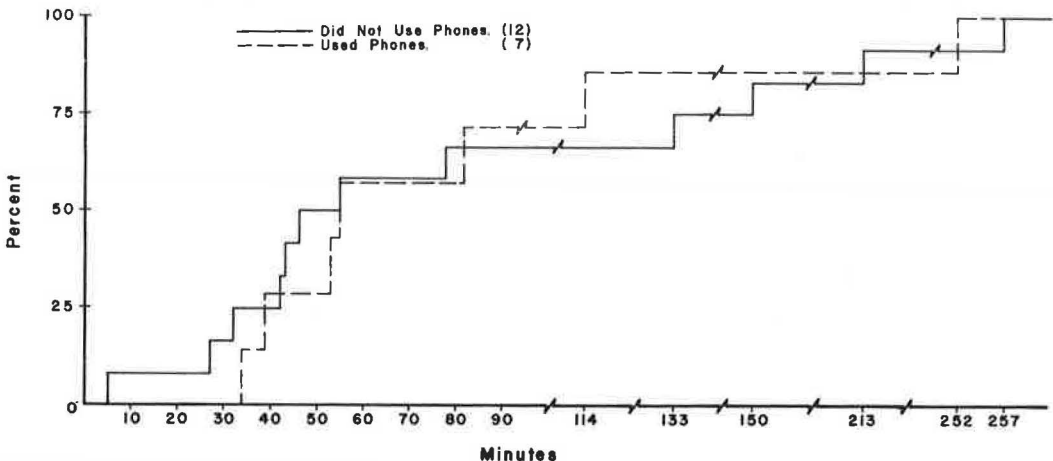


Figure 11. Aid time for trucks with mechanical failure on I-94 during 1968 summer survey.

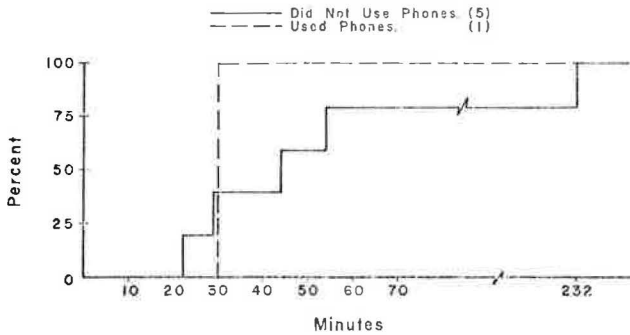


Figure 12. Aid time for trucks needing water, gas, or oil on I-94 during 1968 summer survey.

Fourteen of these 15 drivers were aware of the aid phones before they tried to get help. It is obvious that, to obtain this category of services, drivers will do many things rather than use the aid phones, even though they are well aware that the phones are available. The reason for this reluctance can only be guessed because these motorists were not asked this specific question.

Tire aid was required by 16 trucks. The drivers of 5 (31 percent) used the phones and the drivers of 11 (69 percent) did not (Fig. 10). Of these 11, 4 helped themselves, 1 used the

pay phone, 1 received help from his own company by using his truck phone, 4 drove to the gas station, and 1 received help from another truck.

Nineteen trucks required mechanical aid. Seven drivers (37 percent) used the phones (Fig. 11). There is little difference in the aid time of those who used the phones and those who did not.

The time distribution of the trucks that required gas, water, or oil is very close to that of the ideal curve (Fig. 12). The sample size, however, is very small.

Winter Survey on Study Section

Drivers of 36 stopped vehicles were interviewed during the winter survey. Of these, 15 used the phones and 21 did not. The aid times for the passenger cars are shown in Figures 13, 14, and 15, and for the trucks in Figures 16 and 17. The sample sizes are small; however, they show promise of being close to the ideal as the sample size increases.

Surveys on Control Section

Figures 18 and 19 show the aid-time distributions for the 3 most common needs of stopped vehicles on the control section on US-23. The short time grouping for the passenger cars apparently was a result of 2 factors: The sample is limited, and the percentage of vehicles on US-23 aided by other motorists was twice that on I-94. Hence, some of these short aid times may well have been 2 or 3 hours under other circumstances.

The stranded-truck activity shows a marked reduction in delay in all categories for I-94 trucks whose drivers used the phones compared to the US-23 drivers (Fig. 19) who had to obtain aid by other means. In both of these distributions, the sample sizes are rather limited.

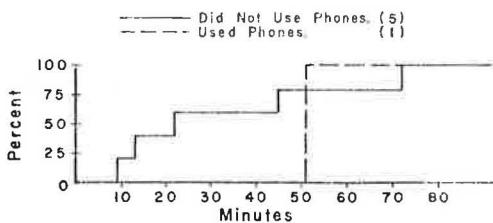


Figure 13. Aid time for passenger cars with tire failure on I-94 during 1969 winter survey.

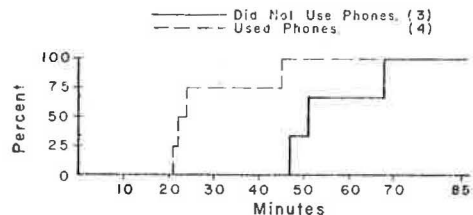


Figure 14. Aid time for passenger cars with mechanical failure on I-94 during 1969 winter survey.

