HIGHWAY RESEARCH RECORD

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Freeway Traffic Characteristics and Control

5 Reports

Subject Classification

53 Traffic Control and Operations
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Foreword

The five papers and nine discussions in this RECORD represent frontiers of on-going research into the operational aspects of freeways. The ultimate objective of such research is to achieve improved traffic movement through various control techniques.

All of these studies are concerned with various aspects of freeway surveillance and control. Because land scarcity in urban areas will undoubtedly have an effect on the construction of new freeways, considerable effort is now being made to unclog bottleneck locations on existing facilities. Surveillance and control research may well provide answers to some of the knotty operational problems on existing freeways, and future freeways may incorporate surveillance equipment and control into their basic design.

The first paper offers some thoughts on how to solve the problems of operating freeway ramp controls during peak traffic periods. The paper proposes a control system where each entrance ramp is metered at a rate which maintains the total merging flow rate at or below a critical rate set by a central computer. The central computer would determine this optimum rate through linear programming.

The second paper reports on the experimental traffic control studies on the Gulf Freeway in Houston and presents and discusses the development of traffic control procedures and their effects on freeway and surface street operation. The control system is also evaluated and future needed research is summarized.

The third paper is a study of gap acceptance and merging behavior at freeway entrance ramps. The author presents a conceptual framework for the analysis of gap acceptance and rejectance at a freeway entrance ramp and reviews previous empirical and theoretical work. The paper also discusses a study of both left- and right-hand entrance ramps to freeways and compares the similarities and differences.

The next paper also deals with gap acceptance. It indicates gap acceptance and merging delay for six ramps of a Texas freeway. Queueing theory was applied to the ramp merging operation and a ramp metering operation and a ramp metering technique combining both a microscopic and macroscopic approach was developed.

Two Michigan researchers, in the last report, compare an urban freeway and an alternate surface street route using a galvanic skin response instrument, which measures a driver's tension, and a Drivometer, which measures driver action in relation to his vehicle. Indices are developed for the facilities, and relations between traffic volume and accident rates are described. Some correlations between the GSR and Drivometer are also presented.

This RECORD should be of prime interest to operations and research personnel of state highway departments who are concerned with providing better traffic flow on freeways. Some indirect applications of the research may be of interest to highway designers.

Contents

Peak-Period Analysis and Control of a Freeway System

JOSEPH A. WATTLEWORTH, Assistant Research Engineer, Texas Transportation Institute, Texas A & M University

•PEAK-PERIOD control of urban freeways is receiving increasing attention as a possible means of reducing congestion on these facilities. Controls on freeway entrance ramps during the morning peak periods are routine procedures on the Gulf Freeway Surveillance and Control Research Project in Houston as well as at other locations in the country. The Houston Research Project is conducted by the Texas Transportation Institute and sponsored by the Texas Highway Department and the U. S. Bureau of Public Roads.

There are several approaches to the philosophy of freeway controls and this paper presents one of them.

The ultimate goal of peak-period freeway control is to allow the entire automobile transportation system, of which the freeways are a part, to accommodate the same number of trips with reduced total travel time. In other words, the same origin-destination demand for trips would be accommodated in the system but these same trips, in aggregate, would require less travel time. This would be accomplished by more efficient use of the freeway and possibly the arterial street system.

The diseconomies of freeway congestion are reasonably well documented. It has been shown that for an oversaturated traffic system with a fixed demand-time function or input-time function, the system travel time can be reduced only by increasing the output rate of the system at some time (1, 2). It has, in fact, been shown for a system with a fixed input-time function, that minimizing the system travel time in any time period t_1 to t_2 is equivalent to maximizing $\int_{t_1}^{t_2} O(t)\,dt$ where $O(t)$ is the cumulative system output (2). The output rate of a freeway system (refers to a one-directional length of freeway) is maximized before congestion develops, that is, when the demand at each bottleneck in the system equals but does not exceed its capacity. When the demand on the freeway system increases, congestion sets in at one or more bottlenecks, and if the congestion becomes severe, the output rate of the freeway system is decreased. The decrease in output rate stems from two sources: (a) congestion at a bottleneck can decrease the flow rate below its capacity level, and (b) congestion or queues forming at a bottleneck can be propagated upstream past exit ramps, thereby decreasing the output rates on these exits (3). Thus, the desirable operation of the system is to maintain capacity flow rates at each critical bottleneck without allowing congestion to develop.

Several control techniques are available for use in a peak-period freeway control system and each has its advantages and disadvantages. The most positive control of vehicles can be accomplished as they are entering the freeway rather than after they are on the freeway. Therefore, entrance ramp metering or one of its special cases, entrance ramp closure, appears to be the most promising technique for controlling a large freeway system. (Entrance ramp metering means the placing of an upper limit on the flow rate on a ramp by controlling the time headways of entering vehicles.)

Since traffic conditions change during the day, the controls must be changeable. The controls must be started at the beginning of the peak period and ended when it is

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over, the metering rates must be determined and (perhaps) ramps must be closed and reopened. There are, of course, many means available for operating the control system, and each has its own advantages. The simplest operational scheme is the so-called fixed time scheme under which closure and opening of ramps, changes in metering rates on the ramps, etc., are all initiated at predetermined times.

For additional flexibility, traffic measurements can be used to operate the control system, i.e., the metering rates and closure times would be determined by measurements and/or observations of traffic on the freeway or surrounding facilities. One technique would be to detect a single variable such as speed, density, or lane occupancy on the freeway near an entrance ramp to determine the control required at that ramp. This technique has been applied in Chicago (4) for metering one ramp. Another method would involve the detection of gaps in the freeway lane adjacent to the entrance ramp and upstream of the ramp and releasing vehicles from the ramp when an "acceptable gap" was detected. This method has been suggested by Drew (5) and May (6).

A fourth, somewhat similar, control philosophy would be to maintain the sum of the flow rates on the entrance ramp and the adjacent freeway lane less than or equal to the single-lane merging capacity. Under this scheme, detection in the adjacent lane would provide the flow rate there and the metering rate on the ramp would be set at the difference between the merging capacity (one lane) and the flow rate in the adjacent freeway lane.

This paper presents some of the possible applications and advantages of another means of operating a peak-period metering (and closure) control system. The method involves the use of total flow (in one direction) which must be maintained at levels less than or equal to the capacity at each freeway bottleneck (2, 7). This control philosophy has greater potential than those previously discussed because the use of total flow across all freeway lanes permits use of the continuity characteristics of traffic flow, thereby making system considerations possible. The above-mentioned techniques can necessarily be concerned only with the operation of individual merging areas and each can (theoretically) maintain a smooth merging operation at each

ramps, frequently caused by grades, and the foregoing methods may not be too well adapted to these situations. Also, the operation of a given entrance ramp is not an independent consideration. Each ramp is one member of a system and the operation of one affects the operation of the system. It is desirable that the control system take this interdependency into account.

CONTROL AT INDIVIDUAL ENTRANCE RAMPS

At each entrance ramp in the controlled freeway system the sum of the total directional flow on the freeway and the flow on the ramp must be maintained less than or equal to some desired merging rate. The desired merging rate will usually be one of the following: (a) the merging capacity of the freeway at the ramp, (b) the capacity of a bottleneck between the entrance ramp and the next downstream ramp, or (c) another rate based on the optimization of the total system operation. Figure 1 is a schematic of a typical metering situation.

The detection station upstream of the ramp provides the flow rate approaching the ramp or the rate of flow of the traffic stream into which the ramp vehicles must merge. Criteria for the location of the metering station and detection station have been presented previously (2, 7). The relationship between critical times in the system is $t_f \geq t_d + t_c + t_r$ where t_f is the travel time on the freeway between the detection station and the merging section, t_d is the detection time, t_c is the computation time or data storage time and t_r is the average travel time on the ramp between the metering location and the merging section.

The upstream flow rate will, of course, vary with time and the desired merging rate will usually be constant for fairly long periods. If fr (t) is the time function of upstream flow rate and MR is the desired merge rate, MR - fr (t) is the rate at which available "capacity" (using the term quite loosely) is approaching the entrance ramp. In order not to allow any unused capacity to pass by the entrance ramp, the i th vehicle

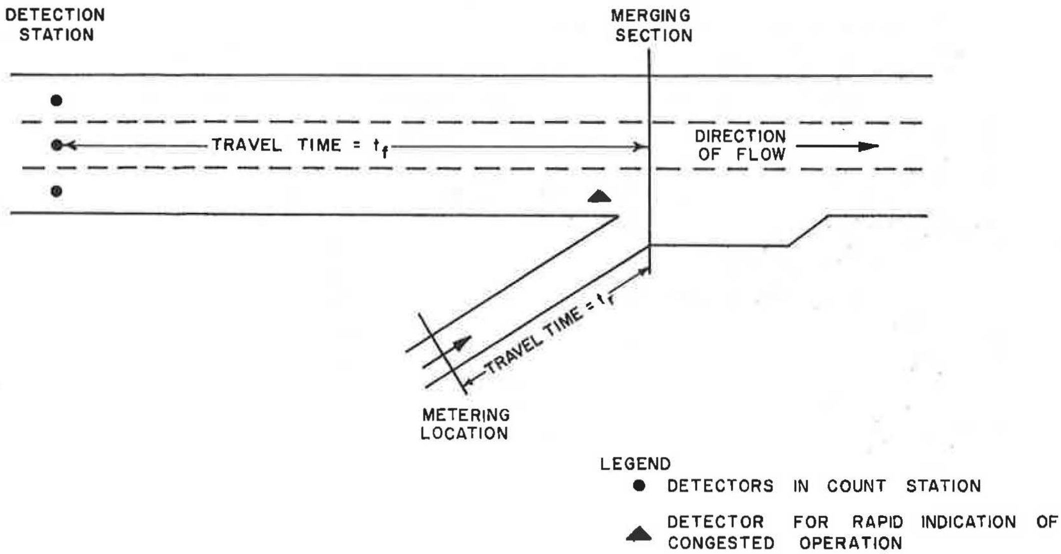

Figure 1. Schematic of metering and detection locations.

should be released from the metering station at time t_i so that

$$\int_{t_{i-1}}^{t_i} [MR - fr(t)]\, dt = 1$$

where t_{i-1} is the time of release of vehicle i - 1. This assures that over a period of time the merging volume will (nearly) equal the desired merging volume if there is sufficient demand on the ramp.

Figure 2 shows the relationships among several of the control variables. The flow rate is shown in digital form but it could equally well be shown in analog form and in practice the type would depend on the computing equipment used.

One method of computing the running average flow rate is shown (Fig. 2). The average flow rate shown for the time period 7:00:15 - 7:00:16 is based on the 15-sec count from 7:00:00 to 7:00:15, the 7:00:16 - 7:00:17 flow rate is based on the 7:00:01 - 7:00:16 count, etc. Thus, t_d = 15 sec and t_c = 1 sec. Therefore, proper location of the detector station would make $t_f = t_r + 16$ sec.

While prevention of congestion would probably be one major goal of a peak-period freeway ramp control system, no practical control system will achieve this goal. If the attempt is made to operate bottlenecks at or near their capacity (which is probable) some congestion will be expected at these locations. Also, the "unusual events" such as accidents, etc., occur all too frequently and can drastically reduce the capacity of the freeway. A method of prompt detection of congestion is, therefore, necessary if the congestion is to be cleared up as soon as possible.

The addition of one detector immediately upstream of the entrance ramp (Fig. 1) would provide the rapid indication of congestion. Lane occupancy, speed or (calculated) density could be the variable used for this purpose and when a critical level is reached an over-ride to the metering rate would be provided to clear the congestion. This is similar to a scheme tried in Chicago (6).

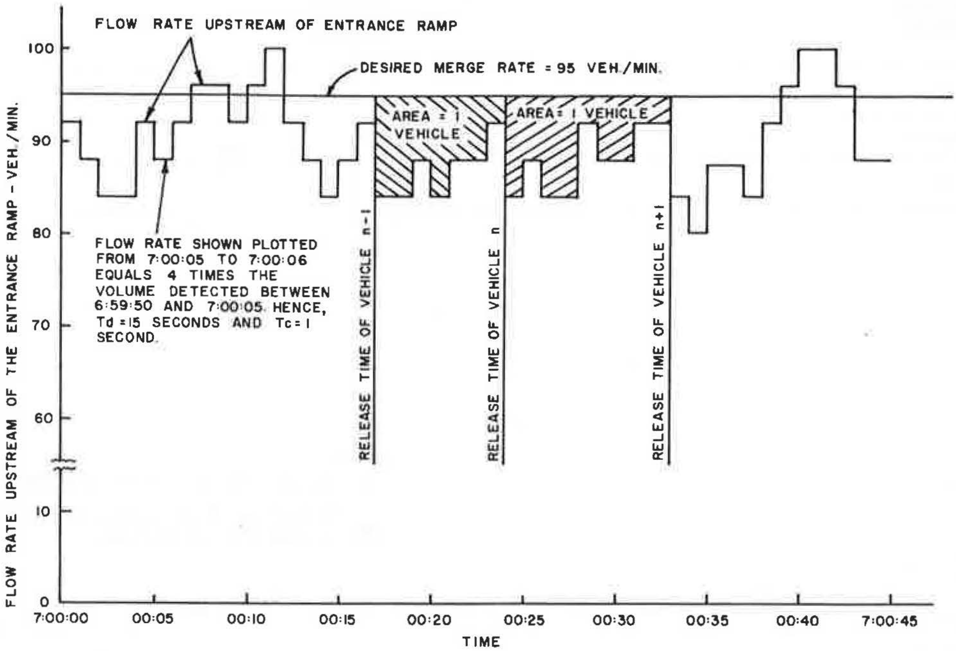

Figure 2. Relationships among detected volume, upstream flow rate, desired merge rate and release times of vehicles on the metered ramp.

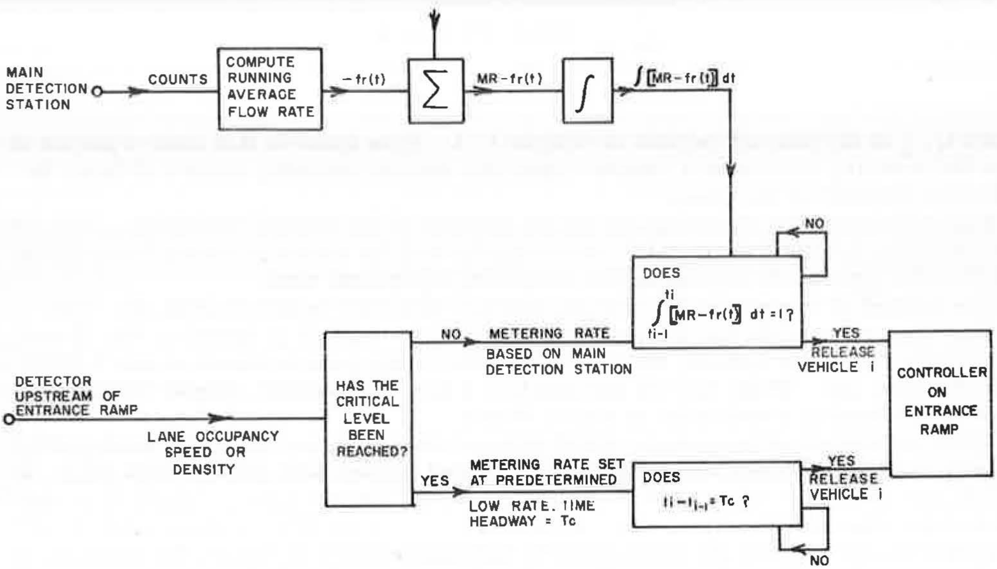

Figure 3. Schematic of controls of an individual entrance ramp.

Figure 3 is a schematic of the logic of the controls at a particular location. Once the desired metering rate (MR) for the location is set by the system monitor and control computer, the individual location can operate independently of all other control locations.

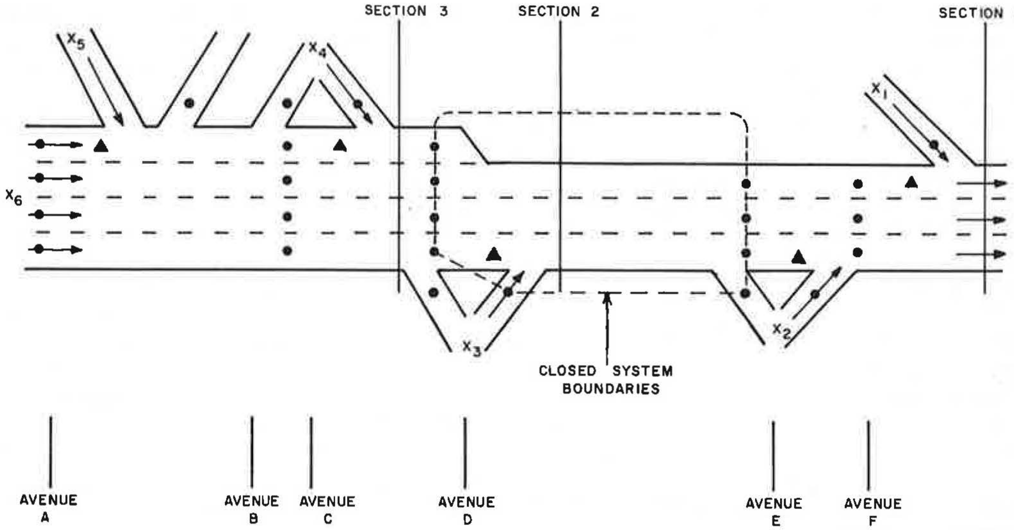

Figure 4. Schematic of freeway subsystem used in the linear programming model.

CONTROL OF THE SYSTEM

Each of the metering controllers at each entrance ramp should be operated so that optimal operation of the system of interest would result, at least when considered over a fairly long period of time. In the discussions which follow a central digital computer is envisioned in the role of monitoring the system operation and controlling the desired merge rates at each entrance ramp. While only one freeway is discussed here, a single computer could probably control several freeways.

In the steady-state analysis presented, the length of analysis period must be such that the traffic demand and origin-destination desires are relatively constant over the time period. For example, the inbound Gulf Freeway in Houston was found to be congested from about 7:00 to 8:00 a. m. Vehicles accumulated on the freeway (when demand on the system is greater than capacity) from about 7:00-7:30 a. m. and cleared from about 7:30-8:00 a. m. (3). Hence in this case one type of steady-state operation might be applicable from 7:00-7:30 a. m. and another from 7:30-800 a. m., so each period would be analyzed individually. In cases in which congestion is more severe, the periods of analysis could be lengthened.

A linear programming model (2, 7) is used for the control of a freeway system (Fig. 4). The simplest form of the model is

$$\text{Maximize} \sum_{j=1}^{n} X_j$$

$$\text{subject to} \sum_{j=1}^{n} A_{jk} X_j \leq B_k \qquad k = 1, \ldots, m$$

$$\text{and } X_j \leq D_j \qquad j = 1, \ldots, n$$

$$\text{and } X \geq 0 \qquad j = 1, \ldots, n$$

The X_j are the input volumes to the freeway system ($j = 1, \ldots, n$), the A_{jk} are the decimal fraction of vehicles entering at input j which pass through section k ($j = 1, \ldots, n$ and $k = 1, \ldots, m$) the B_k are the capacities of the freeway sections and the D_j are the hourly demands at input j. (If the level of service concept (8) is used for the system operation instead of the capacity concept, the B_k would represent service volumes.)

Briefly, this model maximizes the output [due to the first set of constraints, output (very nearly) equals input (2, 7)] of the freeway system subject to constraints which keep the demand less than the capacity at each section and which maintain the feasibility of the solution.

The solution vector contains the optimal inputs to the freeway as well as the values of the slack variables S_k in the active basis. The slack variables for the first set of (capacity) constraints place bounds on the desired merge rates. The desired merge rate at section K conforms to the following relationship: $B_k \geq MR_k \geq B_k - S_k$. For minimizing the queue at the upstream entrance ramp, $MR_k = B_k$. However, a higher level of service would prevail on the freeway if $MR_k < B_k$ so the tradeoff between ramp queues and freeway level of service can be considered.

Potentially, one of the major problems associated with an entrance ramp metering system is the development of long queues at the metering locations. It might be desirable to introduce constraints into the model to control these queues. Since the values of the slack variables in the second set of constraints represent the (approximate) length of the queue at the end of the study period (assuming it was zero at the beginning) the queue constraints can be placed on the slack variables.

Both types of queue constraints are somewhat indirect in that neither places a restriction directly on the queue length. Both restrict quantities that are related to the queue lengths.

The two types of queue constraints which can be placed in the model are

$$S_j \leq Q_j \qquad j = 1,\ldots, n-1 \tag{1}$$

and

$$S_j = S_{j+1} \qquad j = 1,\ldots, n-2 \tag{2}$$

Assuming input n is the freeway input where no queuing is allowed, the first type assures that the number of vehicles S_j which are denied access at the j th input is less [illegible]

each of these ramps.

There are perhaps many other types of constraints which could be added to this linear programming model. One such type would maintain the sum of the merging volumes in the adjacent freeway lane and the entrance ramp less than or equal to the single-lane merging capacity. A constraint which would accomplish this is

$$P_a \sum_{j=a+1}^{n} A_{jk} X_j + X_a \leq L_a \qquad a = 1,\ldots, n-1$$

where P_a is the percent of the total freeway volume upstream of input a which is in the lane adjacent to the entrance ramp, L_a is the single lane merging capacity at the input a entrance ramp, and assuming input n is the freeway input.