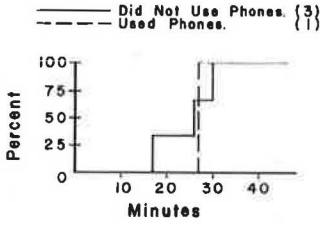


Figure 15. Aid time for passenger cars needing water, gas, or oil on I-94 during 1969 winter survey.

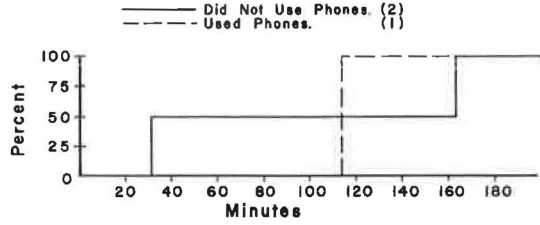


Figure 16. Aid time for trucks with tire failure on I-94 during 1969 winter survey.

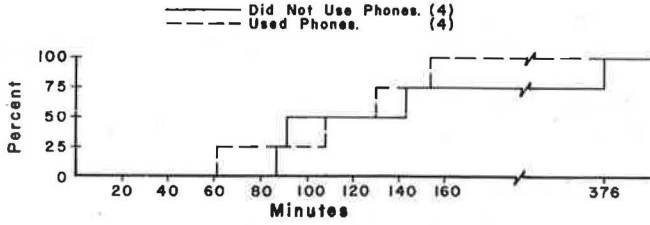


Figure 17. Aid time for trucks with mechanical failure on I-94 during 1969 winter survey.

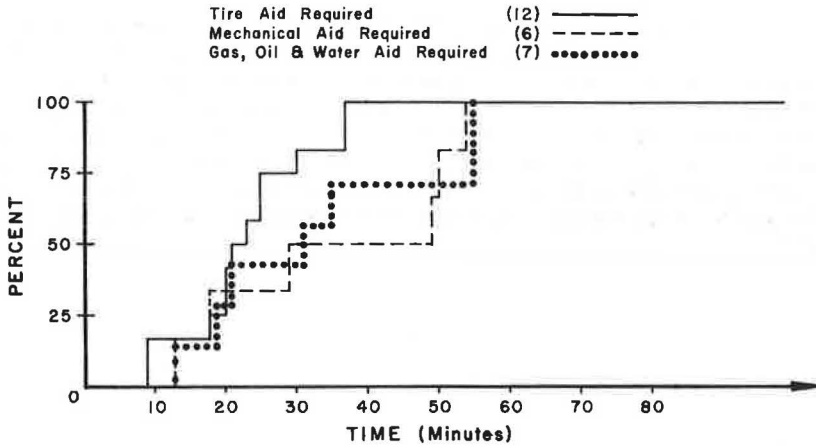


Figure 18. Aid time for passenger cars on US-23 during summer survey.

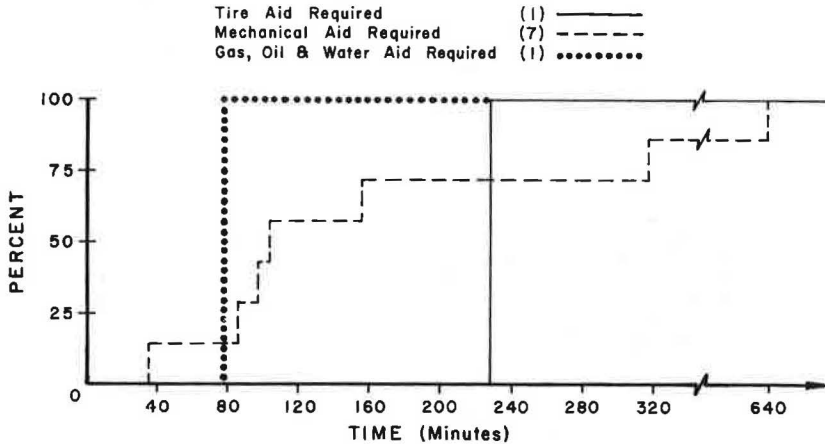


Figure 19. Aid time for trucks on US-23 during summer survey.

ANALYSIS OF HANDOUT QUESTIONNAIRE RESPONSES

In this survey, taken on I-94 in August 1968 and January 1969, a return-mailer questionnaire (Fig. 20) was distributed to approximately 5,000 drivers during each period. About 28 percent were returned; a summary of the responses is given in Table 7. Out-

1. Vehicle Description: Type: _____ Year: _____

2. Are you pulling a trailer? Yes No

3. Registration: Jackson or Calhoun Co. Other, in State Out of State

4. Where did you start your trip? _____

5. What was your destination? _____

6. Where did you enter I-94? _____

7. The reason for this trip was: Social-Recreational School Shopping
 To or from work Work (on the job) Misc. _____ Specify

8. How often do you use this section of I-94?
 Almost every day Almost every week Almost every month Only once or twice a year Less than once a year

9. When did you first become aware of the Motorist Aid Telephones?
 When I saw _____
 Prior to this trip. By this letter Other _____ Specify

10. What do you think of the Motorist Aid Phone System?
 Necessary Service, should be expanded.
 A convenience, would like to see it expanded.
 A convenience, but not necessary.
 Prefer past method of obtaining aid, such as raised hood, flare, handkerchief on door, etc.
 A need for better motorist aid exists, but I recommend: _____

11. Do you consider the motorist aid telephone signing adequate? Yes No

Further Comments: (Do you have further questions on this system? If so, please supply your name and address on the return. We will be glad to respond.)