So far a method has been presented by which system operation is based largely on historical data (the A_{jk} and D_j are historical data and when normal operating conditions prevail the B_k are also based on historical data) which is then converted to control parameters (desired merge rates). The individual entrance ramps are controlled on the basis of present conditions not on historical data. However, if no reduced capacity situations (accidents, disabled vehicles, etc.) occurred, the desired merge rates could be determined once and no central monitor and control computer would be required. Since reduced capacity occurrences are fairly frequent on urban freeways, it is desirable that their effects on the operation of a freeway system be considered.

REDUCED CAPACITY OPERATION

Detection

Prompt detection of a reduced capacity occurrence is important if its adverse effect is to be minimized. In addition it is important to obtain a reliable estimate of the capacity of the freeway section at the occurrence if the system of controls can be adjusted to compensate for the capacity reduction.

Assume that the capacity is reduced between the Avenue D entrance ramp and the Avenue E exit (Fig. 4). Assuming also that the demand there is greater than the capacity, congestion will develop and begin to propagate upstream toward entrance D. The traditional method of detecting the capacity reduction would be by an indication of congestion at the nearest detector upstream of it. Low speed, high lane occupancy, or high density indicates a downstream source of congestion, in this case the event of interest.

With a detector on the Avenue D entrance ramp and on the Avenue E exit ramp, a closed system is defined and is enclosed by a dashed line (Fig. 4). The input locations to the closed system are the freeway section downstream of the Avenue D exit ramp and the Avenue D entrance ramp while the output locations are the Avenue E exit ramp and the freeway section downstream of this ramp. Shortly after the capacity reduction, the output flow rate from the closed system will decrease while the input flow will remain about normal (unless the capacity reduction is close to the input detector stations). If $I(t)$ and $O(t)$ are, respectively, the number of vehicles entering and leaving the system after some time t_0, the rate at which vehicles are accumulating in the system $[I(t+\Delta t) - I(t) - O(t+\Delta t) + O(t)]/\Delta t$ could be monitored and used to detect the reduction in capacity. Thus, when the output rate falls significantly below the input rate for some period of time a capacity reduction somewhere between the input and output location is fairly certain. Since there is a main detection station upstream of every entrance ramp, the freeway is broken into a series of these closed systems. The central computer could monitor each of these systems for unusual behavior.

When a capacity reduction has occurred in one of these closed systems, steady-state conditions will normally exist between the point of decreased capacity and the output section of the system. If steady state does not exist in this area a more severe capacity reduction must have taken place downstream. However, in most cases the steady state exists and the output flow rate will, over a reasonable time period, equal the flow rate across the reduced capacity section. The capacity flow rate of this section, then, can be estimated by simple volume counts downstream of it.

Once the capacity reduction has been detected and the capacity flow rate has been estimated, the modified operation of the freeway system can be determined. The revised capacity flow rate is substituted in the linear programming model and the new desired merge rates are obtained. These, as well as the expected input volume at each entrance ramp, provide a rapid estimate of the severity of the ramp controls necessary upstream of the capacity reduction. All of these operations would be conducted automatically within the central computer.

As an example of the operation of the model under normal and reduced capacity circumstances, the system (Fig. 4) will be analyzed. Only three freeway capacity constraints will be considered in this simple example, on sections 1-3, and no queue constraints are included. An hour period is used in the analysis.

The statement of the model, then, is

$$\text{maximize} \sum_{j=1}^{6} X_j$$

$$\text{subject} \sum_{j=1}^{6} A_{jk} X_j \leq B_k \qquad k = 1, 2, 3$$

$$\text{and } X_j \leq D_j \qquad j = 1, \ldots, 6$$

$$\text{and } X_j \geq 0 \qquad j = 1, \ldots, 6$$

TABLE 1

A_{jk} USED IN MODEL FOR NORMAL OPERATION

		j					
		1	2	3	4	5	6
	1	1.000	1.000	.949	.933	.824	.519
k	2			1.000	1.000	.922	.619
	3				1.000	.969	.777

TABLE 2

B_k USED IN MODEL FOR NORMAL OPERATION

K	B_k
1	5900
2	6000
3	6450

Table 1 gives the A_{jk} used in the model under normal operating conditions. These data would be obtained from O-D surveys at the entrance ramps.

Table 2 gives the B_k, the freeway capacities under normal operation. These would be obtained from historical volume counts.

The D_j, the maximum hourly demand at each input, are given in Table 3. These, too, would be obtained from a series of volume counts.

The simplex method was used to solve this problem and Table 4 contains the

TABLE 3

Dj USED IN MODEL FOR NORMAL OPERATION

j	1	2	3	4	5	6
D_j	600	475	450	500	825	6800

being the slack variables.

The optimal simplex tableau is given in Table 5. The optimal solution is $X_1 = 447$, $X_2 = 475$, $X_3 = 450$, $X_4 = 367$, $X_5 = 825$, $X_6 = 6800$, $S_2 = 213$, $S_4 = 153$, and $S_7 = 133$. Hence, 447 vehicles can enter the freeway via the Avenue F entrance ramp and 153 must be stored or diverted there. Only 367 vehicles can be allowed to enter at Avenue C, and since the demand is 500 vehicles, 133 vehicles in the hour must be stored or diverted there. The second constraint turned out to be redundant and there is a 213-veh/hr excess capacity at Section 2.

Although the tableaux are not presented here, this same problem was solved again assuming that a capacity reduction was detected at Section 2 and that the capacity there was estimated to be 5400 veh/hr. In this solution $X_1 = 600$, $X_2 = 475$, $X_3 = 63$, $X_4 = 367$, $X_5 = 825$, $X_6 = 6800$, $S_1 = 214$, $S_6 = 387$, and $S_7 = 133$. Thus, no vehicles would have to be denied access at the Avenue F entrance; in fact, the capacity constraint at Section 1 is now redundant and has a 214-veh/hr excess capacity. The inputs X_2, X_4, X_5, and X_6 are all unchanged. However, with the capacity at Section 2 reduced, X_3 decreases to 63 vehicles (from 450). Thus, during the hour only 63 vehicles would be expected to be able to enter the Avenue D entrance ramp while 387 would have to be diverted. This ramp would probably be closed due to the capacity reduction.

The effect of the accident on the system output Z_O during the hour can also be seen from these analyses. Under normal operation $Z_O = \sum_{j=1}^{n} X_j = 9364$ while with the

TABLE 4

ORIGINAL SIMPLEX TABLEAU

c_i	x_i \ x_j	x_1	x_2	x_3	x_4	x_5	x_6	s_1	s_2	s_3	s_4	s_5	s_6	s_7	s_8	s_9	b_i
	c_j	1	1	1	1	1	1	0	0	0	0	0	0	0	0	0	
0	s_1	1	1	.949	.933	.824	.519	1									5900
0	s_2			1	1	.922	.619		1								6000
0	s_3				1	.969	.777			1							6450
0	s_4	1									1						600
0	s_5		1									1					475
0	s_6			1									1				450
0	s_7				1									1			500
0	s_8					1									1		825
0	s_9						1									1	6800
$Z_j - C_j$		-1	-1	-1	-1	-1	-1	0	0	0	0	0	0	0	0	0	$Z_o = 0$

TABLE 5

OPTIMAL SIMPLEX TABLEAU

c_i	x_i \ x_j	x_1	x_2	x_3	x_4	x_5	x_6	s_1	s_2	s_3	s_4	s_5	s_6	s_7	s_8	s_9	
	c_j	1	1	1	1	1	1	0	0	0	0	0	0	0	0	0	
1	x_1	1						1		-.933		-1	-.949		.080	.206	447
0	s_2								1	-1			-1		.047	.158	213
1	x_4				1					1					-.969	-.777	367
0	s_4							-1		.933	1	1	.949		-.080	-.206	153
1	x_2		1									1					475
1	x_3			1									1				450
0	s_7									-1				1	.969	.777	133
1	x_5					1									1		825
1	x_6						1									1	6800
$Z_j - C_j$		0	0	0	0	0	0	1	0	.067	0	0	.051	0	.111	.429	Z_o = 9364

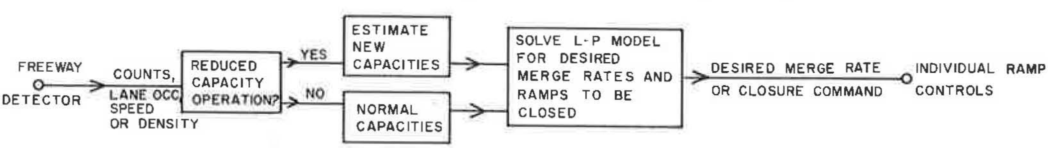

Figure 5. Schematic of operations of central monitor and control computer.

capacity reduction $Z_0 = 9130$. Therefore, the output of the system would be about 234 veh/hr less when the capacity of Section 2 is reduced to 5400 veh/hr.

Figure 5 shows a schematic of the central monitor and control computer operations. It receives inputs from the freeway detector and supplies outputs to the individual controller locations. The other operations and decisions are performed internally.

In summary, some of the advantages of using the total directional flow rate and capacity to operate a system of freeway ramp metering controls have been presented. The main advantages stem from the ability to make use of the continuity characteristics thereby permitting systems analysis and operation. Each local controller obtains the desired merge rate from the central computer and then determines the time headways between entering vehicles to maintain this rate of merge. The central computer monitors the freeway operation by examining the detector outputs. It determines whether normal or reduced capacity operations prevail and establishes the proper merging rates at each entrance ramp in the system. It supplies these rates to the local controllers which then maintains them.

The control system suggested here is fairly elaborate and an investment in such a system should be carefully analyzed. An incremental analysis should be used. If the annual cost of the system as outlined in this report is \$200,000 and the annual benefit to the motorists is \$400,000, it might seem to be justified. However, an extremely simple, fixed-time control scheme with an annual cost of \$20,000 might result in an annual benefit to the motorists of \$300,000. In this case the incremental investment of \$180,000 for an annual benefit of \$100,000 may not be justified.

AKNOWLEDGMENTS

The author would like to thank the following staff members of the Texas Transportation Institute for their many stimulating discussions regarding peak-period freeway control: C. J. Keese, Executive Officer of TTI; Charles Pinnell, Director of the Gulf Freeway Surveillance and Control Project; and William R. McCasland and Donald R. Drew, staff members of this project.

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Discussion

EDWARD F. GERVAIS, National Proving Ground for Freeway Surveillance, Detroit, Mich.—The material in this paper deals primarily with a ramp metering system since the author feels it is the most effective means of reducing travel time and increasing the number of trips obtained on a freeway during peak periods. While ramp control cannot be given determinate values at this stage of the research, it certainly appears to be a logical statement that it is effective and the work done so far at the National Proving Ground for Freeway Surveillance, Control and Electronic Traffic Aids would support this conclusion.

We would certainly agree that if the freeway traffic conditions have reached an unstable condition created either by congestion or extraordinary events, ramp control would be one of the most effective means of returning the freeway to a stable condition. At this point, I must add that we have experimented only with total ramp closure in Detroit for two reasons: (a) adequate storage area back of the entrance ramps necessary for ramp metering is not present at the entrance ramps in our study area, and (b) in the early stages of research, there was little to be gained by two agencies performing work on the same subject.

Where control at individual entrance ramps is discussed, we find the term "merging rate" (MR) employed. Stated in plain language, this term represents the combination of freeway and entrance ramp traffic which will operate downstream of the merge point in a state which will best answer the description of being optimum for a particular condition.

The merging rate is given in vehicles per minute which by itself is a simple enough quantity to work with. However, if we were to acquire a value for the merging rate for a specific time from downstream traffic measurements, we would have to know the downstream flow rate and the headway or speed conditions under which it occurs and then determine what merging rate can be used at the ramp metering point.

Under stable speed conditions the problem would be relatively simple, but when speed differential exists on the freeway under heavy traffic conditions, projections must be made in flow rates to determine whether the area into which traffic is moving can either pass or absorb in "slowdown storage" additional vehicles. This is by no means a simple problem and is one that will require quite extensive research.

An examination of the MR factor indicates that there is considerable complexity not only on what to measure but on how much to measure in order to obtain a usable value. I would certainly agree with the author that the use of entrance ramp volume and volume on the freeway lane adjacent to the ramp will provide an inadequate means of arriving at an appropriate value for merging rate unless other factors are taken in consideration.

This is especially true in view of the varying geometrics of our freeways. It would be possible, for instance, to have one entrance ramp a short distance upstream of an entrance ramp being measured for ramp control. This distance might be too short for entrance traffic to redistribute itself into other lanes of the freeway before it is measured in advance of the second entrance ramp. This does not mean that it could not be done without trouble downstream of the second entrance ramp. This would depend on traffic conditions in the other freeway lanes.

I feel that the merge rate is not as stable as the author describes or we would like to see. The value of the merging rate can be influenced by the design of an entrance ramp. We should consider the big advantages of ramp metering as coming from the reduction of ramp entrance traffic interference rather than withholding vehicles from the freeway to reduce traffic loads. Actually the freeway traffic volumes maintain higher "peaking" levels because of ramp control.

Metering will permit entrance of vehicles to the freeway at spaced intervals and minimize the problem created by "tailgating." Since merging interference is more prevalent with certain geometric designs, we would have to compensate by properly modulating the merging rate so that the optimum downstream flow is preserved. This indicates the need of studying the various ramp designs and determining the types that would work most satisfactorily in a control situation. When considering ramp

designs where control is present we might be able to select less costly designs which still produce a good level of service.

The author discusses a linear programming model for control of freeway traffic. The freeway is broken down into a series of subsystems in which the input-output is measured. One of the important things toward which the research programs must be directed is to find how traffic behaves on a freeway on a system basis and determine essential variable information which we must know in order to satisfy control requirements. Minimizing detection points is certain to keep costs of freeway control systems to a minimum.

One of the reasons why I am an enthusiastic supporter of closed circuit television is that not only does it give you information which cannot be obtained by other means, but it also would permit the use of an automated system which would not require a saturated detection system since anything out of the ordinary could be recognized by television and corrected. One thing that I feel will be important in any experiments on ramp control is not only to regulate a flow of traffic at a particular entrance ramp but anticipate and know where this traffic will go if it is diverted.

I feel strongly that once we know proper values for merging rates we would find it necessary to close certain ramps quite repetitiously, since any interference from entering traffic at certain ramps, no matter in what reduced numbers, would create freeway flow reduction. Even in ramp metering there is a certain amount of traffic which will bypass the entrance ramp if the length of the queue does not appear reasonable to the individual driver. This may subject the downstream ramp to a greater pressure for entrance to the freeway.

What this merely says is that in a complete systems concept, consideration must be given to the performance of traffic off the freeway particularly along those routes which parallel the freeway since it does represent transportation through a corridor and our real purpose is to make most efficient the corridor movement of traffic. This might appear that we are already advocating the introduction of other inputs to an already complicated system but I can see many of these factors being accounted for

For instance, certain discrete measurements of the traffic system could be used to establish a program which would work fluidly in accordance with pre-knowledge of certain factors such as weather and time. By determining beforehand how these variables can be handled in a computer program, distribution of traffic over a surface street and freeway network can be made and the detectors in the system can be made to determine whether the program is working properly in the various components of the system.

We should never lose sight of the fact that a good control system still has its final output in a driver receiving information and performing on the road. Since he is a creature of habit, it would be better that small benefit changes be eliminated from traffic systems so that the driving task is not complicated to a degree which nullifies the benefits. We must also be very careful in designing a control system on a system basis which will have built-in dampeners.

I point out the danger that if we obtain samplings by using a short time base, we may wind up with a situation where we sample a traffic situation, apply a control change, measure the traffic stream again after the change, and come up with still another control change. In other applications, this has many times created a "hunting and seeking" problem which if not properly considered could be a serious fault in any control system.

The delineation of the above problems is not meant to detract from the conclusions of the author, but rather to point out some of the factors which may have to be considered in evolving a practical control system for ramp traffic which will accomplish its intended purpose.

ROBERT S. FOOTE, Tunnels and Bridges Department, The Port of New York Authority—The subject Dr. Wattleworth has treated in this paper is a most important element in achieving the best operation of limited access road systems. I believe he has made a particularly valuable contribution in presenting the system aspects of on-ramp control, and in applying linear programming to develop an optimal solution.

At this stage in the operation of congested roads, I do not believe the point can be stated too often: control of traffic entering and moving over limited access roads (and I include tunnels in this category) is essential when demand exceeds capacity, if full benefit is to be achieved from the road. There has been increasing recognition of this viewpoint in the past few years, and our work at The Port of New York Authority has been concerned with confirming and applying the concepts of traffic flow control. This experience leads me to discuss the paper in the light of my guess that control of freeway traffic may develop along the same lines that control of tunnel traffic is developing.

Lacking experimental work, the author has rightly presented the need for a system control, rather than individual local controls, in a tentative way. It seems to me these are important questions to be answered: Will local control do a good enough job? Is a systems approach needed?

By local controls, I mean that the decision about the rate at which traffic should enter the freeway at a particular on-ramp is based on detection of freeway traffic conditions near the ramp. These controls can prevent congestion from being caused on the freeway at the merge point due to excess traffic from the on-ramp. But, I believe in most cases this sort of control is not adequate.

For one thing, on a given road system where demand exceeds capacity, there will be one original, principal, controlling bottleneck where congestion is first precipitated. If that bottleneck is at an on-ramp, local control at that point could be very effective. But local control at other on-ramps would not be relevant to preventing congestion at the critical on-ramp. And where the bottleneck is not usually at an on-ramp but is located along the closed roadway (such as at a grade change point), the local on-ramp control would again not be relevant or effective in forestalling congestion.

For another thing, our experience in controlling tunnel traffic flow has been that limiting traffic demands entering the roadway on the basis of conditions in the main stream at only one usually critical point is not enough. This, of course, is a function of the geometrics and capacity characteristics of the particular roadway being considered, but we find two or three or more points where congestion might develop. For these reasons, I believe that in most cases a coordinated central control of the entire critical network will prove desirable, rather than local independent controls.

Probably the most important question to ask in considering this paper is: Will steady-state control do a good enough job (except for special reduced capacity occurrences)? Experimentation is certainly needed on each roadway to answer this question, but I predict that steady-state controls will generally not do a good enough job and that more flexible controls will be the preferred mode. By steady-state controls, I mean that, barring special events such as accidents, the merge rate established for each of the access points is based on historical data as to demand, destination and capacity rather than on the particular current traffic situation.

My prediction is based on experience that capacity depends markedly on such random variables as weather conditions and traffic composition, as well as on the special events such as disabled vehicles. But, another variable which could be important on a freeway or other multi-exit road is the fluctuation in the destinations of the traffic on the road at any particular instant. While the demand and destination patterns measured over an hour may be highly stable from day to day, there can be much variation in these patterns measured over 1, 5, or 10 minutes. For these reasons, as a control system is sought which will come closest to maximizing traffic flow, I feel that system will have to be sensitive to current and even predicted conditions of capacity and demand.

As the system becomes more responsive to traffic conditions, problems of control stability become apparent. Since even the steady-state system considered would

respond to reduced capacity occurrences (such as an accident), I feel the problem of stability may arise when experiments are conducted in Texas. The stability problem arises in part due to the time lag in reacting to capacity reduction. During the time lag the road is continuing to receive more traffic than it can handle, as there is a backlog of excess traffic which must be removed. To remove this backlog, the input rate should be lower than even the reduced capacity output rate. But, when the backlog is cleared, there then occurs another time lag (which, in a low-response system could be lengthy) when the road is "starved" for traffic. The tendency then is to over-correct, and the cycle repeats. This at least is what we found we were doing in controlling tunnel traffic.

While various means exist to damp the oscillation, the most effective means to stabilize the input control system would be to predict what conditions on the limited-access roadway are likely to be in the next 5, 10, or 15 minutes, and determine input or merge rates on that basis. We have found that knowing the number of vehicles actually present in critical road sections is an effective predictor. The control system we are now building for the Lincoln Tunnel is based on measuring density. This is a task which requires a computer, which leads me to believe that the most important task for a traffic flow control computer will be to predict, assess and determine when changes in strategy are needed, rather than to run a linear program to determine the best inputs for the new conditions. It may be preferable to store solutions for a variety of conditions in the computer, ready for instant call-up and effectuation when needed.

One of the most interesting areas for discussion is the measure of effectiveness to be used. Wattleworth's formulation seeks to maximize the number of vehicles entering the system. Another measure of effectiveness (included in this discussion) was proposed by Harold Greenberg who consults with us on tunnel traffic problems. This would be to minimize the difference between section capacities and section throughputs. Dr. Greenberg calculated the linear programming solutions for this measure of effectiveness, and when they are compared with the particular solution reported by Wattleworth, it becomes clear that the Greenberg measure would provide a higher number of vehicle-miles traveled in the given system, even though the number of

The point here is not that one measure is necessarily better than another, but rather, that there are a range of plausible measures which should be explored. One criterion which should be used in selecting the desired measure is to consider how each measure alters input strategies as section capacities vary. For example, if the capacity of Section 1 is for some reason cut by one-third, the amount of traffic entering the various ramps might then differ markedly depending on which measure of effectiveness was used.

In conclusion, I would like once again to stress that this is a very valuable paper, and represents a definite step forward in the engineering of traffic controls on limited-access roads. We have a long way to go, but Dr. Wattleworth has taken an important step in his paper.