From: _____

BUSINESS REPLY MAIL

No Postage Needed Outside of Michigan in the United States

POSTAGE WILL BE PAID BY

MICHIGAN DEPARTMENT OF STATE HIGHWAYS
 Michigan State Highways Building
 Lansing, Michigan 48926

Traffic Research

YOUR OPINION COUNTS

Michigan is one of several states conducting experimental projects to determine the needs of a standed freeway motorist and the effectiveness of various types of installations in providing for these needs.

The Michigan installation consists of pairs of telephones mounted on opposite sides of I-94 Freeway at approximately one-mile intervals in the 20-mile test area between Jackson and Battle Creek.

All calls, which are toll-free, from the motorist aid telephones are received by a nearby State Police Post and arrangements are made to provide motorists with needed assistance.

Your response to the attached questionnaire and additional comments will be a meaningful contribution to this study. The results of this study will contribute to a safer and more efficient freeway system.

When you have completed the questionnaire, please place seal and drop in a mailbox. Postage is prepaid.

Thank you.

Figure 20. Questionnaire handed to stranded motorists on I-94 during summer and winter surveys.

TABLE 7
SUMMARY OF RESPONSES OF STOPPED VEHICLE DRIVERS IN
QUESTIONNAIRES HANDED TO THEM

Response	Percent of Drivers	
	Summer	Winter
Type of vehicle		
Passenger, in county	18.8	21.4
Passenger, in state, out of county	43.5	48.4
Passenger, out of state	28.6	16.0
Panel and pickup cars with trailer, in county	0.3	1.3
Panel and pickup cars with trailer, in state	1.9	1.6
Panel and pickup cars with trailer, out of state	1.4	0.3
Truck, single unit	1.1	1.6
Truck, combination	4.1	9.4
Bus and motorcycle	0.3	0.0
Opinion of aid phones		
Necessary service; should be expanded	46.9	45.1
A convenience; would like to see it expanded	40.3	42.4
A convenience; but not necessary	10.3	10.5
Prefer past method of obtaining aid, such as raised hood, flare, or handkerchief on door	1.1	1.3
Other	1.4	0.7
Frequency of freeway use		
Almost every day	12.7	15.8
Almost every week	18.7	27.3
Almost every month	25.4	34.5
Once or twice a year	28.0	19.1
Less than once a year	15.2	3.3
Trip purpose		
Social-recreational	44.1	14.8
School	2.9	5.7
Shopping	2.3	1.6
Business	6.9	56.6
To or from work	28.8	8.8
Miscellaneous	15.0	12.5
Motorist aid phone signing considered inadequate	10.9	6.4

1. DATE _____ TIME _____ TEL. NO. _____

2. LOCATION OF INCIDENT FROM PHONE:
DIST. _____ mi. EAST WEST

3. SEX OF CALLER MALE FEMALE

4. DOES CALLER REQUIRE INFORMATION ONLY?
 YES NO
(If yes to question 4, interview is completed.)

5. CALLER REQUESTING AID FOR
 SELF ANOTHER
(If self, continue to 6 & 7)
A. Did caller talk to stranded motorist?
 YES NO
B. Did stranded motorist indicate distress?
 YES NO
If A & B "NO" comment _____

6. REASON FOR STOP
 Illness Abandoned car
 Police Action Misc. (Further aid not required)
 Car trouble (Indicate)
 Tire Mechanical
 Gas Accident
 Water Struck off road
 Oil Fire
Tow Required? Yes No
Medical Aid? Yes No

7. ACTION TAKEN
 Police Patrol Sent
 Commercial Escort Other
(If Police Patrol sent, form is complete; if not, continue)

8. NAME Mr. Mrs. Miss _____ Age _____
Address _____
City _____ State _____ Zip Code _____

9. WHERE ARE YOU COMING FROM?
WHERE ARE YOU COMING?
 Direct Route Indirect Route

10. PURPOSE OF TRIP
 Social-Recreational Shopping
 Work School
 To and from Work Misc.

11. HOW OFTEN DO YOU DRIVE THIS PART OF I-94?
 Almost every day Once or twice a year
 Almost every week Less than once a year
 Almost every month

12. WERE YOU AWARE OF THE "AID PHONES" BEFORE YOU STOPPED? YES NO

13. VEHICLE
 Passenger car Truck-panel or pick-up
 Truck-single unit Pass. car w/trailer
 Bus Truck-combin. units
 Motorcycle Other

14. REGISTRATION
 California or Jackson Co. Other
 Out of State In State

15. WEATHER
 75+ 50 to 75 30 to 50
 0 to 30 Below 0
 Clear Cloudy Rain
 Fog Snow

16. OFFICER'S COMMENTS (If any)

Form is to be used:

When Dispatcher receives a call on I-94 Motorist Aid Phones:

Specific Instructions:

1-4. Self Explanatory -- If the answer to 4 is Yes (Information only), interview may be terminated.

5. If call for "Another" complete 5(a) and 5(b), then #6 if possible and #7. If "Self", skip to #6.

6. "Police Action": The reporting of a law violation or a traffic hazard (not an accident).

"Misc.": Only when no other category is applicable and further aid is not required.

"Tow Required" & "Medical Aid": Should be checked Yes or No.

7. More than one may be checked: (Fill in name of commercial service. List any other action taken.)

8. If commercial aid is requested, securing RB information should not be a problem -- however, the interview statement may be used at this point.

9. Where are you coming from? If driver asks, "What do you mean?" We mean "the last stop you made other than for gas, food or lodging". "Where are you going?" means the next location where the motorist will fulfill a purpose of the trip other than gas, food or lodging or where the trip ends.

Direct or indirect route is an opinion of the driver.

10, 11, 12, 13, 14 Self explanatory.

At Conclusion:

Dispatcher: "Thank you for your cooperation and information.

A questionnaire will be mailed to you later asking a few questions about the speed and cost (if any) of the service.

Please mail this questionnaire back to us."

15. Self explanatory.

16. Comments -- State reasons if "Misc." is checked in questions 6 or 10.

State type if "other" is checked in 13.

Any further information which is applicable to the phone call not otherwise listed.

If patrol calls in, indicate this action in the box.

Figure 21. Interview form used by State Police dispatcher in answering calls on motorist-aid phones.


1. Description of vehicle you were driving: Body Type _____ Year _____ With Trailer Yes No

2. When did you first become aware of the Motorist-Aid Telephones?

Prior to this trip or
When I saw

AID TELEPHONES
NEXT 30 MILES

MOTORIST AID
PHONE SYSTEM
NEXT 30 MILES



EMERGENCY PHONE
EVERY 1/2 MILE

Other _____

3. How long did it take you to get to the telephone? _____

4. How long did you have to wait for aid after the telephone call? _____

5. Who provided the aid? _____ (NAME OF AGENCY)

If charges were involved, how do you consider them?

Amount _____

No expenses involved
 Reasonable
 High
 Excessive

6. If you needed assistance in the future would you use the Motorist-Aid Telephones? Yes No

7. What do you think of the Motorist-Aid Phone System?

Necessary Service, should be expanded
 A convenience, would like to see it expanded
 A convenience, but not necessary
 Prefer past method of obtaining aid, such as raised hood, flare, handkerchief on door, etc.
 A need for better motorist aid exists, but I recommend:

The following blanks may already be filled in. If not, please complete:

8. Vehicle Registration:

Jackson or Calhoun Co. Other in state Out of state

9. Where did you enter I-94? _____

10. Where did you start your trip? _____

11. What was your destination? _____

12. The reason for the trip was:

Social-recreational School Shopping
 To or from work Work (on the job) Misc.

Figure 22. Questionnaire mailed to motorists who used aid phones during 1968.

of-state traffic is higher in summer (30 percent) than in winter (16.3 percent). The trucks show up less than actual percentages because more of the surveys were taken in the daytime than at night, and the relative volumes of trucks to cars increase greatly at night. Both the summer and winter (87 percent) indicated strong favor for the aid phones.

TABLE 8
DISTRIBUTION OF CALLS BY MOTORISTS WHO USED AID PHONES

Location of Call Phone and Aid Requested	Percent of Calls	Location of Call Phone and Aid Requested	Percent of Calls
Location		Oil	2.0
All phone sites 1 through 34 ^a	52.0	Mechanical, tow required	19.2
All phone sites 35 through 62 ^b	48.0	Mechanical, no tow required	16.8
Total eastbound phone sites	47.6	Accident, medical aid and tow required	1.1
Total westbound phone sites	52.4	Accident, medical aid required and no tow	0.0
Eastbound phone sites 2 through 34 (even)	25.0	Accident, tow required and no medical aid	1.3
Eastbound phone sites 36 through 62 (even)	22.6	Accident, neither medical nor tow required	4.9
Westbound phone sites 1 through 33 (odd)	26.6	Stuck off road	3.1
Westbound phone sites 35 through 61 (odd)	25.8	Fire	1.3
Aid requested (1968)		Police action	2.0
Tires	22.0	Miscellaneous	0.9
Gas	21.0		
Water	4.4		

^aAverage 3,400-ft spacing between pairs.

^bAverage 5,400-ft spacing between pairs.

TABLE 9
SUMMARY OF RESPONSES OF PHONE USERS IN
QUESTIONNAIRES MAILED TO THEM

Response	Percent of Users
Frequency of travel on this section of I-94	
Almost every day	28.6
Almost every week	31.6
Almost every month	17.7
Once or twice a year	18.6
Less than once a year	3.5
Total	100.0
Trip purpose	
Social-recreational	34.6
School	5.7
Shopping	1.9
Business	9.5
To or from work	37.3
Miscellaneous	11.0
Total	100.0
Opinion of motorist aid phones	
Necessary service, should be expanded	69.2
A convenience, would like to see it expanded	28.2
A convenience, but not necessary	1.7
Prefer past methods of obtaining aid, such as raised hood, flare, or handkerchief on door	0.9
Total	100.0

The seasonal change in the composition of traffic is shown strongly in the high social-recreational percentages in the summer and the high business and work group percentages in the winter.

ANALYSIS OF REPORTS RECEIVED FROM STATE POLICE

Data recorded by the State Police dispatcher on the interview form shown in Figure 21 are given in Table 8. The call distribution rates vary somewhat along the study section; however, it cannot be stated with any confidence on the basis of these variances that the closer spaced phone sites (1 through 34) provide a better service to the motorist than the longer spaced sites (35 through 62). The categories and percentages of aid requested are similar to those reported by most other motorist-aid system studies. Tires and gas are about 21 percent each and mechanical aid, about 36 percent. The 7.3 percent of the calls for accident aid seems rather high for this type of need; information is not presently available concerning details of these accidents.