Alternate Linear Programming Formulation (Greenberg)

We desire to achieve maximum use of each section of the freeway. Thus, we would like to obtain the B_k level for each section. This can be done by an alternate linear programming formulation. The constraints

$$\sum_{j=1}^{n} A_{jk} X_j \leq B_k \qquad k = 1,\ldots,m$$

are written as

$$\sum_{j=1}^{n} A_{jk} X_j + S_k = B_k \qquad k = 1,\ldots,m$$

in the usual way, where S are the slack variables. To achieve maximum usage, we want to minimize

$$W = \sum_{k=1}^{m} S_k$$

subject to

$$\sum_{j=1}^{n} A_{jk} X_j + S_k = B_k$$

$$X_j \leq D_j, \; X_j \geq 0, \; S_k \geq 0 \quad j = 1,\ldots,n \quad 1,\ldots,m$$

Using the example starting on page 6, we obtain the following optimal solutions:

$$X_1 = 411, \quad X_2 = 475, \quad X_3 = 450, \quad X_4 = 500, \quad X_5 = 825,$$

$$X_6 = 6629, \quad S_1 = 0, \quad S_2 = 186, \; S_3 = 0, \quad W = 186$$

or

$$X_1 = 600, \quad X_2 = 286, \quad X_3 = 450, \quad X_4 = 500, \quad X_5 = 825,$$

$$X_6 = 6629, \quad S_1 = 0, \quad S_2 = 186, \quad S_3 = 0, \quad W = 186$$

Any convex combination is also a solution. Here $Z = X_j = 9290$, which compares to 9364 for maximizing Z. The max Z solution gives W = 213.

ADOLF D. MAY, JR., Associate Professor of Civil Engineering and Associate Research Engineer, Institute of Transportation and Traffic Engineering, University of California, Berkeley—The work of Dr. Wattleworth over the past several years in this area of peak-period control of a freeway system has been stimulating and provocative to other investigators. His work pertaining to the use of linear programming techniques for freeway control has opened new avenues for study. This paper has encouraged the discusser to become actively involved in the proposed linear programming model and to investigate modifications for extension and possible improvement. The following comments will be directed primarily to reporting on the discusser's experience with the model and to suggest some further avenues of possible study.

As a first step in becoming familiar with the linear programming model, a computer program was prepared and using the same traffic data, the two examples given in the paper were run on an IBM 1620 computer. The results obtained were identical to those described in the paper; the computer output formats are given in Tables 6 and 7. After further study of the model and the accompanying results, certain modifications of the model were undertaken.

The original model permitted the freeway input volume to be equal to or less than the freeway input demand. This would permit queuing on the freeway if the freeway input volume was in fact less than the freeway demand. Fortunately with the particular traffic data provided, no queuing did occur. But this part of the model was modified to insure that such queuing would not occur with other data inputs, since it does not seem appropriate to develop a control scheme to eliminate congestion on one freeway section by creating congestion at another. If in fact this does not permit a solution to the model, consideration could be given to extending the ramp control further upstream.

TABLE 6

ORIGINAL PROBLEM

Input	Demand	Allowable Volume	Queue or Diversion	Value[a]
Ramp 1	600	447	153	—
Ramp 2	475	475	—	0.0000
Ramp 3	450	450	—	0.0510
Ramp 4	500	367	133	—
Ramp 5	825	825	—	0.1111
Freeway	6800	6800	—	0.4289

Bottleneck	Capacity	Actual Flow	Excess Capacity	Value[b]
Section 1	5900	5900	—	1.0000
Section 2	6000	5787	213	—
Section 3	6450	6450	—	0.0670

[a]Increase in objective function per unit increase in input demand.
[b]Increase in objective function per unit increase in bottleneck capacity.

Note: Total output = 9364 veh/hr (objective function).

TABLE 7

ORIGINAL REDUCED CAPACITY PROBLEM

Input	Demand	Allowable Volume	Queue or Diversion	Value[a]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Ramp 3	450	63	387	—
Ramp 4	500	367	133	—
Ramp 5	825	825	—	0.0780
Freeway	6800	6800	—	0.3810

Bottleneck	Capacity	Actual Flow	Excess Capacity	Value[b]
Section 1	5900	5686	214	—
Section 2	5400	5400	—	1.0000
Section 3	6450	6450	—	0.0000

[a]Increase in objective function per unit increase in input demand.
[b]Increase in objective function per unit increase in bottleneck capacity.

Note: Total output = 9130 veh/hr (objective function).

$X_{j=6} \leq D_{j=6}$ **(original model)**

$X_{j=6} < D_{j=6}$ **(queue on freeway)**

$X_{j=6} = D_{j=6}$ **(suggested revised model)**

The original model was formulated to maximize $\sum_{j=1}^{n} X_j$ which is the sum of the input volumes to the freeway system. Perhaps this is the best quantity to maximize. The

TABLE 8

MAXIMIZE VEHICLE-MILES PROBLEM

Input	Demand	Allowable Volume	Queue or Diversion	Value[a]
Ramp 1	600	447	153	—
Ramp 2	475	475	—	0.5435
Ramp 3	450	450	—	2.0780
Ramp 4	500	367	133	—
Ramp 5	825	825	—	0.7541
Freeway	6800	6800	—	0.3633
Bottleneck	**Capacity**	**Actual Flow**	**Excess Capacity**	**Value[b]**
Section 1	5900	5900	—	0.0190
Section 2	6000	5787	213	—
Section 3	6450	6450	—	2.5803

[a]Increase in objective function per unit increase in input demand.
[b]Increase in objective function per unit increase in bottleneck capacity.

Note: Total output = 21,040 veh/mi (objective function).

TABLE 9

UNEQUAL QUEUE RESTRICTION PROBLEM

Input	Demand	Allowable Volume	Queue or Diversion	Pre-Set Maximum Queue or Diversion
Ramp 1	600	500	100	100
Ramp 2	475	425	50	50
Ramp 3	450	443	7	50
Ramp 4	500	399	100	100
Ramp 5	825	791	34	75
Freeway	6800	6800	—	0
Bottleneck	**Capacity**	**Actual Flow**	**Excess Capacity**	**Value[a]**
Section 1	5900	5900	—	1.0537
Section 2	6000	5782	218	—
Section 3	6450	6450	—	0.1359

[a]Increase in objective function per unit increase in bottleneck capacity.

Note: Total output = 9359 veh/hr (objective function).

discusser, however, gave some consideration to maximizing $\sum_{j=1}^{n} \ell_j X_j$ which is the sum of the vehicle-miles of travel on the freeway system instead of $\sum_{j=1}^{n} X_j$. The logic of this consideration was that if more vehicle-miles were traveled on the freeway then one might expect less vehicle-miles of travel being diverted to the surface streets. In addition, if the various ramps contributed a wide range in average trip lengths, a scheme which would be more restrictive to ramps contributing short trip lengths, would appear logical. The original model was modified to maximize $\sum_{j=1}^{n} \ell_j X_j$ and

TABLE 10

EQUAL QUEUE RESTRICTION PROBLEM

Input	Demand	Allowable Volume	Queue or Diversion	Value[a]
Ramp 1	600	532	68	1.0000
Ramp 2	475	407	68	1.0000
Ramp 3	450	382	68	—
Ramp 4	500	432	68	—
Ramp 5	825	757	68	—
Freeway	6800	6800	—	0.9731
Bottleneck	**Capacity**	**Actual Flow**	**Excess Capacity**	**Value[b]**
Section 1	5900	5860	40	—
Section 2	6000	5722	278	—
Section 3	6450	6450	—	—

[a]Increase in objective function per unit increase in input demand.
[b]Increase in objective function per unit increase in bottleneck capacity.

Note: Total output = 9312 veh/hr (objective function).

TABLE 11

Queue Constraint	Excess Capacity			Sum of Input Volumes
	Section 1	Section 2	Section 3	
none	none	+213	none	9364
unequal[a]	none	+218	none	9359
equal[b]	+40	+278	none	9312

[a]Assumed queue constraints of 100, 50, 50, 100, and 75 for the five ramps.
[b]Resulted in equal queues of 68 veh/hr per ramp.

analysis undertaken of the traffic data (2). The permitted input volumes were identical to the author's results which indicated that for the given example, maximizing input volumes actually did maximize vehicle-miles of freeway travel. The results of this modified model are given in Table 8. One would not expect that a selected control scheme would always maximize both criteria, and consequently the discussor suggests that this offers a possibility for further study.

The author suggested in his paper that two types of queue constraints could be placed on the model: $S_j \leq Q_j$ and $S_j = S_{j+1}$. In the first case, a maximum permitted queue is established for each ramp; in the second case, the queue length (number of vehicles denied access) would be the same for all ramps. These schemes appear plausible, particularly considering the more favorable effect of the public and the probability that certain ramps might handle the queues more effectively.

The results of such modified models for the given traffic situation are given in Tables 9 and 10. Table 11 compares the results obtained from the original model with these two queue constraints.

In the reduced capacity example, the permitted volume on ramp three was reduced to 63 veh/hr, and the author suggested that this ramp would probably be closed since the permitted ramp volume was so small. Consideration was given to undertaking this analysis assuming this ramp was closed ($X_{j=3} = 0$), and this possibility demonstrates the flexibility available with the linear programming technique. Further inspection of the author's reduced capacity example, however, revealed that the ramp closure would not increase the permitted ramp volume at any other ramp but would slightly reduce the freeway flow downstream of the ramp, and consequently, increase the level of service. This demonstrates another advantage of the linear programming technique in that the results of initially conceived models can be used to indicate appropriate directions for further refinements and improvements.

A further interpretation of the linear programming results is the determination of the effect of unit changes in constraints on the objective function. The author includes a description of this further interpretation in his earlier paper (7) but he did not include it here. Each table contained in this discussion includes such results. For example, in the original problem (Table 6), a unit increase in the bottleneck at Section 3 would result in an objective function increase of 0. 0670. In other words, increasing the capacity at this section by one vehicle increases the total ouput of the system by 0. 067 vehicles. On the other hand, the last column of Table 6 shows that increasing the capacity at Section 1 by one vehicle increases the total output of the system by 1. 0 vehicles. Consequently this would indicate that a greater benefit would result by increasing the bottleneck capacity at Section 1 than by doing so at Section 3.

Considering other study possibilities, there is a need for the quantitative measurements of the time variation of traffic demand and O-D desires. The application of linear programming for traffic control purposes requires that the various traffic demands and O-D desires be relatively constant over the time period selected. The author mentions this matter and suggests that 30-min intervals might be applicable and, in the case of severe congestion, the periods of analysis might be lengthened. Measurements of traffic demand available to the discussor indicate that selection of shorter time periods for applying linear programming techniques may be required. For example, on the San Francisco-Oakland Bay Bridge the westbound morning peak-hour volume averages approximately 8200 veh/hr. Congestion is not normally encountered in this direction at this location during the morning peak period and therefore the volume count is essentially the traffic demand. During the peak hour, however, the 6-min volumes (expressed as hourly rates) increase from 7200 to 9200 during the first half-hour and then decrease from 9200 to 7200 during the second half-hour. Such analyses of the time variation of freeway and ramp demands as well as O-D desires, in order to select appropriate time periods for linear programming applications, appear to be needed.

The time period determined by such studies will play a significant part in the final selection of a freeway control system configuration and of the size of computing facilities required if linear programming techniques are to be applied. A second avenue of possible studies then is directed toward the selection of freeway control system configurations. Undoubtedly different locations will require specially adapted control-system configurations, but perhaps some portion of the spectrum of control system ranging from a strictly time-clock operation to a second-by-second dynamic linear programming traffic-adjustable operation would be most suitable. The author suggests the need for such evaluations and proposed that both the costs and benefits of various configurations be estimated.

The final suggestion for possible study is the extension of the author's work toward the consideration of a network rather than a one-directional length of freeway. Optimizing the use of a directional freeway length may not necessarily be the same as optimizing the use of the network. For example, there are some who contend that the best strategy for network operations is to store the excess traffic demand on the freeway. Others suggest that in the event of a freeway capacity reduction event,

freeway controls should be terminated. The specific strategies for ramp control might be modified if alternate under-capacity routes (such as frontage roads) are available at some ramp locations but not at others. The engineer must consider the consequences of freeway control systems to surface street users as well as to freeway users, and therefore the total network being affected by the control system should be considered.

Acknowledgment

The discusser wishes to acknowledge the assistance received from John Brantley, a transportation graduate student at the University of California, in conducting the computer analyses presented in this discussion.

JOSEPH A. WATTLEWORTH Closure—The author most certainly appreciates the excellent reviews by Edward Gervais, Robert Foote, and Adolf May and would like to compliment the discussers for the depth and stimulating nature of the reviews. Many new ideas emerged from the written reviews and the subsequent discussions with the three authors.

It was gratifying that each of the discussions stresses the need for a systems approach to the problem of traffic control since this is the approach taken at the Texas Transportation Institute.

Dr. May and Mr. Foote deserve special credit for taking the time to analyze the linear programming model in detail and for even proposing models of their own. The author made several solutions to the proposed models to compare them with his model using the example problem in the paper and very little practical difference existed among them. While the practical differences are slight, philosophically I continue to favor the criterion of maximizing input (or really output) to the freeway since it is so [illegible]

[illegible] analysis in which the entire arterial street-freeway system is considered in the optimization. The proposed model does not do this. The network model, perhaps of the type used by Pinnell (9), would determine the proper amount of traffic to route or assign to the street system and to the freeways. When this has been done it would be surprising if a sufficient demand would be diverted from the freeways to the arterial streets to eliminate peak-period operational problems on the freeways. Whatever network analyses are performed it is probable that the final problem will be a freeway with more demand than capacity (otherwise there is no need for peak-period control) and the proposed model can be used to aid in the allocation of capacity to the demands.

All three discussers made the point that we have to determine how elaborate a control system is really needed, or rather, what control system is economically warranted. Probably they are implicitly questioning the justification for the elaborate detection system presented. This same question was also raised in the paper. What we are all saying, I believe, is that a Rolls-Royce solution has been proposed and possibly the need is more for a Volkswagen. This, of course, is an important area requiring further research and this is an important phase of the research planned at the Texas Transportation Institute.

Mr. Foote and Mr. Gervais raised the question of the stability of the control system. This is always a question in any control system and is another area of further research in the freeway control field. I believe that this matter can be resolved but this will be another area of intensive research on the Gulf Freeway Surveillance and Control Research Project during the coming year.

Dr. May and Mr. Foote both suggested that shorter time periods of analysis be used in the linear programming models because of the changes in the O-D patterns during the peak period. In some cases this may be desirable but realistic simulations cannot be expected if extremely short time periods are used because of the travel time in-

volved in passing through the system. I think that half-hour analysis periods would be about right for use in the analysis of the operation of the Gulf Freeway.

There is perhaps a more serious problem involved in the use of the L-P model which none of the discussors mentioned. While the O-D patterns may change a bit during the peak period, this pattern probably changes quite extensively when a control system is implemented. Thus, the use of O-D data obtained before the initiation of control can lead to quite misleading interpretations of the operation of a controlled facility.

The author would again like to express his thanks and gratitude to the three reviewers for their discussions of the paper. These discussions will certainly be of benefit in future research.

Evaluation of Entrance Ramp Control on a Six-Mile Freeway Section

CHARLES PINNELL, DONALD R. DREW, WILLIAM R. McCASLAND, and JOSEPH A. WATTLEWORTH, Texas Transportation Institute, Texas A & M University

•THE LEVEL of service on the inbound Gulf Freeway during the morning peak period has been well documented (1, 2, 3) as a result of research conducted by the Texas Transportation Institute in cooperation with the Texas Highway Department and the U. S. Bureau of Public Roads. In addition to the freeway studies, critical intersections on the arterial streets and freeway frontage road were studied and a capacity improvement was recommended at one very critical location (4). Several general techniques of studying freeway operations were also developed in order to complete the freeway analyses (5).

The operational studies on the inbound Gulf Freeway pointed to the need for some form of freeway ramp control. An initial control study (6) was conducted on a moderate scale in August 1964. The studies reported on here are an extension of this research effort.

Early operational studies (3) indicated that by controlling the inbound entrance ramps a significant improvement in the inbound freeway level of service could be achieved and the total travel time expended during the morning peak period could be greatly reduced. As a result of this research, a control study was initiated on five inbound freeway entrance ramps between Wayside Drive and Dowling Street (6). In this study four entrance ramps were closed and one entrance ramp was manually metered

control additional ramps between Wayside Drive and the Reveille interchange to further improve freeway operations and to permit greater use of ramp metering and less use of complete ramp closure (by spreading the excess demand over more ramps). The present study was developed to fill this need and to allow the evaluation of a trial ramp control signal installation at the Dumble entrance ramp.

This report presents the development of, preparations for, and results of Inbound Gulf Freeway Ramp Control Study II which was conducted between January 26 and March 12, 1965. In addition, the traffic operation after the termination of the control study was also studied and these results are presented. Evaluation of the operation during the control period centered mainly on the freeway but also included the inbound frontage roads and the arterial street system.

BEFORE STUDIES

Freeway and Frontage Roads

Traffic studies were conducted on the inbound Gulf Freeway during 1964 and early 1965 in order to identify the critical bottleneck locations and to determine the duration and amount of excess demand at each of these locations. All studies were conducted during the 6:30 and 8:30 a. m. period between Broadway and Dowling streets.

Closed system input-output studies conducted during January, March, and April, 1964, provide much of the basic volume, density and system travel time data on which the before-and-after comparisons are based. Table 1 shows the dates of each input-output study which was used for the "before" data in the before-and-after comparisons. These studies have been reported previously (3) and were used in the development of the plans

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TABLE 1

1964 INPUT-OUTPUT STUDY DATA USED IN BEFORE-AND-AFTER COMPARISONS

System Boundaries	Data Used
Broadway-Griggs	Jan. 28-30, 1964
Griggs - S. HB&T RR	March 16, 17, 20, 1964
S. HB&T RR - Cullen	March 21, April 1, 2, 1964
Cullen - Scott	April 13, 17, 1964
Scott - Dowling	April 20, 23, 24, 1964

for the Inbound Gulf Freeway Ramp Control Study I (6) which was conducted in August 1964.

While the January 1964 input-output studies were being conducted, a specially equipped vehicle was used to obtain data on travel times for an individual vehicle. These data are presented in the form of travel time contours and are compared to similar data obtained during the control study reported herein.

Aerial photographs were also used in the collection of data during the summer of 1964 and during the January-April period of 1965. These data were used to supplement the density and system travel time data obtained from the input-output studies and to provide data on the operation of the frontage roads and arterial street intersections near the freeway. Such data from early 1965 were incomplete because the light was insufficient for good photography during the early part of the 7-8 a. m. peak period which was used in most analyses.

Arterial Streets

The term "Freeway Control" refers to the control of the input volumes to the freeway at the entrance ramps. Controls of this type increase traffic on the arterial street system, because of diversion from one entrance ramp to another, or to the street system for the entire trip. Also, the control system, if successful, will increase the output of the freeway so that exit ramp volumes may increase for short periods of time. The objective of this phase of the study was to determine the change in travel on the street system and to determine if traffic flow was impaired or penalized by the control system.

To determine the total effect of freeway control over the system of streets and freeways, the studies on all traveled ways had to be compatible, or comparable. However, input-output studies that describe so well the conditions on the freeway were not practical on the arterial streets. The numerous entrances and exits to a system of any length on arterial streets would require a very large number of observers. Aerial photography could provide the data if trees, buildings and shadows did not interfere with the line of sight.

A network of arterial streets essentially consists of sections of uncongested roadways and sections of congested roadways, even during peak traffic flows. The congested sections are the approaches to signalized intersections. A certain number of stopped vehicles is to be expected at almost any signalized intersection. Only when an approach becomes saturated, so that some vehicles are delayed for more than one cycle, can we say that the approach is congested. To estimate the effect of ramp closure on the arterial streets, approaches to many intersections on arterial streets considered to be located in the area influenced by the freeway were studied.

The procedure of study involved taking demand counts on the approaches from 6:30 a. m. to 8:30 a. m. for several days before the control study period. From the demand or input count and the output count on an approach to an intersection, the delay in vehicle minutes could be estimated, where "delay" is defined as the delay to vehicles not clearing the signal during the first green phase after arrival at the signal.

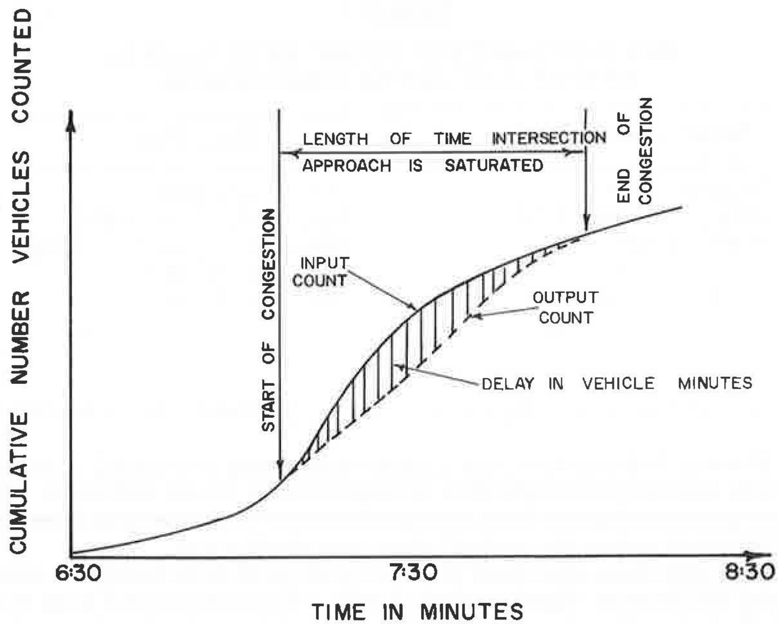


Figure 1. Capacity demand relationships.

This delay, or excess delay, can be illustrated by plotting the cumulative input and output counts as shown in Figure 1.

The shaded area represents the delay to vehicles, as defined above, in vehicle-

slightly over the time period. It is normally assumed that the capacity of an approach is constant, i. e., that this output count is a straight line. It was felt that accepting this as true would introduce only small errors in the estimates of delay. Furthermore, in view of the fact that the difference in delays before and during the study period were of primary interest, this assumption would probably introduce only negligible error into the estimated difference. The capacity of each approach under study was thus measured by taking output counts during saturated cycles and evaluating the average maximum output (capacity). Then only demand counts were required at the approaches under consideration.