ANALYSIS OF MAILED QUESTIONNAIRE RESPONSES

An analysis of data received from questionnaires (Fig. 22) mailed to those who used the motorist-aid phone system in 1968 is given in Table 9. Only 73 percent of the phone users were aware of the aid phones before they stopped. The data indicate that those who were in need of aid placed a high value on the motorist-aid system; 69.2 percent considered the system a necessity, and 28.2 percent more thought the system a convenience that should be expanded. Almost all of the users, 97.6 percent, replied that they would use the phones again if the need arose.

Designing the First FLASH Installation

IVOR S. WISEPART, Airborne Instruments Laboratory, A Division of Cutler-Hammer, Farmingdale, Long Island, New York

The feasibility has been demonstrated of a technique that relies on passing motorists to report vehicles needing help. The system is named FLASH, which is an acronym for Flash Lights And Send Help. This paper describes the design and operation of the first installation on Interstate 4 between Lakeland and Orlando, Florida.

•THE PROBLEM of quickly detecting, locating, and aiding disabled vehicles on limited-access highways has received considerable emphasis in recent years. In 1962, Airborne Instruments Laboratory (AIL) began an investigation of the extent of the disabled-vehicle problem and a review of the various detection techniques and their economics (1).

COOPERATIVE-MOTORIST FEASIBILITY

In 1966, the Bureau of Public Roads asked AIL to investigate a technique that could be used in detecting and locating vehicles needing help, that was safe and simple, and that could be implemented quickly with minimum equipment cost. The technique was to rely on passing motorists to spot vehicles needing help and to report them at convenient locations along their route. It was desired that these passing motorists remain in their cars without slowing down or deviating from predetermined trip plans. Quick implementation required that no new equipment be installed in the vehicle. The use of familiar instruments, such as lights and horns, also minimized the need for special training of the motorist. Because many miles of rural roads are in desolate areas, it was desired that motorists needing help remain with their vehicles and not abandon them to seek assistance.

AIL conducted experiments on short sections (3 to 6 miles long) of the following routes: Long Island Expressway in New York, I-70 in Kansas, I-80 in Nebraska, I-15 in California, and Richmond-Petersburg Turnpike in Virginia. These roads were selected to cover a cross section of highway types and user characteristics. Experimental signs were installed requesting motorists to flash their lights (or sound their horns) 3 times if they saw vehicles needing help. A vehicle and driver were staged along each test section, simulating various scenes of disablement. Observers at the reporting location recorded the responses of the passing motorists. The data recorded indicated that passing motorists could be relied on to report motorists needing help and that the cooperative-motorist concept was indeed feasible (2).

THE FIRST OPERATIONAL INSTALLATION

After the operational feasibility of the cooperative motorist technique had been proved, the next logical step was to design and install the equipment and evaluate a fully operational system.

A 50-mile segment of Interstate 4 between Lakeland and Orlando, Florida, was selected because it satisfied the criteria for an initial installation. Factors included in the selection were number of interchanges, interchange spacing, traffic volume, and proximity to major cities at both ends of the test section. The varied availability of motorist services at interchanges along I-4 was a characteristic typical of rural Interstate mileage.

Local support was a basic requirement for achieving a successful first installation. The Florida Department of Transportation recognized the need for a disabled-vehicle location system and made important contributions to the system design and installation. The Florida Highway Patrol is closing the system loop by operating the terminal equipment and responding to motorists' needs for help. This combined effort will be the means for achieving the goals of public understanding, cooperation, and confidence in the FLASH System.