This technique is not well refined and is not considered as a highly accurate estimate of the exact value of the delay, mainly because both the demand and the output are considered as continuous rather than discrete functions. However, any serious effect on the delay on an approach will certainly show up in such an analysis and it was thus considered adequate for the purposes of this study. In addition, travel time runs were made by individual vehicles to provide another means of determining the effect of the freeway control on traffic operations on the surface street system.

The closure of the Griggs Entrance Ramp for a 15- or 20-minute period during the peak hour was of major concern since 170 to 200 vehicles would be diverted to the city street system in an area that had several congested intersections. A license plate origin-destination survey of the ramp traffic was conducted to determine the alternate routes and ramps that this traffic would probably use to enter the freeway. The reassignment of this traffic was used to determine the time the Griggs Ramp should be open and the metering rates for the downstream entrance ramps.

DEVELOPMENT OF CONTROL PLANS

Philosophy of the Controls

The philosophy of Control Study II was essentially the same as that of Control Study I (6), namely that the demand be kept less than or equal to the capacity at each bottleneck. Demand and capacity both represent total directional flow rates (three lanes). In Control Study I the control area was limited to the region between Wayside Drive and Dowling Street, whereas in Control Study II all inbound entrance ramps between Broadway and Dowling were considered for control.

Upstream of each inbound entrance ramp (starting in the Broadway area and proceeding toward Dowling) the 5-minute demand rates were estimated (6, 7). For each entrance ramp, the difference between the estimated upstream demand and the estimated capacity provided the basis for metering or closing the ramp.

The capacity flow rates for the critical sections were based on counts obtained from March to August 1964. These capacities were somewhat higher than would have been obtained from the counts during January and Febraury 1964, since the January-February 1964 counts were found to be considerably lower than counts obtained at the same locations in the March-August 1964 period. The original control plan which was based on these higher capacities was tested during the first four days of the study (January 26-29, 1965). In this period the improvement in the freeway level of service was not as great as was anticipated so the controls were made slightly more restrictive at the beginning of the second week to compensate for the possible overestimation of capacities. Most of the discussions in this report refer to the revised control plan, since this was in effect much longer than the original plan.

Location and Severity of Controls

The demand estimates and various other studies showed that no controls were required at the Detroit Street entrance ramp or at any entrances upstream of this location. The congestion normally does not back upstream to the Broadway entrance ramp and the demand was found to be less than the capacity at the Detroit entrance ramp throughout the entire peak period. Similar considerations also indicated that control of the Scott entrance ramp was unnecessary.

Control was considered at each entrance ramp from S. H. 225 to Cullen Street, as shown in Figure 2. A discussion of the considerations leading to the final control plan at each ramp follows.

S. H. 225 Entrance Ramp.—This entrance ramp is a directional turning roadway which accommodates the right-turning vehicles from the southbound S. H. 225 (La Porte Freeway) to the inbound Gulf Freeway.

Some merging problems were anticipated at this location but the metering of this high-volume ramp would probably have created a queue which almost certainly would have backed onto the La Porte Freeway, blocking one of its two southbound lanes. Even though the volumes on the La Porte Freeway are less than the capacity of one lane, it was decided that the possible benefits to the Gulf Freeway traffic of metering this ramp did not outweigh the possible adverse effects (especially the accident hazards) to the La Porte Freeway traffic.

From a design standpoint the S. H. 225 entrance ramp is one of the best entrance ramps on the inbound Gulf Freeway. However, under normal operating conditions on the Gulf Freeway, many vehicles bypass the S. H. 225 entrance ramp and enter at Woodridge or Mossrose. By doing this, they miss a great deal of freeway congestion but tend to compound the problem by entering downstream at more critical locations. For this reason, it was deemed advisable to leave the S. H. 225 entrance ramp uncontrolled in order to encourage greater usage of it. Thus, for the purposes of Control Study II, the S. H. 225 ramp was not metered.

S. H. 35 Entrance Ramp.—The 5-minute demand rates upstream of the S. H. 35 entrance ramp were estimated based on count data from the input-output studies (Table 1). The allowable metering rate for each 5-minute period was calculated as the difference between the merging capacity and the upstream demand. A capacity of 475 vehicles per 5 minutes was used in the derivation of the initial control plan.

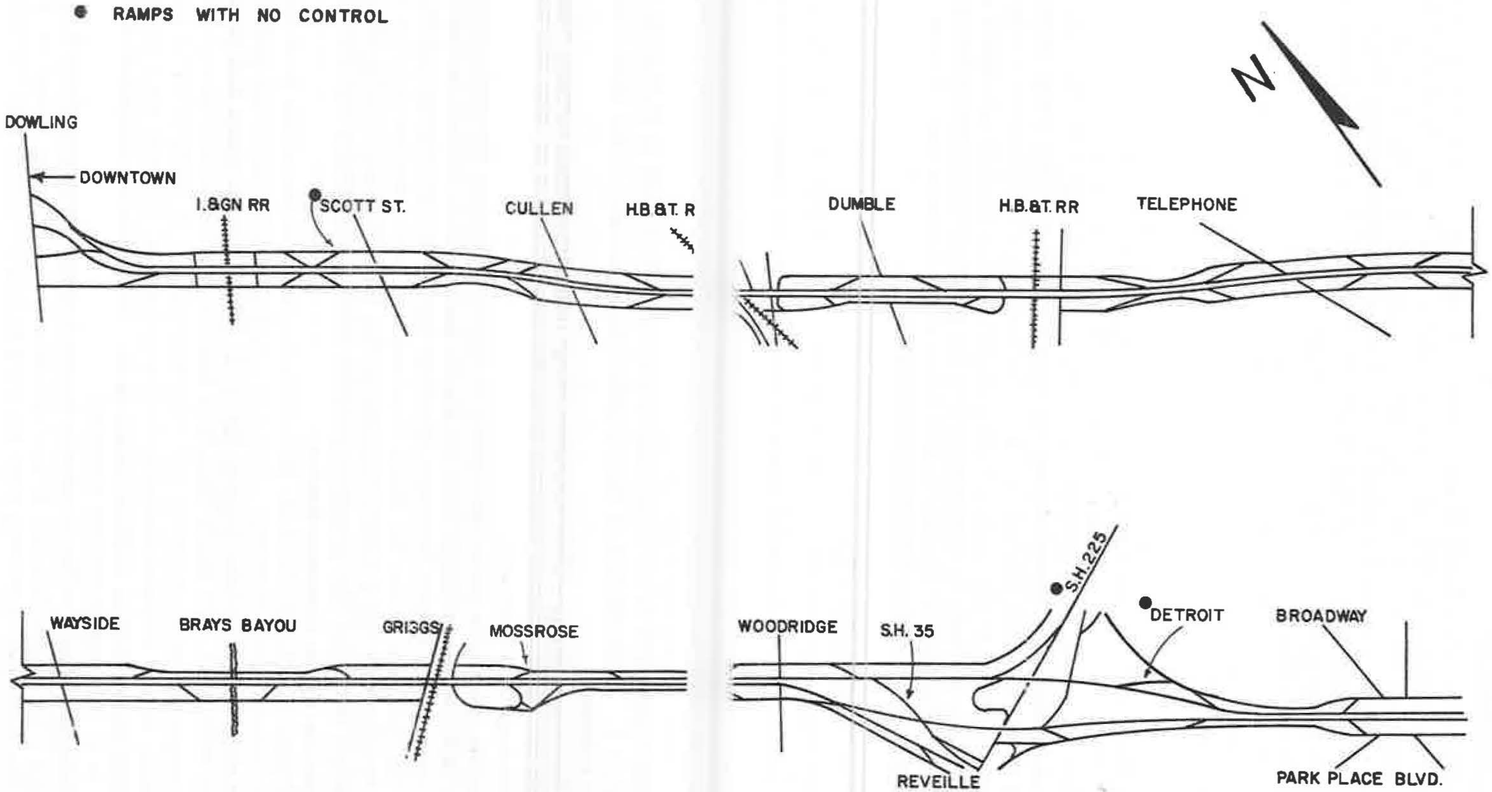


Figure 2. Gı reeway study system.

The initial control plan was in effect during the first four days (January 26-29, 1965) of this study. The metering scheme used at the S. H. 35 entrance ramp during this period is shown below:

Time—a. m.	Metering Rate	
	Veh/5 min	1 Veh/x sec
6:55-7:00	75	1/4
7:00-7:05	50	1/6
7:00-7:15	75	1/4
7:15-7:30	100	1/3

The metering rates are shown in terms of both the number of vehicles per 5 minutes and the metering headway in seconds. In addition, the personnel operating the metering station were instructed to discontinue the metering if ramp vehicles had to stop before merging and queued in the merging area past a predetermined point (about 12-15 vehicles in the queue). When the queue cleared to another predetermined point (about 3 or 4 vehicles in the queue) the metering was resumed.

At the beginning of the second week of the control study, a slightly more restrictive control plan was initiated. This plan was in effect until the controls were terminated on March 12, 1965. The revised metering scheme at the S. H. 35 entrance ramp is shown below:

Time—a. m.	Metering Rate	
	Veh/5 min	1 Veh/x sec
6:55-7:20	50	1/6
7:20-7:45	75	1/4

The override for stopped vehicles in the merging area was also used in this scheme.

A queue of considerable length was anticipated at this location because of the high demand rate and the metering rates which were used. The storage of the vehicles in the queue was not considered critical because of the length of the ramp and the fact that up to 100 vehicles were queued at this ramp on some days when no controls were in effect.

Woodridge Entrance Ramp.—Under normal freeway operating conditions about 45 percent of the vehicles which enter the freeway at Woodridge did so after bypassing S. H. 225 entrance ramp. Thus, it was assumed that when the Woodridge entrance ramp was metered the vehicles would not bypass the uncontrolled S. H. 225 ramp to wait in the queue at Woodridge. It was anticipated, however, that about 20 percent of the vehicles which normally enter the freeway at Mossrose would choose to enter at Woodridge. Hence, the expected ramp demand at Woodridge during the control plan was 55 percent of the normal Woodridge demand plus 20 percent of the normal Mossrose demand.

The freeway demand was estimated upstream of the Woodridge entrance ramp and a merging capacity of 485 vehicles per 5 minutes was assumed in the calculation of the allowable metering rates. The controls at S. H. 35 were taken into consideration when the freeway demand was estimated. The metering scheme used during the first week of the controls at Woodridge was as follows:

Time—a. m.	Metering Rates	
	Veh/5 min	1 Veh/x Sec
6:55-7:10	30	1/10
7:10-7:20	50	1/6

During the remainder of the control study the rate of 50 vehicles per 5 minutes (1/6 seconds) was extended from 7:20 to 7:45 a. m. Otherwise, the control scheme at this location was unchanged.

An override to the metering, similar to that at the S. H. 35 ramp, was used to clear vehicles from the ramp which had to stop in the merging area to wait for an acceptable gap. When four ramp vehicles were stopped in the merging area the metering was temporarily halted until only one stopped vehicle remained. Then the metering was resumed. This same override to the metering was used at the Mossrose, Wayside and Telephone entrance ramps.

Mossrose Entrance Ramp.—The geometric features of the Mossrose entrance ramp make it one of the most critical merging areas on the inbound Gulf Freeway. The ramp itself is very short and provides a high-angle, direct entry onto the freeway. In addition, the ramp enters the freeway at the foot of the upgrade of the Griggs Road overpass structure on which a difference of elevation of about 30 feet occurs in a distance of about 1000 feet. Because of the upgrade, vehicles which have to stop or slow down drastically in the merging area have a severe adverse effect on the freeway traffic. Because of the high ramp volume (approximately 650 vehicles from 7 to 8 a. m.) and the inferior ramp geometrics, a great many ramp vehicles are forced to stop before merging.

The results of an origin-destination study conducted at this ramp (8) showed that about 80 percent of the vehicles which normally enter at Mossrose bypass either the

freeway. Thus, only about 20 percent of the vehicles which normally enter at Mossrose should enter there; the other 80 percent should enter at S. H. 225 or Woodridge.

In addition to the demand-capacity philosophy used to plan the controls at the other ramps, other considerations were also made. An extremely low metering rate was considered desirable during the early part of the peak period at this ramp for three reasons: (a) a low metering rate would allow most of the ramp vehicles to enter the freeway at high speeds, (b) a low metering rate and its associated high delay would discourage vehicles from bypassing upstream ramps, and (c) the low metering rate at Mossrose was expected to produce a higher level of service on the freeway, thereby encouraging some vehicles to enter the freeway upstream of Mossrose rather than bypassing to Mossrose.

Considerable thought was given to the possibility of closing this ramp instead of metering it. One disadvantage of metering the Mossrose ramp was that the personnel and the various signs involved in the metering would be plainly visible to the motorists on the freeway. Thus, the possibility of the formations of a "gapers block" (a traffic slowdown caused by drivers looking at an accident, disabled vehicle or other distraction which is not actually blocking their path) existed. It was decided, however, that it would be better to meter than to close this ramp to avoid causing circuity of travel for approximately 125 vehicles whose trips originate near the ramp and for which the use of the ramp is most natural. The discontinuity in the frontage road at Griggs Road makes the Mossrose ramp especially important for these vehicles. It was reasoned, however, that a higher level of service on the freeway would probably have resulted from the closure of this ramp.

During the first week of the control the following metering scheme was used at the Mossrose entrance ramp:

Time—a. m.	Metering Rate	
	Veh/5 min	1 Veh/x sec
6:55-7:10	20	1/15
7:10-7:20	30	1/10
7:20-7:30	50	1/6

During the remainder of the control study the metering scheme was changed as follows:

Time—a. m.	Metering Rate	
	Veh/5 min	1 Veh/x sec
6:55-7:20	20	1/15
7:20-7:45	50	1/6

In order to allow stopped vehicles from the ramp to clear from the merging area, the same override to the metering that was used at Woodridge was employed at Mossrose.

Griggs and Wayside Ramps.—Because of the proximity of the Griggs and Wayside entrance ramps and because there are no ramps between them, the controls imposed at one ramp would affect the controls needed at the other ramp. The demand upstream of the Griggs ramp and the capacity downstream of the Wayside ramp determine the allowable entrance volume for the two ramps together. This allowable volume could come from either ramp or a combination but the total volume entering from the two ramps must not exceed the total allowable volume. For this reason control considerations were made at the two ramps simultaneously.

If a large amount of traffic were allowed to enter the freeway from the Griggs ramp, it would have been necessary to impose a low metering rate on the Wayside entrance ramp. The problem of a traffic queue at Wayside, especially the possibility that it might back into the intersection of Wayside Drive and the inbound frontage road, made it unfeasible to have an extremely low metering rate at Wayside. As explained previously, the metering rate could be increased at Wayside only through a corresponding decrease in the metering rate at Griggs. Because of the high volume on the Griggs ramp (about 700 from 7-8 a. m.) a low metering rate there would have created severe queueing problems at this location. Even though the frontage road there could accommodate a large queue without having any intersections blocked, the possibility of a gapers' block formation on the freeway was considered to be great.

Since the allowable ramp volume at Griggs (with a high metering rate at Wayside) was so low during part of the peak period the ramp would have been essentially closed. This suggested the possibility of closing the ramp instead of metering it. The advantages of closure are its simplicity and the elimination of a large queue at this ramp. One disadvantage of closure is that the intersection with Wayside Drive is a critical bottleneck on the inbound frontage road (4) and the diversion of a large amount of traffic through this already congested intersection approach would certainly create an extremely bad situation.

It was decided that the frontage road congestion was less critical and the decision was made to close the Griggs entrance ramp. A reassignment of some of the traffic from this ramp indicated that most of this traffic could use more direct routes to other freeway ramps and would not use the frontage road between Griggs and Wayside. Demand-capacity analyses of the alternate routes to be used by most of the traffic diverted by the closure of the Griggs ramp indicated that until about 7:20 a. m. these

routes could accommodate the extra traffic. However, after about 7:20 a. m. the additional traffic would be expected to cause severe congestion and excessive delay on many of the alternate routes.

Because of these considerations the decision was made to close the Griggs entrance ramp from 7:05 to 7:20 in spite of the fact that the demand would be close to or slightly over the capacity of the merging area of this ramp for several minutes after the ramp was opened. Were it not for the consideration of the effects on the arterial street system this ramp should desirably have been closed until 7:25 or 7:30 to preserve the high level of service on the freeway.

Demand-capacity considerations (assumed capacity 490 veh/5 min) at the Wayside entrance ramp, including the effects of the closure of the Griggs ramp on freeway and ramp demand, led to the establishment of the following metering scheme:

Time—a. m.	Metering Rate	
	Veh/5 min	1 Veh/x sec
7:00-7:05	30	1/10
7:05-7:20	50	1/6
7:20-7:25	38	1/8
7:25-7:30	50	1/6

After the first week of the study the following metering scheme was adopted:

Time—a. m.	Veh/5 min	1 Veh/x sec
7:00-7:25	30	1/10
7:25-7:45	50	1/6

The same override to the metering to clear stopped ramp vehicles from the merging area was used at the Wayside, Telephone and Dumble ramps while they were being metered.

Telephone Entrance Ramp.—A demand-capacity analysis at the merging area of the Telephone Road entrance ramp provided the basis for the metering plan which was used there. Consideration was made of all controls on upstream entrance ramps and the reassignment of the vehicles which would be diverted from the Griggs Road ramp during its closure period. About 50 percent of these were assigned to the Telephone Road entrance ramp and a merging capacity of 490 veh/5 min was assumed. The following metering scheme was developed for the Telephone entrance ramp:

Time—a. m.	Metering Rate	
	Veh/5 min	1 Veh/x sec
7:00-7:15	20	1/15
7:15-7:45	40	1/7. 5

Dumble Entrance Ramp.—The Dumble entrance ramp was selected as the location for testing a traffic signal for metering the ramp traffic. This installation will be discussed more fully in a later section.

A demand-capacity analysis of the merging area of the Dumble entrance ramp was made. It included the effects of upstream controls and the expected diversion of vehicles from their normal routes and assumed a merging capacity of 475 veh/5 min. This analysis yielded the following metering plan which was in effect during the first week of the study:

Time—a. m.	Metering Rate—Veh/5 min
7:05-7:10	40
7:10-7:15	30
7:15-7:20	25
7:20-7:25	40
7:25-7:30	50

During the second week of the study the rate of 50 vehicles per 5 minutes was extended until 7:45 a. m.

At the beginning of the third week an automatic timing device was installed at the signal to eliminate the necessity of having someone present to turn the signal on and off. At this time it was necessary to limit the metering to one rate during the entire period of control. The rate of 40 vehicles per 5 minutes was chosen.

Cullen Entrance Ramps.—The demand-capacity analysis at the merging area of the two Cullen entrance ramps indicated that these ramps should be closed from 7:05-7:30 a. m. each day. Because the closure of these ramps on two other occasions (1, 6) resulted in few, if any, problems to motorists normally using them, the decision was made to close the two ramps for the 25-minute period.

PRELIMINARY PREPARATIONS FOR CONTROLS

Advance Publicity

The details of the control plan were announced to the general public through a news release issued on January 19, 1965. The Appendix contains several articles regarding the controls which appeared in local newspapers. Also on January 19, signs were erected at each of the ramps to be controlled, displaying the date and time of control. A traffic bulletin was issued to each motorist who used the ramps in the study area during the morning peak period on January 25 as a reminder that the study would start the next day and also to indicate the extent of the study area. Figures 3 and 4 show samples of the ramp signs and traffic bulletins.

The effectiveness of the advance publicity was evident the first few days of the study by the changes in the pattern of traffic approaching the study area. In many instances the shift in traffic was not expected. Many motorists tried to bypass part of the control area to enter the freeway at downstream ramps.

Signing

In addition to the advisory signs erected at the ramps, alternate routing signs such as the one shown in Figure 5 were located at upstream entrance ramps and on the arterial streets at major intersections, and portable "stop" and "stop ahead" signs were placed at the ramps.

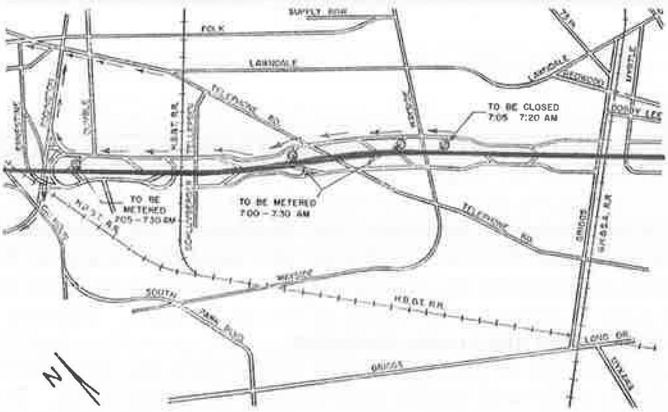
Signals

The signals at the Dumble entrance ramp were installed on Friday, January 22 and operated on flashing amber until the start of the control study January 26, as described in detail later.

Figure 3. Advisory signs erected at ramps to be controlled.

FREEWAY SURVEILLANCE AND CONTROL PROJECT

TEXAS TRANSPORTATION INSTITUTE
COOPERATIVE RESEARCH
WITH
TEXAS HIGHWAY DEPARTMENT – CITY OF HOUSTON – BUREAU OF PUBLIC ROADS



As part of an Experimental Freeway Control System to improve traffic operations, the entrance ramps to the Gulf Freeway from State Highway 225 to Downtown will be controlled during the morning peak traffic demand. The traffic using this ramp will be metered by limiting the number of vehicles to enter the Freeway from 7:00 to 7:30 a.m. on weekdays from January 25 to February 5. This will result in some additional delay to ramp traffic during this time period. Motorists who prefer to bypass the control area are advised to take one of the alternate routes shown on the map above.

The map on the reverse side of this traffic bulletin indicates the controls to be placed on the other ramps. It should be noted that traffic diverting to one of the other ramps during the time of control may be delayed several minutes before entering the Freeway.

Figure 4. Traffic bulletin issued to motorists using ramps.

Figure 5. Advisory signs on alternate routes.

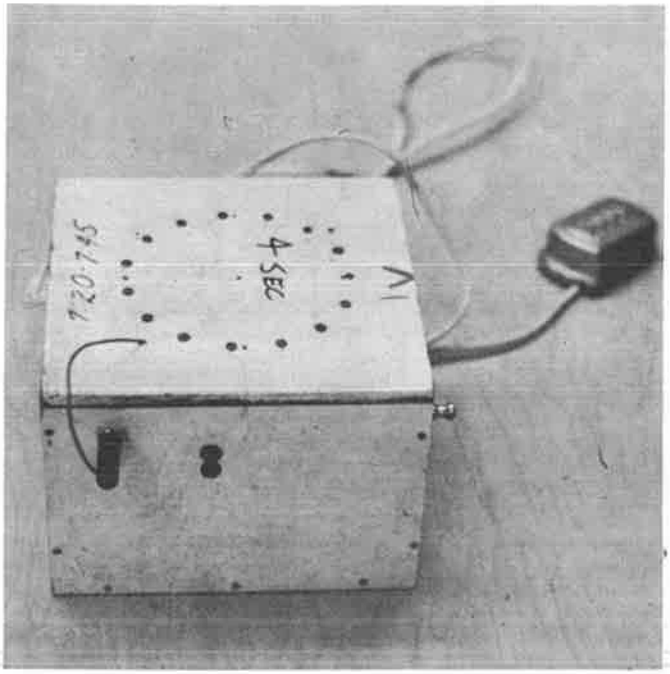


Figure 6. Timer used for ramp metering.

Timing Devices

The metering rates were determined on the basis of capacity-demand relationships on the freeway. Timers (Fig. 6) that indicated the various metering rates to the ramp control officers were built for the study. They provided a buzz each time a vehicle on the ramp was to be released onto the freeway. The design was crude but effective. A 6-volt motor with a speed of 1 rpm, mounted in a 6- by 6- by 4-inch box, powered a rotary switch. Pins in the lid of the box closed the circuit to a buzzer. The metering rate, represented by the number of pins, was adjusted by changing the lid.

OPERATION OF THE CONTROLS

Manual Metering

Five of the metered ramps were controlled by policemen from the City of Houston, who directed the traffic onto the freeway at specified intervals. The sixth ramp was controlled by a fixed time traffic signal for assigning the right of way to ramp traffic.

The metering stations controlled by the city policemen were located at the junction of the entrance ramps and the frontage road as shown in Figure 7. A stop-ahead sign was placed 200 feet in advance of the metering station. A stop sign was placed at the metering station, but it was easily seen by the freeway motorists and tended to cause a gapers' block. The stop signs were removed after the second day on all but one ramp.

The policemen were instructed to direct one vehicle onto the ramp each time the buzzer on the timer sounded. If the vehicles did not move directly into the freeway, but queued up at the merge point, the policemen were instructed to hold all vehicles at the metering station until only one vehicle remained on the ramp. The policemen changed from one metering rate to another at the times specified in the control plan by a simple adjustment of the timer.

Figure 7. Ramp metering location.

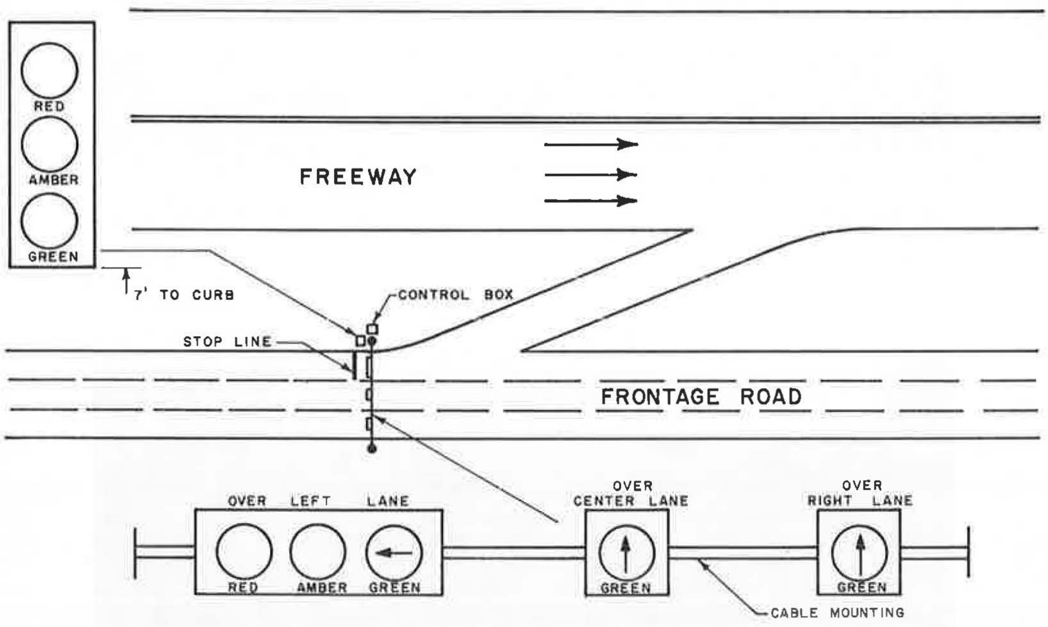


Figure 8. Schematic of signal installation at the Dumble entrance ramp.

Signal Installation at Dumble

One of the most crucial components of a ramp metering system is the type of con- traffic control techniques not commonly encountered by motorists, it is especially necessary that the basic elements of control devices be realized in the development of a ramp control system. The control devices employed must therefore compel the attention of the motorist and must present a message that is clearly understood. Proper location of the control device should allow ample time for the motorist to respond and apply appropriate actions as required by the device.

Ramp metering requires a more sophisticated type of control device than fixed-time ramp closure. It is realized that a metering device must not only be able to meter effectively, but it is envisioned that the device must also operate to close a ramp completely at certain high volume periods on the freeway.

The objective of this phase of the study was to observe some of the characteristics associated with semiautomatic metering in order to determine equipment requirements for future automatic systems. Some aspects of metering considered in the design of the Dumble experiment were the location and type of control signal and the signal phasing.

The selected location of the metering control was on the frontage road as opposed to a location on the ramp. Three advantages were anticipated: (a) a ramp could be closed without trapping a driver on the ramp, (b) signalization of the frontage road-ramp would be similar to operation at a normal intersection and therefore be less of a novelty to the driver, and (c) there is less chance of a metered driver given the green to assume that he has the right-of-way in the merging situation with the freeway.

The initial study was directed toward determining driver requirements with respect to metering. Since driver responses to a signal using an amber phase following the green were to be evaluated, as against using just the red and green phases, a post-mounted traffic signal with red, amber and green lenses was installed adjacent to the stop line. Overhead signals mounted over each lane of the frontage road were employed to separate the two movements (ramp usage and frontage road usage). Figure 8 shows a diagram of the installation.

The phasing was designed for bulk-service metering. A three-dial pretimed controller was utilized with a constant 30-second cycle length. The three dials were set to give 10½, 8 and 13½ seconds of green with a constant amber of 2½ seconds. Dial No. 1 was used from 7:05 to 7:10 and from 7:20 to 7:25, dial No. 2 from 7:10 to 7:20 and dial No. 3 from 7:25 to 7:35.

In order to evaluate the proposed metering operation, the plan was (a) to measure starting headways in order to compare actual metering rates with the theoretical rates, (b) to record the number of violations by motorists who either ignored the signal or did not understand its significance and (c) to measure gap acceptance characteristics of ramp vehicles in the merging area and compare them to characteristics observed during normal operation.

Ramp Closure

Cones and barricades were placed in the outer separation at each of the three ramps that were closed. One city policeman was assigned to each ramp to effect the closure. At the time designated in the control plan, the cones were placed parallel to the frontage road across the entrances to the ramps, and the barricades were placed across the ramp roadway. The policemen were instructed to move away from the ramps and out of the line of sight of the freeway traffic.

At the end of the first two weeks of the control study, a reassignment of personnel was made to extend the study. It was decided at this time that the closure of the ramps did not require a person with police authority. The maintenance department of District 12, Texas Highway Department, which has the responsibility for closing the freeway when the roadway is made impassible by weather, accidents, or maintenance operations, assigned personnel to the project for the remainder of the study. During the final five weeks of the operation, the highway personnel encountered no difficulty in effecting the ramp closures at the specified times.

IMMEDIATE EFFECTS OF THE CONTROLS

The evaluation of the effects of the controls on freeway operation is divided into two time periods, the first two weeks of the study and the last five weeks, which include the entire seven weeks of the operation of the control study. The results obtained during the first two weeks are classed as immediate results while the results from the next five weeks are classed as long-term results.

The evaluation of the effects of the freeway controls on arterial streets is based on studies taken over the seven weeks of operation, but the results will be included in the section on immediate effects of the controls.

From 1500 to 2000 motorists were directly affected by the controls. That is, about 1500 to 2000 motorists normally entered the freeway at the ramps which were controlled during the time that each was controlled. It was anticipated that many motorists would search out new routes to avoid the controls or to enter the freeway at points further upstream because of improved freeway traffic operation. With such a large number of motorists suddenly having their normal travel routine changed, a period of transient system behavior was expected before a steady-state condition was achieved.

Such a transient condition was noted and is the reason for separation of the analysis of the data into two periods. During the first week of the study, the transient effect was especially evident as the freeway level of service consistently improved during successive days of the study. This can be seen in Figure 9. For this reason, only the data from the second week of the study were used in the freeway analyses of the immediate effects of the controls.

The transient effects on the arterial streets were not so noticeable. Therefore, it was decided to concentrate the studies on the freeway system to provide adequate coverage over the two time periods of control, and to continue all arterial street data obtained during control.

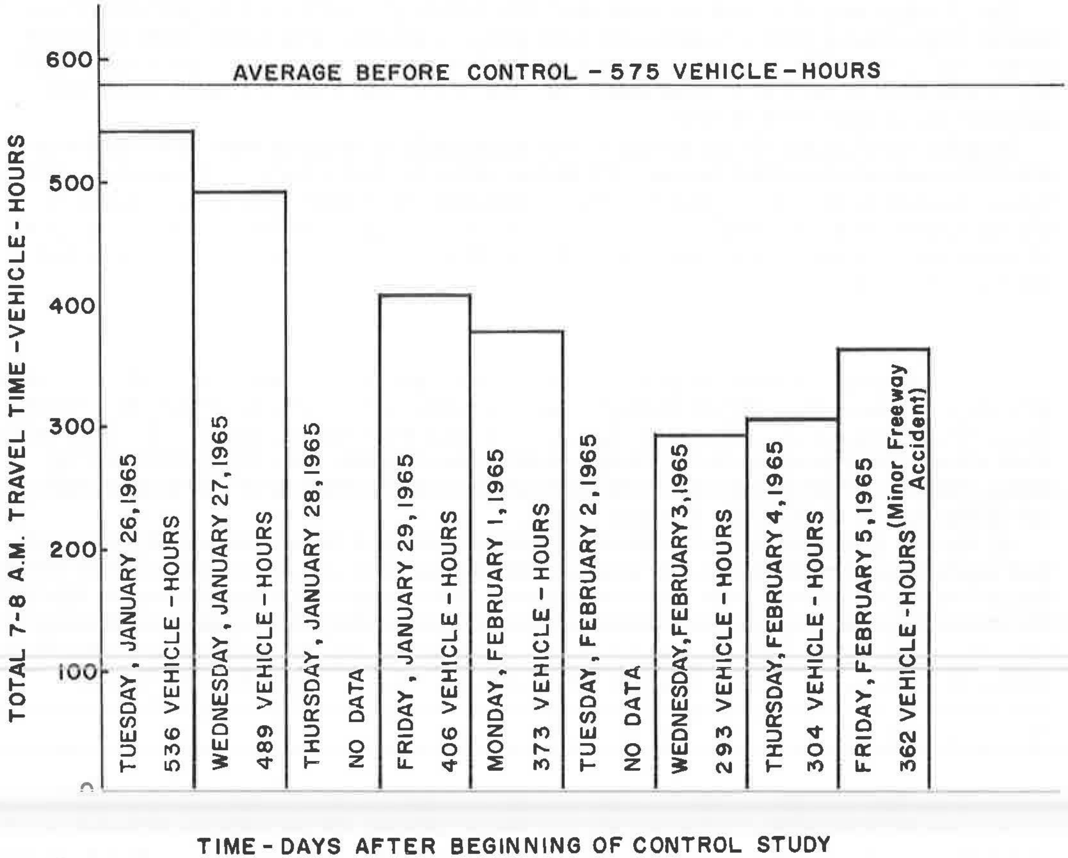


Figure 9. Transient behavior of total 7-8 a.m. travel time in Broadway-Griggs subsystem at the beginning of the control study.

Freeway and Frontage Roads

Three data collection techniques were used during this study—manual counts and speed recordings (most of which were part of the closed system input-output studies), aerial photography and moving vehicle travel time recordings. The aerial photography data were used exclusively in the frontage road studies and in the freeway studies from the South HB&T RR overpass to Dowling and were also used to supplement and to check the data from the (manual count) freeway studies. The basic analysis period is 7-8 a. m. but due to insufficient light conditions the aerial photographs did not encompass this entire period on some days. Hence, the frontage road data necessarily represent best estimates based on the photographs available. This same statement holds true for the evaluation of freeway operations from the South HB&T RR overpass to Dowling Street.

Table 2 contains the schedule of data used for the "after" portion of the before-and-after studies on the effects of the controls on the freeway and frontage road operation.

Total System Travel Time.—The total amount of travel time expended by all vehicles using a particular facility during the peak period is one good measure of its operational efficiency. The units of this travel time are vehicle-minutes or vehicle-hours. A vehicle-minute represents one vehicle in the system for one minute; a vehicle-hour represents 4 vehicles in the system for 15 minutes each, 5 vehicles in the system for 12 minutes each or some other combination totaling 60 vehicle-minutes.

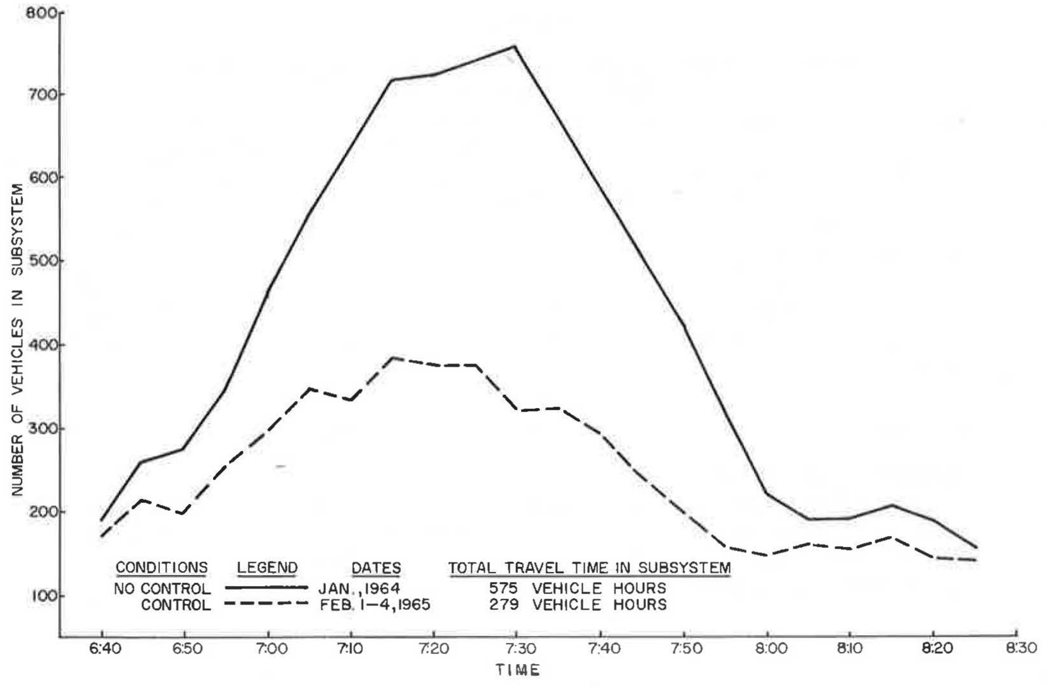


Figure 10. Number of vehicles on Broadway-Griggs subsystem.

TABLE 2

"AFTER" DATA USED TO DETERMINE IMMEDIATE EFFECTS OF CONTROLS

Manual counts (input-output)	February 2-4, 1965
Aerial photography	February 2, 3, 1965
Moving vehicle travel times	February 2-4, 1965

If the number of vehicles in a given system is a known function of time (such as a graph) the total travel time in the system is the integral of the time function (or the area under the graph) between the times of interest (3). This analysis was made during the 7-8 a. m. peak hour for the Gulf Freeway studies. Thus, for each freeway subsystem and the inbound frontage road the total travel time was calculated for the before-and-after comparisons.

The portion of the freeway on which the greatest operational improvement was anticipated was that between Broadway and Griggs (that is, the Broadway-Griggs subsystem). An improvement in the operations in this subsystem would be reflected by fewer vehicles in the subsystem (lower density). The number of vehicles in the Broadway-Griggs subsystem before and during the controls are shown in Figure 10. The number of vehicles in this subsystem decreased significantly during the control study and the 7-8 a. m. total travel time in this subsystem decreased from 575 vehicle-hours to 297 vehicle-hours, a 48 percent reduction.

Table 3 is a summary of the total travel time before and during the control study on the inbound freeway and frontage road. The total travel time on the inbound freeway was 371 vehicle-hours (30 percent) less than it was before the controls were put into effect. The travel time on the frontage road increased from about 190 to 201 vehicle-

TABLE 3

IMMEDIATE EFFECTS OF CONTROLS—TOTAL SYSTEM TRAVEL TIME, 7-8 A. M.

System or Subsystem	Total Travel Time, Vehicle Hours		
	Before Control	During Control	Difference
Inbound freeway			
Broadway to Griggs	575	297	-278 (-48%)
Griggs to S. HB&T RR	367	310	-57 (-16%)
S. HB&T RR to Dowling	302	266[a]	-36 (-12%)
Total inbound freeway	1244	873	-371 (-30%)
Inbound frontage road	190[a]	201[a]	+11 (+6%)
Total inbound freeway and frontage road	1434	1074	-360 (-25%)

[a]Based on incomplete data caused by inadequate light conditions which made aerial photography during the early parts of the 7-8 a.m. period impossible.

TABLE 4

AVERAGE 7-8 A. M. VOLUMES OF ENTRANCES TO THE FREEWAY

[illegible]	Before Control	During Control	
Freeway near Broadway	2831	3185	+354
Detroit on ramp	218	122	-96
S. H. 225 on ramp	559	649	+90
S. H. 35 on ramp[a]	818	726	-92
Woodridge on ramp[a]	426	398	-28
Mossrose on ramp[a]	643	318	-325
Griggs on ramp[b]	683	496	-187
Wayside on ramp[a]	335	332	-3
Telephone on ramp[a]	413	356	-57
Dumble on ramp[a]	345	294	-51
Cullen on ramps (combined)[b]	574	348	-226
Scott on ramp	63	257	+194
			(-5. 3%)
Total	7908	7481	-427

[a]Ramps which were metered.
[b]Ramps which were closed.

hours, a 6 percent increase. The total effect on the inbound freeway and frontage road travel time was a reduction of 360 vehicle-hours, which represents a 30 percent decrease.

Average 7-8 a. m. Freeway and Ramp Volumes.—The initiation of the control plan naturally caused some significant changes in the 7-8 a. m. volumes on the entrances to the freeway. Table 4 contains the average volumes before and during the control study. As can be seen, a large increase in volume took place on the freeway near

Broadway. This is attributable to the improved level of service on the freeway and to the controls on the downstream ramps. Some vehicles undoubtedly entered the freeway at or upstream of Broadway instead of entering at their usual ramps farther downstream because (a) the freeway trip was more attractive during the control study because of the reduced freeway congestion and (b) the use of downstream entrance ramps was less attractive because of the ramp controls which produced some delay on entering. Also the overall traffic possibly increased in the year's time between the studies.

The decrease of about 100 vehicles entering at the Detroit entrance ramp does not represent a decrease in freeway traffic but rather is a direct result of the improved freeway level of service. Normally, during the periods with no ramp control, about 100 vehicles between 7 and 8 a. m. exit at the S. H. 225 (northbound) exit and reenter at the Detroit ramp to avoid about 1500 feet of freeway congestion. During the control study, congestion did not develop in this region so the exit-reentry maneuver would not save the motorist any time; hence the decrease in the frequency of this maneuver. The number of vehicles exiting at the S. H. 225 northbound exit ramp decreased by 112 during this same time period, further substantiating the explanation of the decrease in the Detroit Street entrance ramp volume.

The increased volume on the S. H. 225 entrance ramp is explained by the fact that it was not controlled while the nearby entrance ramps which are alternate entrances for the La Porte Freeway traffic were controlled. Thus, the 90-vehicle increase represents diversion from other entrance ramps which were controlled.