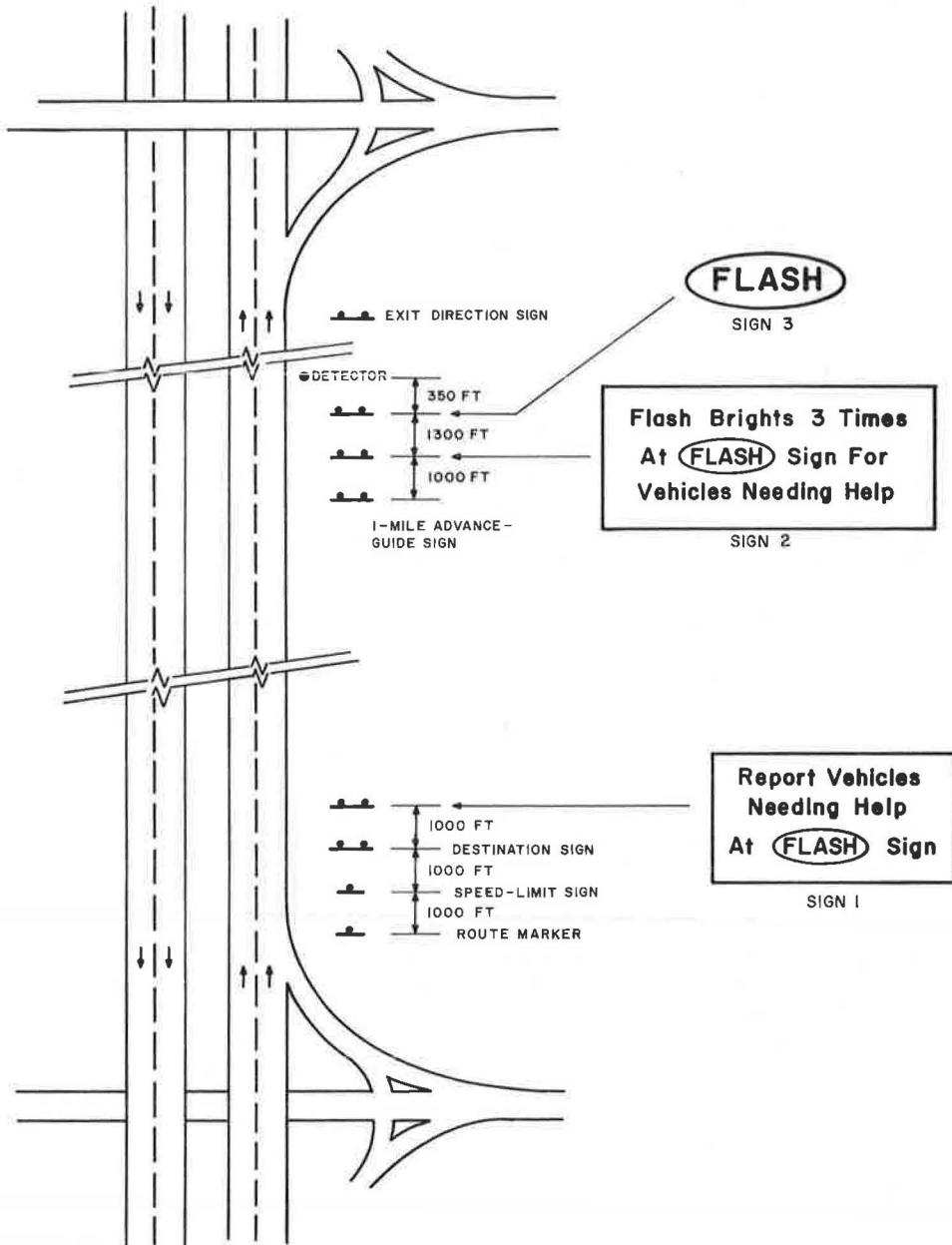


Figure 1. Content and typical locations of roadside signs.

INFORMING MOTORISTS

The successful operation of the FLASH System depends on the participation of the motoring public. Maximum participation of motorists can be achieved only through an extensive public education campaign and through widespread and uniform adoption of the FLASH System. Because this is not practicable for this first demonstration installation, conventional methods for soliciting motorists' participation have been adopted.

Motorists traveling along I-4 are informed how to report by a sequence of roadside signs (Fig. 1). Considerable attention was given to the design of these signs for maximum effectiveness. They are designed fully in accordance with pertinent Interstate signing specifications and current safety standards for placement and construction. The characters are reflectorized, and blue reflective background is used as indicative of motorist services. The signs have 30-ft offsets and breakaway support structures.

Sign 1 is located so that motorists entering I-4 will be quickly informed what to report (Vehicles Needing Help) and where to report (At FLASH Sign). As motorists approach the subsequent interchange, sign 2 relates how they should report (Flash Brights 3 Times) and repeats where and why. Sign 3, the FLASH sign, is located about one-quarter mile beyond sign 2 to allow cooperative motorists sufficient time to prepare to flash their bright lights.

The FLASH sign is intentionally designed to attract attention by its nonuniform shape and color. The sign has 15-in. reflective blue letters on a reflective white elliptical background. The elliptical shape has major and minor diameters of 100 in. and 40 in. respectively. It is intended that widespread application and motorist familiarization with the operation of the FLASH System will require only the presence of FLASH signs to designate the reporting location. Thus, future installations will use instructional signs at infrequent intervals as a reminder or be eliminated entirely.

Signs have been placed selectively (Fig. 2) to permit testing of the motorist's comprehension and retention of the sign instructions. The 50-mile section has 20 FLASH reporting stations—10 in each direction for an average 5-mile spacing. Each reporting station is designated by the presence of a FLASH sign. Instructional signs 1 and 2 are frequent at the beginning of the section. Toward the end of the section, signs 1 and 2 occur only after major interchanges that generate new traffic. Sign 4 (End FLASH Area) is located at each end of the instrumented section.

FLASH EQUIPMENT

In the design of the equipment that transmits motorists' flashes to Highway Patrol Troop Headquarters, maximum consideration was given to reliability, maintainability, compactness, and simplicity of operation. For example, wherever possible, throw-away plug-in modules and integrated-circuit components are used. The basic

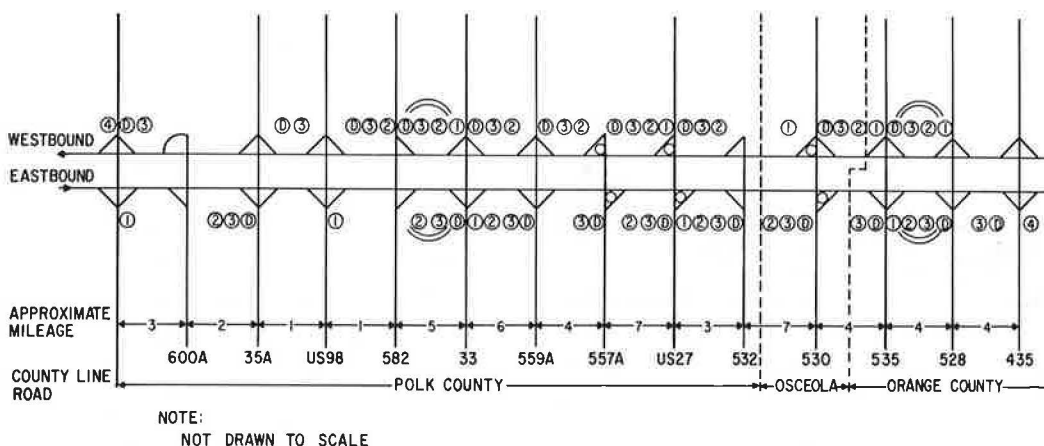


Figure 2. Test section and sign and detector layout.

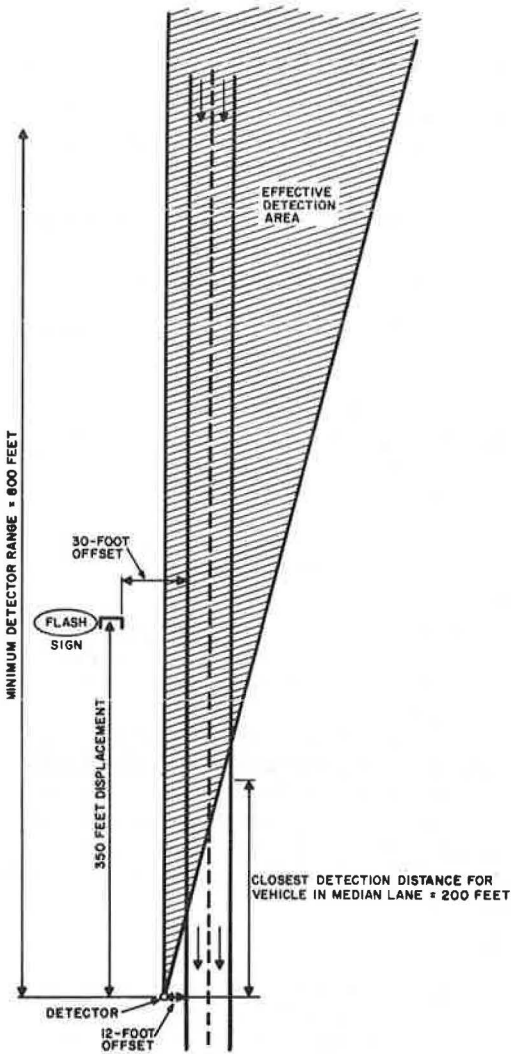


Figure 3. Detector coverage.

equipment required consists of roadside detectors, roadside computers, a central processor, and a monitor console.

Detectors

Associated with each FLASH station are a detector and roadside computer. Because motorists must be within the effective detection area (Fig. 3) for their flashes to be counted, it is necessary to determine the optimum detector location that would account for variations in motorists' initial flashing positions and their rates of flashing. Preliminary tests conducted on I-4 during August 1968 indicated that this optimum location was 350 ft beyond the FLASH sign.

The detector installation (Fig. 4) is designed and located to be as inconspicuous as possible and to minimize vandalism. Thus, the detector is placed so as to exactly substitute for a delineator and supports the delineator reflector. Because the detector is a line-of-sight device, each location must be carefully selected with consideration given to both horizontal and vertical roadway alignment.

Physically, the detector is a 4-in.-diameter plastic cylinder about 48 in. high. If hit by a vehicle, a notch cut around its perimeter will shear the detector off at the base. At the same time, a connector supplying power to the detector will also separate, and a signal will be transmitted to the monitoring console to inform the observer of the event. If the impact has not damaged the detector tube or its electronic operation, it can be reused by covering the separated pieces with a short plastic sleeve.

Photosensitive cells within the detectors sense motorists' flashes during the day or night and send signals through underground cabling to roadside computers.

Roadside Computers

Roadside computers, located at the edge of the right-of-way near the detectors, discriminate between valid flashes from cooperative motorists and spurious alarms caused by random reflections, lightning flashes, and the like. If a detector receives 3 flashes within a 5-sec interval, its roadside computer will transmit a coded tone signal through telephone lines to a central monitoring station located at the Florida Highway Patrol Troop Head-



Figure 4. Typical installation of roadside detector.