All of the entrance ramps from S. H. 35 to Cullen had decreases in 7-8 a. m. volume. All of these ramps were controlled and the volume decreases were caused by the expected delays at the metered ramps or by the closure in the cases of the Griggs and Cullen ramps. At some ramps, such as Dumble, the decrease in volume was greater than the expected delay would seem to warrant, indicating a reluctance on the part of some motorists to undergo control. This could be caused either by rebellion or by reluctance to try something which is unknown, but also undoubtedly means that some good alternate routes on arterial streets were available. Otherwise, the rebellion or reluctance would have given way to the desire to reduce travel time.

The increase in volume at the Scott entrance ramp can be entirely attributed to the closure of the Cullen entrance ramps just upstream. Many of the vehicles which normally enter at Cullen during the closure period proceeded down the frontage road and entered at the Scott ramp.

The total decrease in the volume entering the inbound freeway was 427 vehicles (not correcting for the decrease in the frequency of the exit-reentry maneuver at the S. H. 225 exit and Detroit entrance ramps) which represents a 5. 3 percent decrease in traffic entering the freeway. From Broadway to Griggs (after correcting for the decrease in the frequency of the exit-reentry maneuver) the total entering traffic was virtually identical before and during the controls.

Vehicle-Miles of Travel and Average Speed Between Broadway and Griggs.—Just as the number of vehicle-hours in a system in a given time period represents the total amount of travel time spent by all vehicles in the system during the time period, the number of vehicle-miles accumulated in the same system in the same time period is the total amount of travel which took place. One vehicle mile is accumulated by one vehicle traveling one mile in the system in the time period of interest. The average speed of all vehicles in the system during the time period is the total number of vehicle miles of travel divided by the total number of vehicle-hours (the units are miles per hour).

Since changes in the volumes of most of the freeway entrance and exit ramps occurred during the control study, a change in the total amount of travel (vehicle-miles) was to have been expected; also a change in the total travel time was found. Thus, the average speed probably also changed. Table 5 contains a summary of these statistics for the Broadway-Griggs subsystem before and during the control study. From the table it can be seen that the total amount of travel between Broadway and Griggs from 7 to 8 a. m. increased 11 percent, from 7990 to 8865 vehicle-miles. This at least partly reflects a more efficient use of the system of streets and freeway caused by clearing the congestion on the freeway, thereby encouraging its greater use.

TABLE 5

VEHICLE-MILES, VEHICLE-HOURS AND AVERAGE SPEED OF BROADWAY-GRIGGS SUBSYSTEM, 7-8 A. M.

Category	Before Control January, 1964	During Control February, 1965	Difference
Total travel, vehicle-miles	7990	8865	+875 (+11%)
Total travel time, vehicle-hours	575	297	-278 (-48%)
Average speed, miles/hour	14	30	+16 (+114%)

Meanwhile, the total travel time decreased by 48 percent, from 575 to 297 vehicle-hours. These changes caused the average speed between Broadway and Griggs to increase from 14 to 30 mph between 7 and 8 a. m.

Individual Vehicle Travel Time.—Data on the travel time required for an individual vehicle to travel from various points on the freeway to the end of the freeway near Dowling Street were obtained in January 1964, before the controls, and again from February 1-5, 1965, during the control study. These data are presented in the form of average travel time contours (Figs. 11 and 12). Figure 11 is the average travel time contour map for January 1964, and Figure 12 represents conditions in February 1965, during the control study.

Figure 13 is a contour map of the average savings in travel time for a vehicle traveling from a certain point on the freeway at a certain time to the end of the freeway. A maximum of about 8 minutes was saved on the average by vehicles traveling from the Reveille Interchange area to the end of the freeway at about 7:30 a. m. Although not plotted, it was found that the maximum travel time between Broadway and Dowling

showed that before any controls were initiated the flow rates on the freeway at Griggs Road decreased as the peak period progressed. This decrease in flow was caused largely by the congestion from downstream backing over the Griggs Road overpass and was no doubt caused partly by the congestion forming upstream of this overpass. One objective of the controls initiated early in 1965 was to increase the flow rates over the Griggs Road overpass by (a) reducing downstream congestion by ramp controls at the Griggs, Wayside and Telephone entrance ramps and (b) reducing the congestion immediately upstream of the Griggs overpass primarily by strict control at the Mossrose entrance ramp.

Figure 14 shows the results of these attempts. The average flow rate over the Griggs Road overpass during the control study remained close to 450 vehicles per 5 minutes until about 7:25-7:30 a. m. The Griggs Road entrance ramp (down-stream) was reopened at 7:20 a. m. and caused a large part of the volume decrease after this time during the control study. Before the controls the 5-minute volumes dropped much sooner and to much lower values than they did during the control study. Thus, the controls did succeed in increasing the flow rate over the Griggs overpass. This was accomplished, not by raising the maximum flow rate (capacity), but by sustaining the maximum flow rate for a longer period of time, i. e., by preventing the large decrease in flow due to downstream congestion. The total inbound freeway volume at the Griggs overpass from 7:00-7:30 a. m. was increased from 2419 to 2699 vehicles during the control study, while the 7:30-8:00 a. m. volume decreased from 2451 to 2326. The total volume from 7-8 a. m. was increased by 155 vehicles.

Arterial Streets

The street system covered during the control study is shown in Figure 15. This section was selected for study because the traffic diversion expected from the Reveille area and the Griggs Road entrance ramp would have to move through the critical

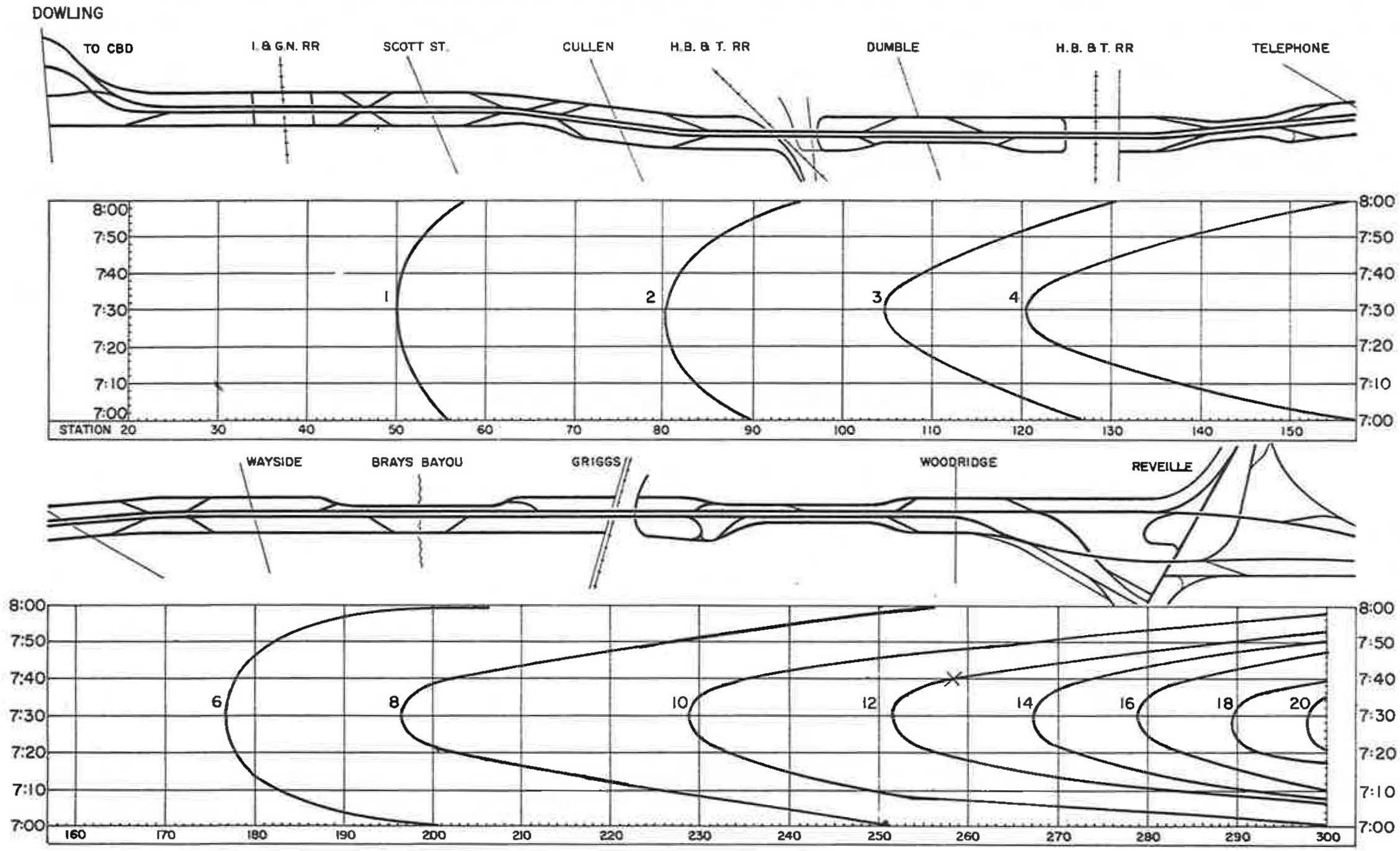


Figure 11. Average travel time (minutes) to end of freeway before control—January 28-30, 1964. Explanatory Note: Point X indicates that a vehicle traveling on the inbound Gulf Freeway at Woodridge at 7:42 a.m. would spend an average of 12 minutes on a trip to Dowling Street.

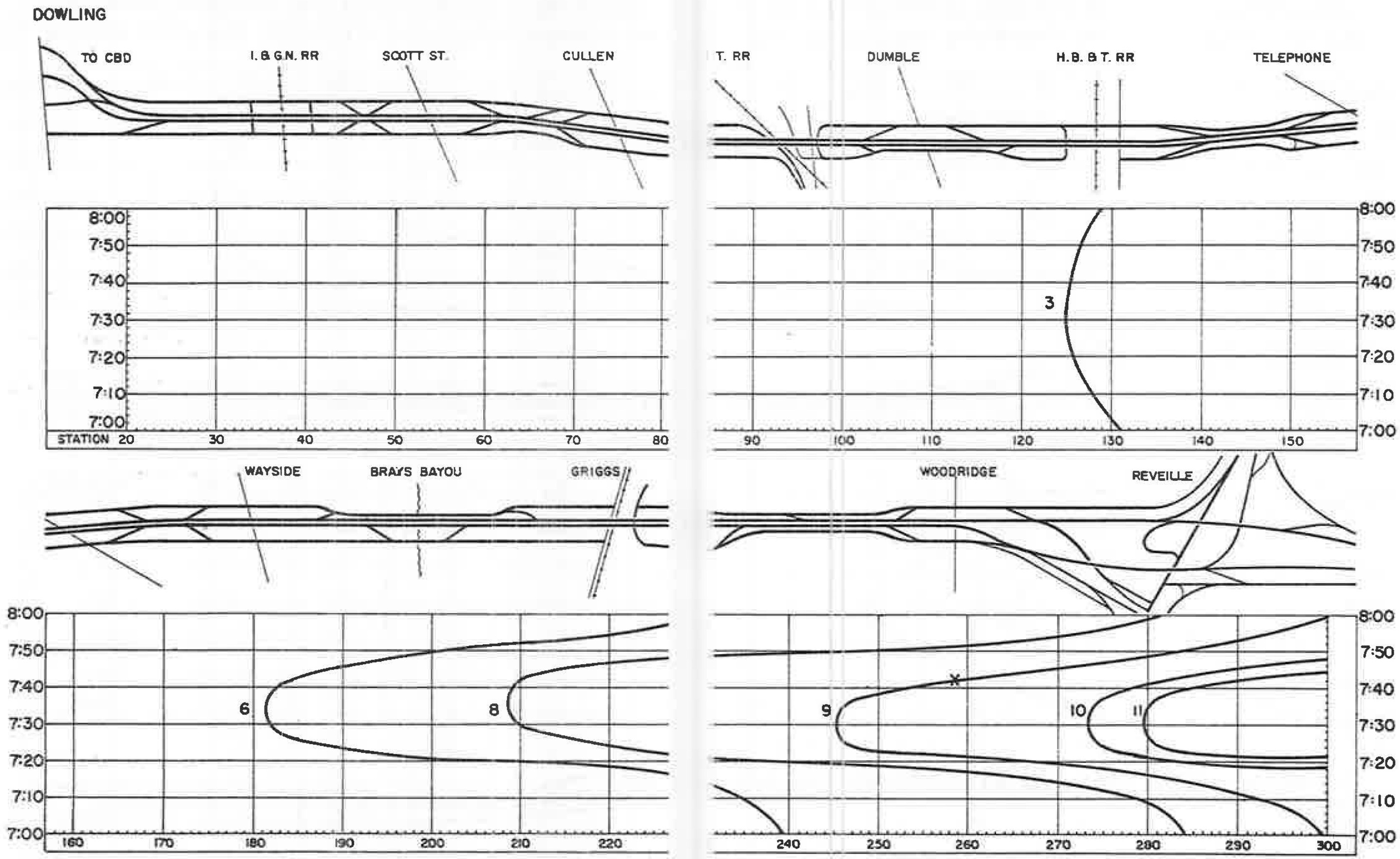


Figure 12. Average travel time (minutes) to end of freeway during con —February 2-4, 1965. Explcnatory note: Point X indicates that a vehicle traveling on the inbound Gulf Freeway at Woodridge at 7:42 c would spend an average of 9 minutes on a trip to Dowling Street.

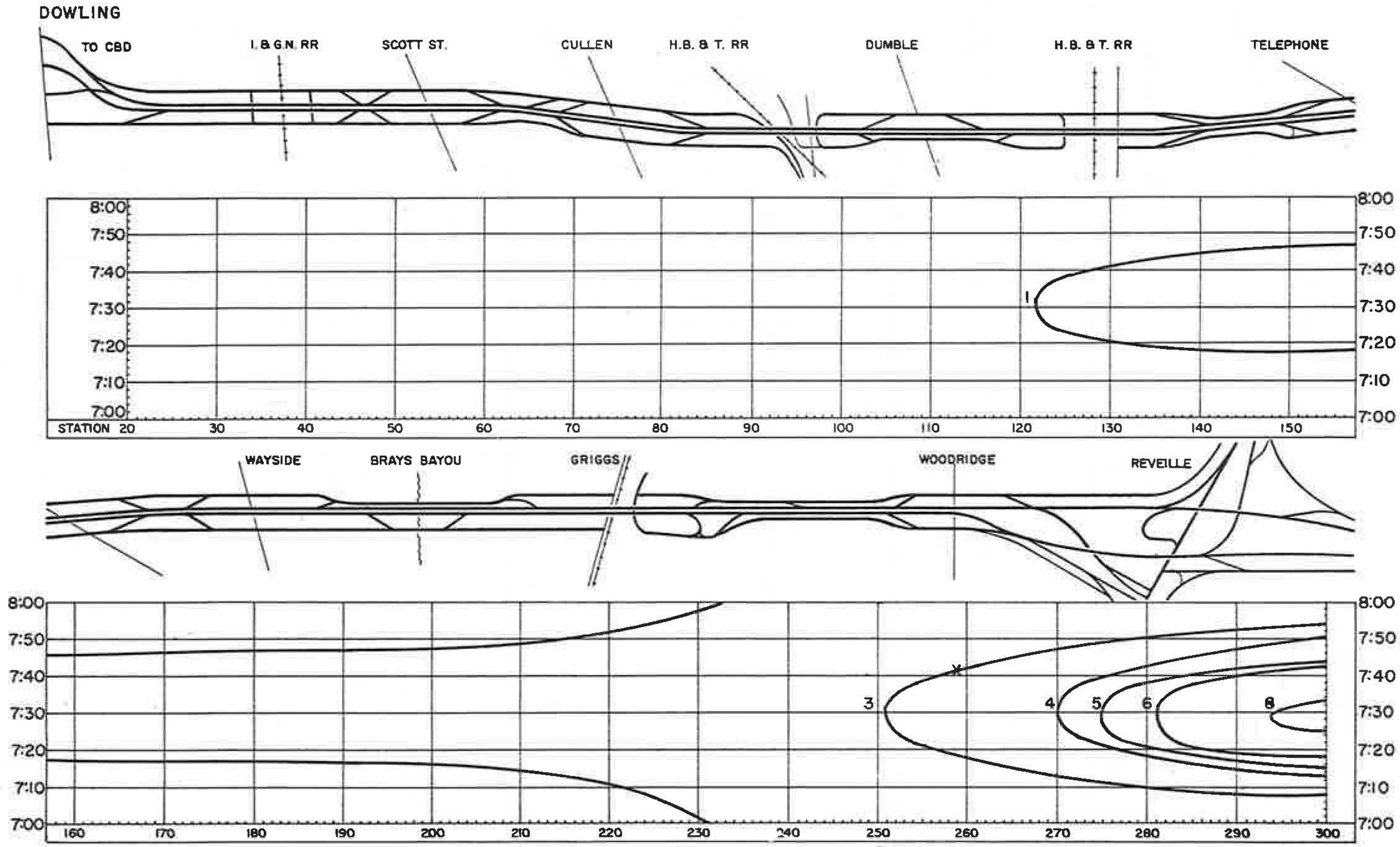


Figure 13. Saving in travel time (minutes) to end of freeway. Explanatory note: Point X indicates that a vehicle traveling on the inbound Gulf Freeway at Woodridge at 7:42 a.m. saved an average of 3 minutes during the control study on a trip to Dowling Street.

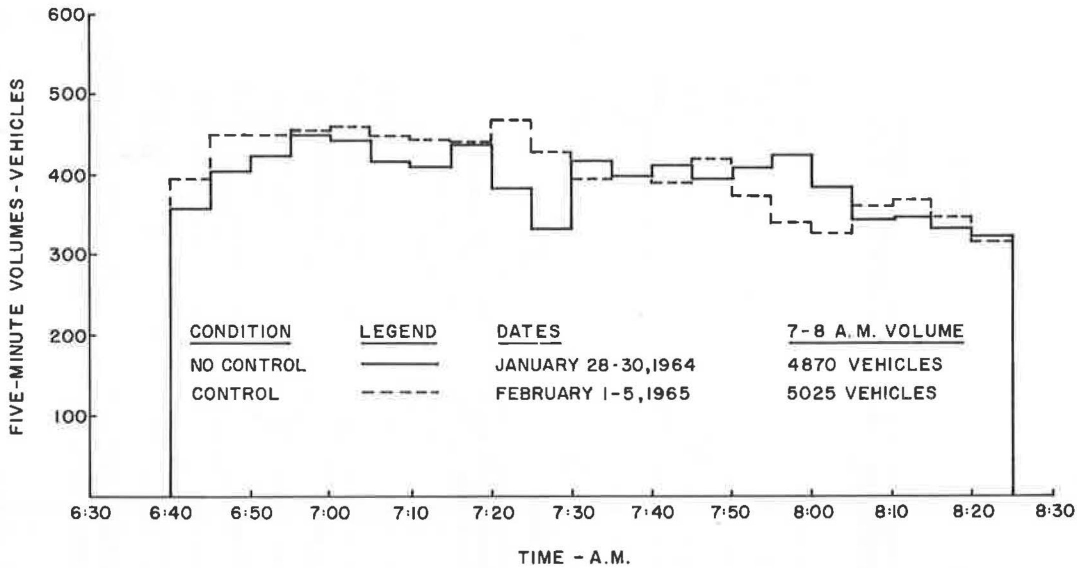


Figure 14. Five-minute volumes on Gulf Freeway at Griggs Road.

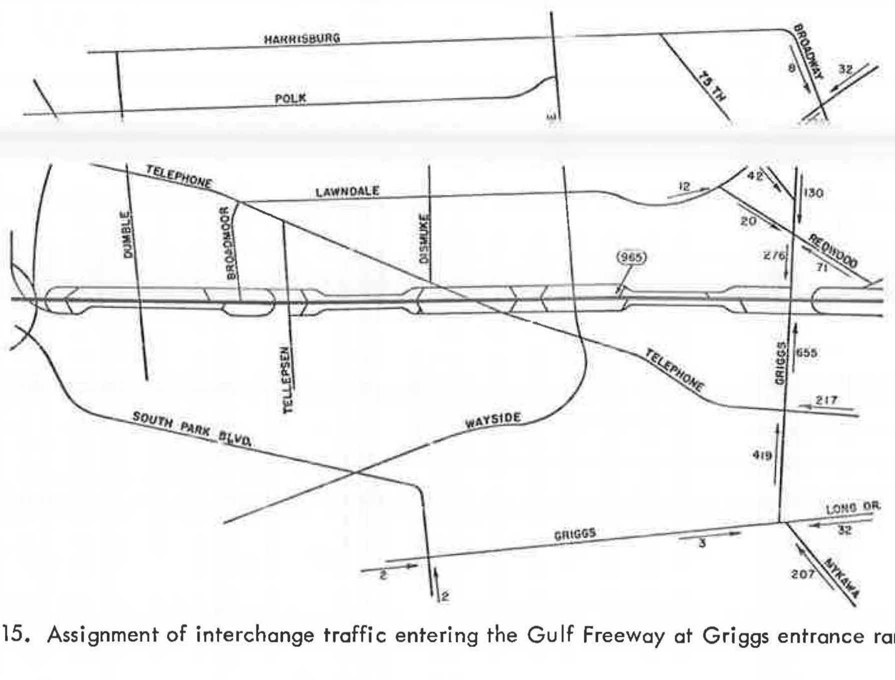


Figure 15. Assignment of interchange traffic entering the Gulf Freeway at Griggs entrance ramp.

intersections identified on the maps. Traffic diversion from other ramp locations would be accommodated on the frontage roads. Also indicated on the map are the approach volumes for the Griggs Road ramp which was closed for a 15-minute period.

Travel Time Runs.—Travel time runs were taken on the alternate routes before, during and after the control study to determine if the shift of traffic from the freeway created any problems on the street system. Only three or four runs were made during each time period for each alternate route. The average of these runs, which are sum-

TABLE 6

SUMMARY OF TRAVEL TIME RUNS ON ARTERIAL STREETS

1. Holmes Road to Gulf Freeway—Via Telephone Road

	7:00-7:10	7:10-7:20	7:20-7:30
Before	7 min 0 sec	8 min 0 sec	9 min 0 sec
During	6 min 30 sec	9 min 15 sec	11 min 0 sec
After	6 min 30 sec	7 min 0 sec	9 min 0 sec

2. Mykawa-Griggs to Cullen Blvd.—Via South Park

	6:45-7:00	7:00-7:15	7:15-7:30
Before	7 min 15 sec	11 min 0 sec	15 min 0 sec
During	7 min 0 sec	8 min 0 sec	11 min 0 sec
After	6 min 0 sec	6 min 30 sec	10 min 0 sec

3. Griggs Road to Broadmore—Via Lawndale

	6:45-7:70	7:00-7:15	7:15-7:30
Before	5 min 30 sec	6 min 0 sec	6 min 0 sec
During	5 min 45 sec	6 min 0 sec	7 min 0 sec
After	6 min 0 sec	6 min 15 sec	6 min 15 sec

marized in Table 6, shows no significant difference in travel times, except in the times recorded on Mykawa-Griggs alternate route. The decreases in travel times are due to the small sample and the traffic signals that are not interconnected, and not to the effect of ramp controls. No large increases in travel times were recorded at any time during the study period on the city streets, indicating that the increased travel did not affect traffic operations.

Changes in Volume.—The shifting volume pattern on the street system is illustrated in Figure 16. There was a substantial shift in traffic the first week of the study, January 26-29, after which the volumes in most cases dropped back to the normal pattern. At the high-volume intersection, the increase in volume came during the time 7:00-7:30 a. m. when the normal demand on the intersection is low. This accounts for the low travel time runs even though the volumes at the intersection increased.

Traffic Delay.—Five major intersections on the alternate routes that operate at or near capacity were studied. The effect of diverting traffic through the intersections was noted by the time congestion began, the length of time the intersection was congested (see Fig. 1) and the total travel time for the traffic passing through the intersection. The results in Table 7 indicate congestion started earlier, but lasted about the same length of time as without control. The travel time was slightly increased, but this was primarily due to the increase in volume, rather than an increase in delay.

LONG-TERM EFFECTS OF THE CONTROLS

The control study was originally scheduled for a two-week period but was later extended for an additional five-week period. For the original two weeks a field crew of up to 20 men was available for the closed-system, input-output studies and for other counts on the freeway and arterial streets. The field crew consisted of hourly workers who are students from Texas A&M University and the University of Houston and some supervisory personnel from Texas A&M. After the initial two-week period the size of the field crew was reduced to eight because of costs and scheduling difficulties.

After February 5, the field crew was used for freeway and arterial street counts. The aerial photographs were used to obtain the total travel time on the inbound freeway and frontage roads. Moving vehicle studies were conducted to obtain trip time for individual vehicles.

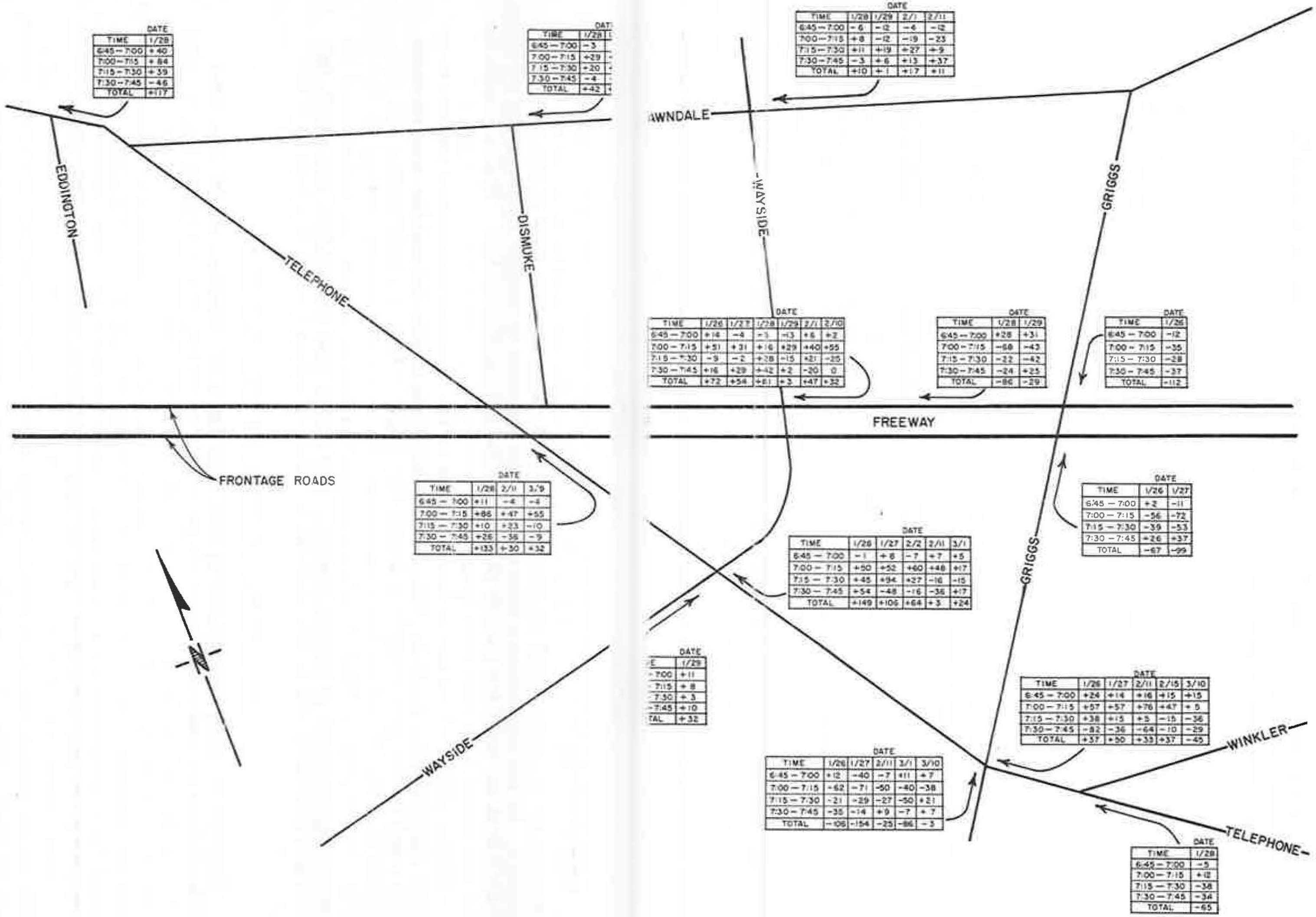


Figure 16. Change in traffic volumes during freeway cc l (volumes compared with 3-day average before control).

TABLE 7

EFFECTS OF CONTROL ON MAJOR INTERSECTION APPROACHES

Intersection	Start of Congestion (a. m.)	Length of Congestion (min)	Total Travel Time (veh-min)
South Telephone Approach to Griggs:			
Before Control	7:12	50	2123
During Control	7:09	47	2265
South Telephone Approach to Wayside:			
Before Control	7:23	37	1800
During Control	7:11	47	2805
South Telephone Approach to Gulf:			
Before Control	7:18	46	1030
During Control	7:10	45	1627
West Griggs Approach to Telephone:			
Before Control	7:18	25	380
During Control	7:24	17	262
East Lawndale Approach to Wayside:			
Before Control	7:17	48	1668
During Control	7:14	49	1409

TABLE 8

AVERAGE 7-8 A. M. VOLUMES ON ENTRANCES TO THE FREEWAY

Entrance to Freeway System	Before Control January, 1964	During Control Feb. 1-5, 1965	During Control Feb. 8 - Mar. 12, 1965
Freeway near Broadway	2831	3185	3274
Detroit on ramp	218	122	no data
S. H. 225 on ramp	559	649	707
S. H. 35 on ramp[a]	818	726	750
Woodridge on ramp[a]	426	398	361
Mossrose on ramp[a]	643	318	412
Griggs on ramp[b]	683	496	493
Wayside on ramp[a]	335	332	315
Telephone on ramp[a]	413	356	341
Dumble on ramp[a]	345	294	318
Cullen on ramps (2)[b]	574	348	296
Scott on ramp	63	257	no data
Total	7908	7481	7656[c]

[a] Ramps which were metered.
[b] Ramps which were closed.
[c] Assumes Detroit entrance ramp volume = 122 and Scott entrance ramp volume = 257.

Freeway and Frontage Roads

Total System Travel Time, 7-8 a. m.—During the remainder of the control study (February 8-March 12, 1965) aerial photographic data were obtained for the purpose of determining the total 7-8 a. m. travel time in the freeway and frontage road system. However, bad weather forced the cancellation of flights on several days and on each

of the days in which aerial photographs were taken the data proved unusable because of freeway accidents or similar traffic disturbance. Since a field crew of sufficient size for the closed system input-output counts was not available, these studies were not made. Hence, no data for total freeway system travel time are available.

Freeway and Ramp Volumes, 7-8 a. m.—The average 7-8 a. m. volumes on the freeway entrance ramps and the inbound freeway at Broadway during three time periods are shown in Table 8. The average freeway volume during the February 8 to March 12 control period increased by about 90 vehicles over the February 1-5 control period. This probably reflects more motorists finding that the freeway operation had been improved and thereby being attracted to enter the freeway at a point farther upstream than normal.

An increase in volume of about 70 vehicles was observed at the S. H. 225 entrance ramp, again reflecting the improved freeway operation. A 35-vehicle decrease in the volume at Woodridge was found. These vehicles probably normally bypassed the S. H. 225 entrance ramp to enter at Woodridge to reduce the amount of congested freeway driving. However, when the effects of the controls became well known, some of these vehicles probably began entering at S. H. 225 since the freeway congestion was greatly reduced.

The volume at the Mossrose entrance ramp increased by about 100 vehicles in the latter control period. At least two factors may have contributed to this. The first is that the initial diversion from this ramp may have been greater than the delay would have warranted. The notices distributed before the start of the control study warned of very large delays at this ramp. During the first two weeks of the study, some vehicles did find large delays at this ramp but during some portions of the control period there was little or no delay at the ramp. Hence, during the latter control period some vehicles probably tried this ramp and found the delay tolerable. The second factor contributing to the increased ramp volume is the increase in the metering rate by the policeman in charge of metering this ramp. Instead of adhering to the predetermined metering rates, the officer used his judgment as to the proper rates. This usually resulted in an increase in the metering rate and reduced delay, resulting

resulted from this change in metering rates.

A slight increase in volume was observed at the S. H. 35 entrance ramp. The volume at the Griggs ramp was virtually unchanged from the first control period to the second.

Small decreases in volume were observed at the Wayside and Telephone ramps and a slight increase was found at the Dumble ramp indicating a slight change in the travel patterns in this area. At the Cullen ramps a decrease of about 50 in the 7-8 a. m. volume was found. One possible explanation is that some drivers found that use of the frontage road instead of the freeway between Cullen and Dowling Streets resulted in little increase in travel time.

No data were collected at the Detroit and Scott entrance ramps since anticipated volume changes at these ramps were slight.

Individual Vehicle Travel Time.—A contour map of travel time on the freeway to Dowling Street is shown in Figure 17. The contours are quite similar to those obtained during the February 1-5, 1965, control period. Thus, the data obtained during the February 1-5 period probably represented steady-state or equilibrium conditions.

Five-Minute Volumes on the Freeway at Griggs Road.—Figure 18 shows the 5-minute volumes on the freeway at the Griggs Road overpass. The solid line represents the data from January 1964, and the dashed line represents the data obtained during the last five weeks of the control study.

It can be seen that the 5-minute volumes during the control study are consistently higher than the corresponding volumes obtained in the "before" study up to about 7:30 a. m. and are approximately the same after 7:30 a. m. This means that during the control study more vehicles were able to get out of the congested Broadway-Griggs subsystem during the early period, leaving less storage to be cleared in the latter period.

The average 7:00-7:30 a. m. volume was 2703 vehicles during this period compared to 2414 vehicles before the control study. This represents a 12 percent increase in volume during this time period.

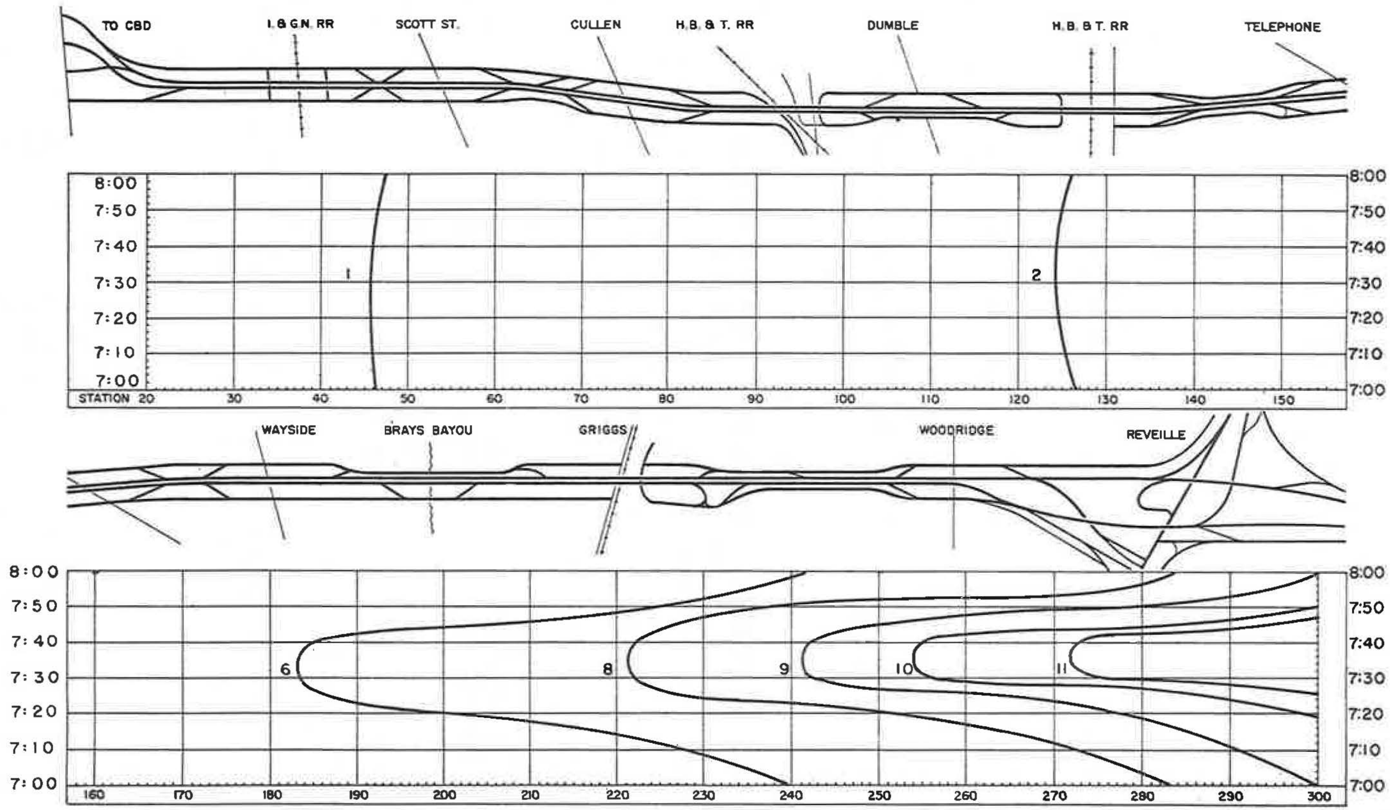


Figure 17. Average travel time (minutes) to end of freeway during control—February 8-March 12, 1965.

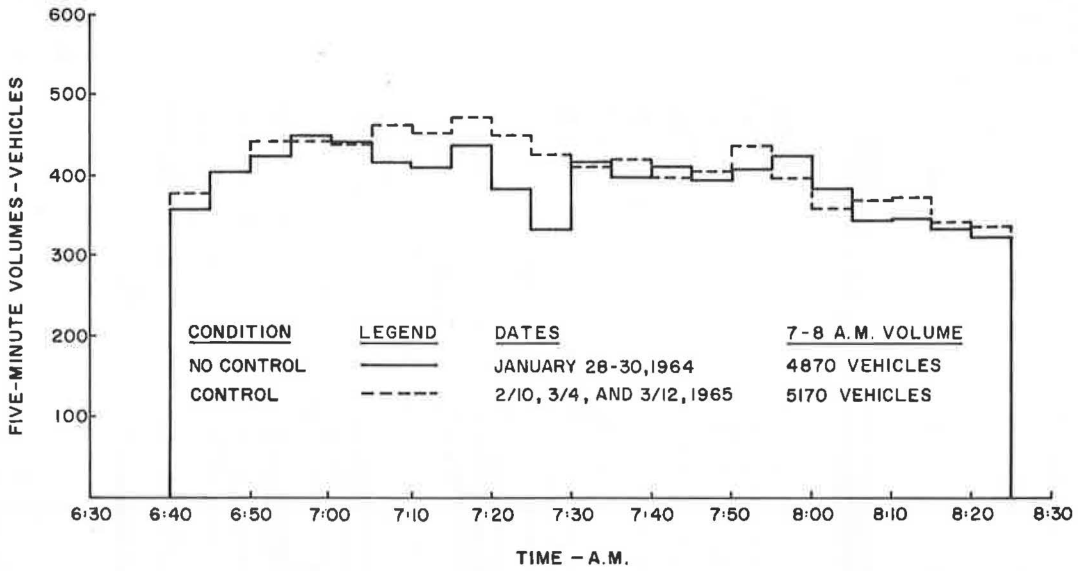


Figure 18. Five-minute volumes on Gulf Freeway at Griggs Road.

TABLE 9

TOTAL SYSTEM TRAVEL TIME BEFORE, DURING AND AFTER THE CONTROL STUDY

System or Subsystem	Before Control	During Control	After Control
Inbound freeway:			
Broadway to Griggs	575	297	578
Griggs to S. HB&T RR	367	310	386
S. HB&T RR to Dowling	302	266[a]	260
Total inbound freeway	1244	873	1224
Inbound frontage road	190[a]	201[a]	214
Total inbound freeway and frontage road	1434	1074	1438

[a]Based on incomplete data caused by inadequate light conditions which made aerial photography during the early part of the 7-8 a.m. period impossible.

Arterial Streets

The results of the freeway controls on arterial streets were not divided into immediate and long-term effects. However, the shift in traffic volume during the control period can be seen in Figure 16. After the first two days of control, the traffic pattern on the arterial streets did not appear to differ significantly from the conditions that prevailed before the controls were initiated.

OPERATION AFTER TERMINATION OF THE CONTROLS

The last day that the controls were in effect was March 12, 1965. Hence on Monday, March 15, the freeway returned to an uncontrolled state. This section contains the results of studies during the period of time from March 15 to April 30, 1965.

Freeway and Frontage Roads

Total System Travel Time, 7-8 a. m.—Several attempts were made to take aerial photographs for the purpose of determining the total system travel time from 7-8 a. m. after the controls had been removed. Adverse weather or unusual traffic conditions (due to accidents or other reduced-capacity situations) reduced the number of good days of data to one—April 28, 1965. This was approximately a month and a half after termination of the control study.

Table 9 shows the total 7-8 a. m. travel time on the inbound freeway and frontage roads on April 28, 1965, as well as comparable figures before the control study and during the second week of the control study. The data in this table indicate that, as far as the total travel time is concerned, the freeway conditions after the termination of the controls returned approximately to the conditions which existed before the beginning of the control study. However, it should be borne in mind that only one day's data were used in the "after" analysis.

Freeway and Ramp Volumes, 7-8 a. m.—Several counts were made at entrances to the freeway during the period March 15-April 30, 1965, after the termination of the controls. Table 10 contains the average 7-8 a. m. volumes at the entrances to the freeway before, during and after the control study.

It would seem that the traffic pattern would return to that which prevailed before the controls were initiated. Indeed, the total volume entering the freeway returned almost to the level of January 1964. Also the S. H. 225, Telephone and Scott on ramp volumes returned to the volumes obtained before the control study. However, the volumes at other locations did not return to the volume level of January 1964.

TABLE 10

AVERAGE 7-8 A. M. VOLUMES ON ENTRANCES TO THE FREEWAY BEFORE, DURING AND AFTER THE CONTROLS

Entrance to Freeway System	Before Control	First 2 Weeks	Last 5 Weeks	After Control
Freeway at Broadway	2831	3185	3274	3306
Detroit on ramp	218	122	—	—
S. H. 225 on ramp	559	649	707	549
S. H. 35 on ramp[a]	818	726	750	777
Woodridge on ramp[a]	426	398	361	343
Mossrose on ramp[a]	643	318	412	558
Griggs on ramp[b]	683	496	493	637
Wayside on ramp[a]	335	332	315	257
Telephone on ramp[a]	413	356	341	401
Dumble on ramp[a]	345	294	318	317
Cullen on ramps (2)[b]	574	348	296	509
Scott on ramp	63	257	—	71
Total	7908	7481	7646[c]	7847[d]

[a] Ramps which were metered.
[b] Ramps which were closed.
[c] Assumes Detroit entrance ramp volume = 122 and Scott entrance ramp volume = 257.
[d] Assumes Detroit entrance ramp volume = 122.