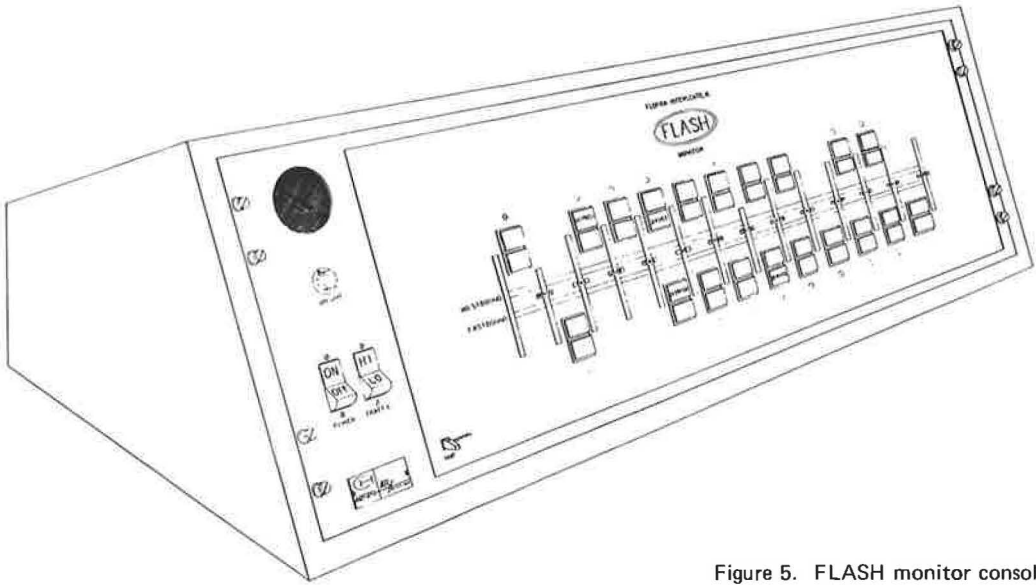


Figure 5. FLASH monitor console.

quarters in Lakeland. Power requirements for each roadside computer and detector combination are less than 40 watts.

Central Processor

The equipment in the Lakeland Headquarters consists of a central processing unit and a monitor console. When the central processor receives a report signal from the roadside computer, an electronic timer is started. False-alarm indications are minimized by the requirement that valid reports be received from more than 1 vehicle before a disabled-vehicle alarm is generated. False alarms may be caused by pranksters, curious motorists, and well-meaning motorists who misunderstand instructions.

The number of cooperative vehicles required to generate a disabled-vehicle alarm and the timer interval vary depending on the highway and user characteristics. For example, on high-volume road sections, 4 or 5 vehicles must flash within 3 min to generate a disabled-vehicle alarm; on low-volume roads, 2 or 3 vehicles must flash within 5 min. Each FLASH station can have its own setting; a HI/LO TRAFFIC switch on the monitor console permits the setting to be changed easily to account for volume variations caused by day and night cycles, inclement weather restrictions, and the like.

Monitor Console

The monitor console (Fig. 5) in the Lakeland Troop Headquarters is within arm's reach of the radio operator. On the console face panel, 2 horizontal light strips represent the eastbound and westbound directions of I-4. Vertical plastic strips represent the interchange crossroads. A pair of illuminated pushbutton switches is associated with each section of I-4 having a FLASH station.

When the road is clear of disabled vehicles, the I-4 light strips are illuminated green. When a disabled-vehicle alarm is received, a momentary tone sounds to alert the radio operator, the section of I-4 associated with the alarm signal turns from green to red, and the upper pushbutton adjacent to the highway section illuminates red with the word DISPATCH. After the radio operator dispatches a vehicle to investigate the red section, he presses the DISPATCH pushbutton to extinguish it and illuminate in amber the lower pushbutton with the word SERVICE. When the dispatched vehicle driver reaches the disabled motorist and ascertains the trouble, he informs the radio operator who then presses the SERVICE pushbutton. This extinguishes the SERVICE pushbutton and the roadway section returns to its normal green color.

SYSTEM TEST

The faithful operation of each detector link can be ensured by monitoring the progress of a test vehicle that flashes 3 times as it passes each detector. When the TEST switch on the console is in the TEST position and a valid report is received, a small indicator light adjacent to each pushbutton switch flashes at a fixed rate. This simple test checks out the detector alignment and optics, the underground roadside link, the telephone communications link between the detector and the monitor station, and the monitor station electronics. If a detector is knocked down, the associated indicator light illuminates continuously, informing the observer of the exact detector involved.

EVALUATION EQUIPMENT

During the first 12 months of system operation, AIL will collect data on system effectiveness and operation. Data will be recorded on punched paper tape. The number of vehicles flashing during the preceding 1-min interval will be recorded every minute of the day for each FLASH station together with radio-operator console manipulations.

FLASH SYSTEM EVALUATION

During the 12-month evaluation period, which began on November 14, 1969, we will determine the extent to which the FLASH System is improving service to stranded motorists. This will be accomplished through analysis of the punched paper tapes and also of Highway Patrol reports, staged experiments, and motorist questionnaires and interviews. The system will be refined during the evaluation period as operating experience is developed. As operators of the system, the Florida Department of Transportation and Highway Patrol will be consulted for their suggestions. The cooperation of the mass media will be solicited to assist in indoctrinating the public to the FLASH System through a carefully planned and coordinated publicity effort.

CONCLUSIONS

The greatest advantage of the FLASH System is that all road vehicles are presently equipped to participate and that minimum learning is required of the driver. This system is a tool that will enable the highway patrol to use its equipment more efficiently and thereby provide faster service for assisting the disabled motorist. This indirectly leads to increased highway safety.

This system is suited for use on highways in rural areas, including those with tourist traffic. The tourist, who is unfamiliar with the location of service facilities in the area, simply stays with the disabled vehicle and waits for official assistance to arrive.

Because this is a prototype system, the major effort has been concentrated on the development of reliable detection and monitoring. Future roadside detector stations could have a self-contained power source and communication link. This would facilitate the installation of detector stations in areas where power and telephone service are not available.

In the final analysis, the public will judge the acceptability of the system through its expression of confidence and satisfaction with the improved service provided.

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