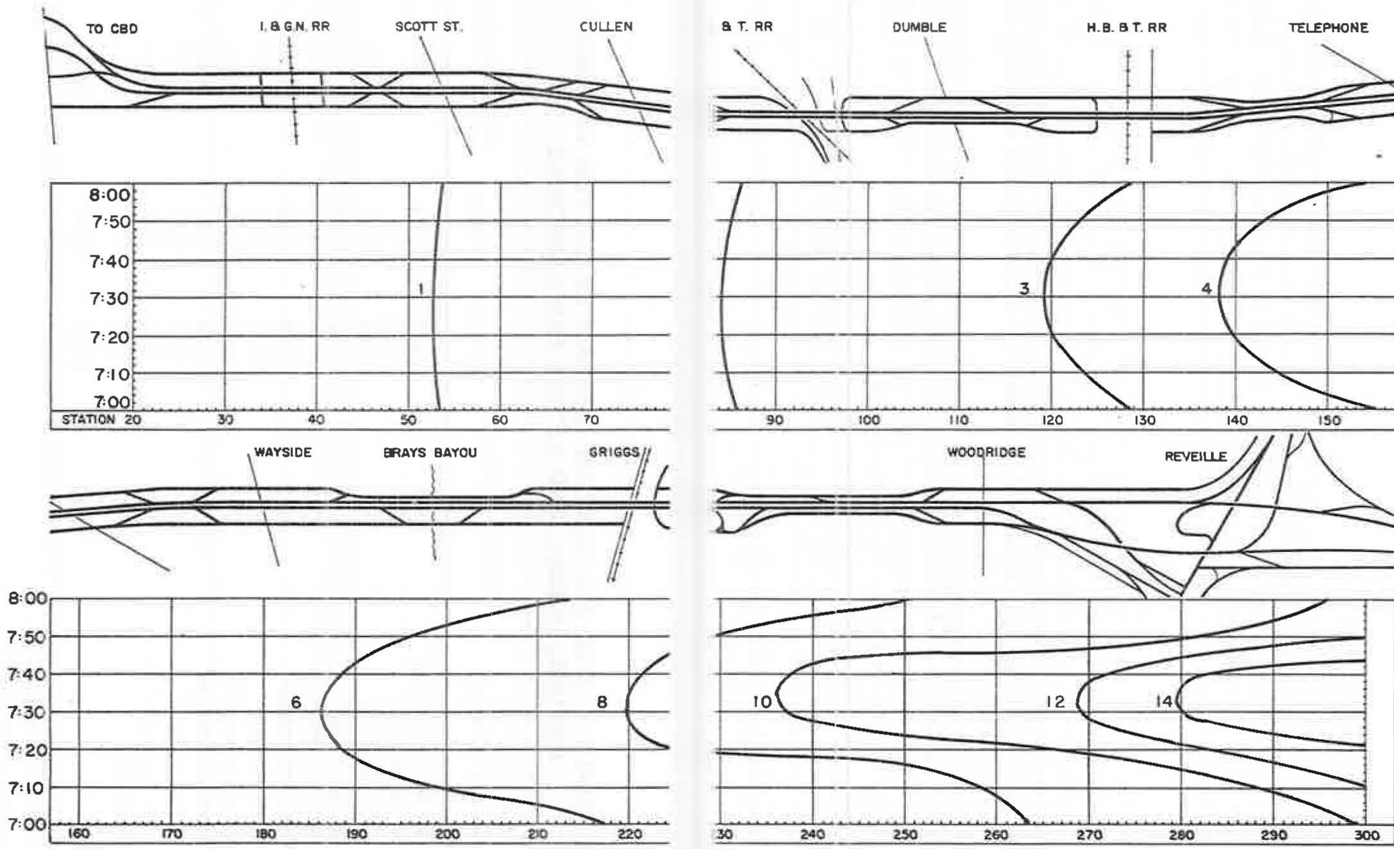


Figure 19. Average travel time (minutes) to ɛ ɔf freeway after controls—March 15-April 30, 1965.

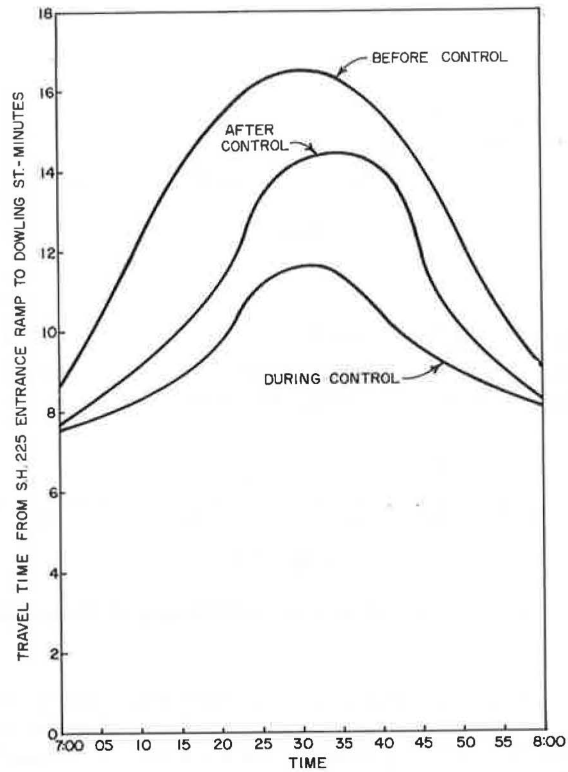


Figure 20. Average travel time from S. H. 225 entrance ramp to Dowling St. before, during and after control study.

The 7-8 a. m. volume on the freeway at Broadway remained about 475 vehicles greater than the volumes obtained before the controls. There was a decrease in volume on each of the S. H. 35, Woodridge and Mossrose entrance ramps with a total decrease of about 200 vehicles after the controls relative to the volumes in January of 1964. Thus, a net increase in volume of about 275 vehicles was found between Broadway and Griggs Road.

The volumes on the Griggs, Wayside, Dumble and Cullen ramps were also considerably lower (a total of about 215 vehicles) after the controls terminated than they had been before the start of controls. The volume on the Dumble entrance ramp remained virtually unchanged from the last 5 weeks of the control study after the termination of the controls. This seems to suggest that many motorists who normally used the Dumble entrance ramps and who diverted during the control study found alternate routes which they liked as well as their old freeway routes.

Individual Vehicle Travel Time.—The contour map of average travel time to the end of the freeway after the termination of the controls is shown in Figure 19. It can be seen that these conditions were somewhere between those before the controls and those during the control study. In other words, conditions were better after the termination of the controls than they were before the control study but worse than they were during the control study. Figure 20, which is a plot of the average travel time from the S. H. 225 entrance ramp to Dowling Street before, during and after the controls, clearly shows this. The line of average travel time from the S. H. 225 entrance ramp vs time after the controls falls between similar lines obtained from data taken before the controls and during the controls.

Five-Minute Volumes on the Freeway at Griggs Road.—Figure 21 shows the 5-minute volumes on the freeway at Griggs Road before the controls were initiated and after

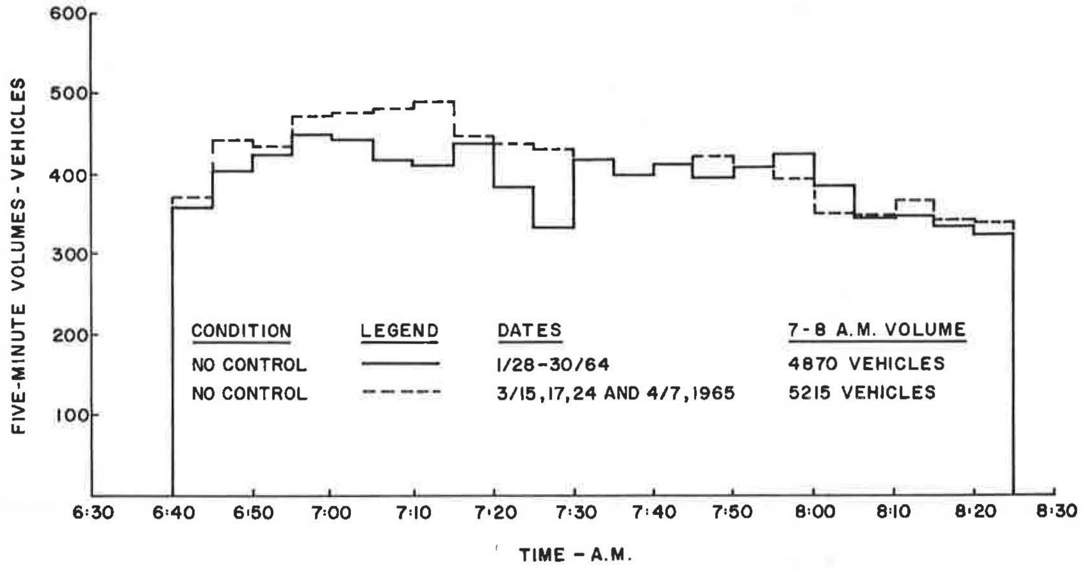


Figure 21. Five-minute volumes on Gulf Freeway at Griggs Road.

they were terminated. As can be seen, a substantial change occurred between the two time periods. Until 7:30 a. m. the volumes after the controls were terminated were higher than those of January 1964, and after 7:30 a. m. they were both about the same. The total 7-8 a. m. volume was about 350 vehicles greater in the latter period. The increased volume (after the controls ended) up to about 7:30 seems to come from two

suggesting a higher capacity (probably due to the better light conditions in the latter period). The second is the decreased severity of the volume decrease caused by downstream congestion.

Arterial Streets

Several counts were made on the arterial streets during the period March 15 to April 15, 1965, after the termination of the controls. Figure 22 shows the change in volume from 6:45 to 7:45 a. m. when compared to volumes before control.

It would seem that the total hourly volumes have in most cases returned to the level before control. There is still shifting of traffic away from the Griggs entrance ramp approaches during the 7:00 to 7:15 time period when the ramp was closed.

There was no significant change in travel times on the alternate routes after the controls were removed (Table 6).

PUBLIC OPINION

The objective of the numerous traffic counts, travel time runs, and aerial surveys was to determine the effect of the freeway control plan on the peak period traffic. The changes that are brought about in travel time or the length and severity of congestion may be readily evident to the researcher or the traffic engineer, but not to the individual motorist. Since public acceptance of a control system will be so vital to its successful operation, special attention was given to a study of this aspect. It was desired to obtain as much data as possible to evaluate the reaction of individual motorists to the operation of the controls.

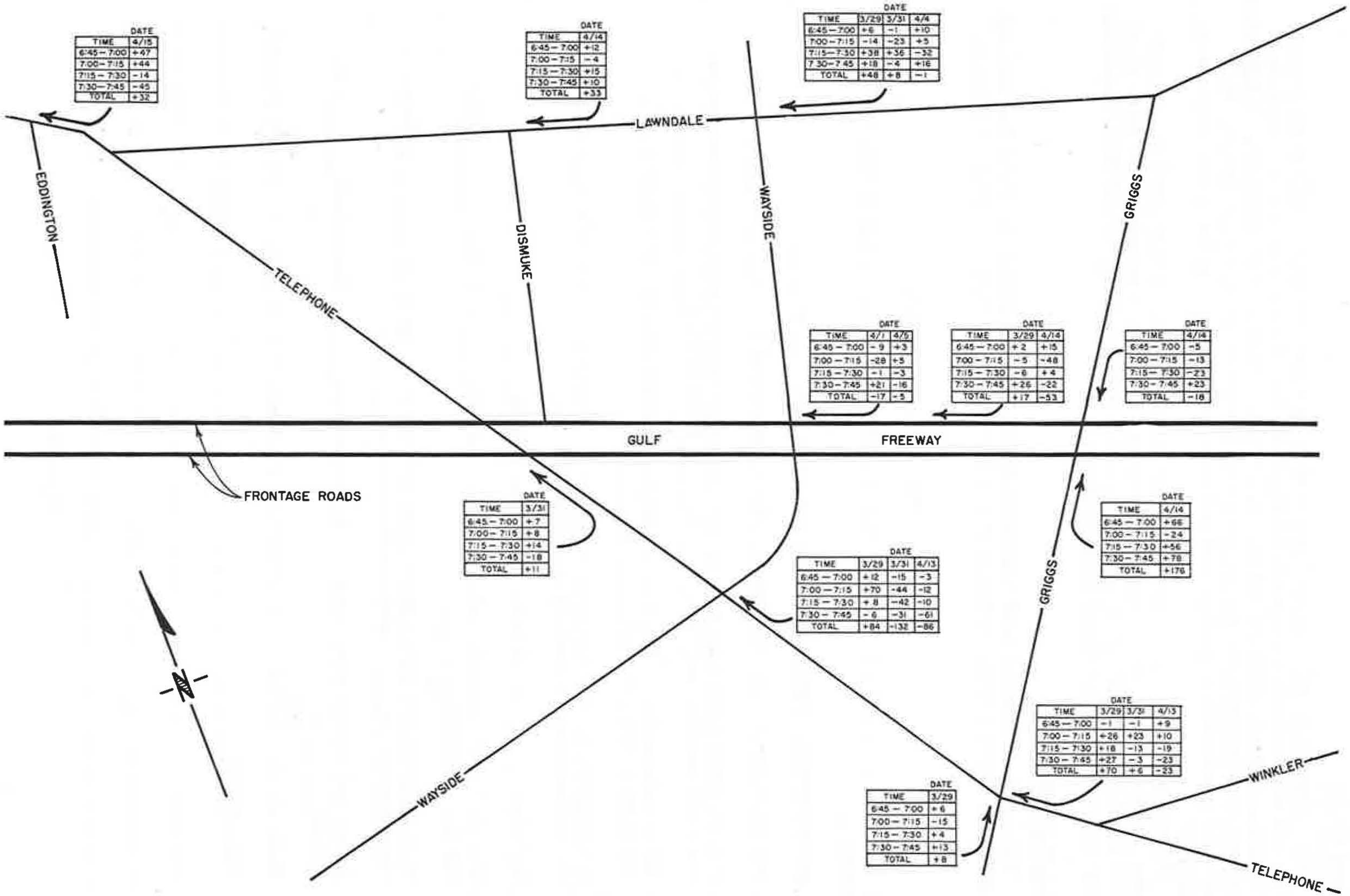


Figure 22. Change in traffic volumes after freeway control (volumes compared with 3-day average before control).

News Media

The newspapers and radio-television stations were given the details of the control plan in a news release on January 19 (Appendix A). No special effort was made to encourage support of the plan by the news media. Routine news articles were published and reported on radio and television one day before the control study began. No editorial comments or special news features were observed in the news media during the study.

Public opinion as expressed through the news media was represented by one comment, contained in a news article. A Houston City Councilman stated that the control plan was denying Houston taxpayers access to a public facility. No official complaints were received from the Councilman's office.

Questionnaire Study

The experience of the first control study conducted in August 1964 was that although significant improvements in traffic characteristics were made, the public did not ex-

FREEWAY CONTROL QUESTIONNAIRE

All questions below pertain to travel from 6:45 - 8:00 a.m. during the period, January 26 to February 5.

1. Did the freeway ramp controls affect your trip? Yes_____ No_____

2. Did you avoid using the Gulf Freeway due to the controls? Yes_____ No_____

3. If you traveled on the Gulf Freeway, did you use a different entrance ramp than the one you would normally use? Yes_____ No_____ Name of Ramp_____

the freeway controls? Yes_____ No_____

5. If you entered the freeway, please indicate your impression of the delay encountered at the entrance ramp by checking one of the following:

   No Delay_____ Slight Delay_____ Long Delay_____

6. Was the overall travel time of your peak trip reduced_____, increased_____, about the same as normal_____.

7. Do you feel that the control plan greatly improved traffic operation_____, produced no noticeable change_____, made conditions worse_____.

8. Do you feel that the control plan should be continued_____, discontinued_____, no opinion_____.

9. Please comment on the control plan giving any suggestions or criticisms you feel pertinent.

   COMMENTS:_____

Figure 23.

press its opinion, one way or another. Except for a few phone calls to the Highway Department or City Hall, no criticism or praise of the control plan was received. Therefore, a questionnaire survey was conducted as part of the control plan of January 1965 in an attempt to determine the consensus of the motoring public involved in the study.

The survey was made of those persons who enter the freeway by one of the inbound entrance ramps from S. H. 225 to Dumble. It did not include traffic that entered the study area via the freeway lanes from upstream of the Reveille Interchange area.

Addresses of many of the motorists who entered the freeway by the ramps in the control area were obtained from origin-destination surveys (8) conducted during the past 18 months. A questionnaire (Fig. 23) with an attached letter of explanation was mailed to these addresses on February 3, two days before the end of the first two-week period. The forms were to be completed and returned by mail.

The distribution by mail had certain disadvantages, such as (a) occupants had moved or changed their trip, (b) addresses were incomplete or (c) addresses were copied down wrong. However, distribution at the ramps would have disrupted traffic while other field studies were being made to determine the effectiveness of freeway control.

Results of Questionnaire Survey

The results of the questionnaire survey are summarized in Tables 11 and 12. Because of the different volumes on the ramps and the different percent returns, the number of motorists responding to the questions are not easily compared. Table 12 presents the data in terms of the percent of total returned by each ramp.

As each question is summarized by entrance ramps, it is important to consider the 3000 vehicles that enter the study area upstream of S. H. 225. These motorists receive the maximum benefit of any control system that improves the flow on the freeway lanes, and suffer none of the disadvantages such as added delay at entry or diversion to other ramps. Yet it was impossible to contact these motorists since they had never participated in an origin-destination survey.

Percent Return.—The number of forms distributed and returned is indicated by ramp at the top of Tables 11 and 12. The total return of 28. 4 percent is considered good for a mailed survey. The percent return by individual ramps was close to the average with the exception of three ramps:

1. S. H. 225—The 16. 5 percent return could be attributed to the fact that this ramp was not controlled.
2. S. H. 35—The high return from this ramp, 38. 6 percent, can be attributed to two things. The metering control changed the operation of this ramp from a two-lane type to a one-lane type and thereby improved merging conditions. The motorists are very interested in methods for improving overall travel conditions. A very high return was received from this ramp for the origin-destination survey conducted in 1963.
3. Wayside—The low return of 18. 4 percent can be attributed to the fact that this ramp had the smallest number of forms distributed. A larger percent of forms was sent to motorists who, during the origin-destination survey, used the ramp during a time not affected by the controls.

Returns on Each of Eight Questions.—

Question 1—Did the freeway ramp control affect your trip?

The forms were distributed to persons who had used the ramp during the peak hour. Some of these persons had changed their trip or had entered the freeway when the controls were not in effect. Those returns that answered Yes (527 of 771) to question 1 were analyzed separately with the following results:

		Percent of Return
Diversion	Used City Streets	9. 5
	Used Different Ramp	18. 5
	Changed Time of Trip	20. 0
Delay at Ramp	No Delay	42. 0
	Slight Delay	43. 0
	Long Delay	6. 0
Travel Time	Reduced	63. 0
	Increased	18. 0
	Same	17. 0
Traffic Conditions	Improved	76. 0
	Made Worse	10. 0
	No Change	11. 0
Study Plan	Continue	76. 0
	Discontinue	15. 0
	No Opinion	9. 0

Question 2—Did you avoid using the Gulf Freeway due to the controls?
Traffic conditions on the city street system must be considered in the design of freeway control. The survey indicated that 7. 8 percent of the traffic diverted to the arterial streets. Volume counts made several days after the O & D survey was mailed indicated that this percentage decreased as the control study continued.

Question 3—Did you use a different entrance ramp during the control study?
Where continuous frontage roads and alternate routes on the arterial street system are available, the traffic moves from one ramp to another, depending on the condition [illegible] trol plan.

Question 4—Did you change the starting time of your trip?
To spread the traffic demand at the ramps, it was suggested that the motorists could avoid unusual delays at the ramps if the time of arrival was changed to miss the control period. The survey indicated that 16. 2 percent of the traffic changed the time of trip.

Question 5—Indicate your impression of the delay encountered at the entrance ramp.
A distinction between slight and long delays cannot be made in terms of time since the normal delay at one ramp may be 3 minutes and at another ramp, only 1 minute. Those that encountered long delays (4. 6 percent) were in most cases expressing their dissatisfaction with the control plan. For example, one-fourth of this group normally use the Griggs Ramp which was closed. Forty-one percent of the motorists indicated a slight delay at the entrance. This was anticipated in the design of the plan. However, only 7 percent of the 41 percent delayed also noted that the total travel time was increased.

Question 6—What effect did the control system have on the overall travel time of your trip?
The results indicated that 13. 5 percent (average for all controlled ramps) of the traffic entering the freeway by one of the controlled ramps had longer travel times. Half of this group, however, did not want the control study to be discontinued. It should be noted again that traffic entering the study area on the freeway lanes is affected by the control study. If a survey had been made of this traffic, the results should have approximated those from S. H. 35, except there would be no additional delays caused by metering the traffic.

Question 7—How do you feel that the control plan affected traffic operation?

Only 7 percent of the returns indicated that the control system made conditions worse. One-half of this group used the arterial street system during the control; the other half experienced long delays at the ramps and increased travel times. Many persons who indicated no change or increased travel times also noted an improvement in the traffic conditions. Comments received on these returns were of improved merging operations and a smoother, more uniform flow of traffic on the freeway lanes.

Question 8—Do you feel the control plan should be continued?

The returns indicate that 12.7 percent of the motorists were in favor of discontinuing the study. The ramps that benefit the most from the control plan are S. H. 225, S. H. 35, Woodridge and Mossrose. Only 6 percent of this traffic is in favor of discontinuing the controls, as compared to 25 percent of the traffic downstream of Griggs Road. The belief that the ramps in the Reveille area are favored over those downstream of Griggs Road is reason for the opposition to the plan. The closure of Griggs entrance ramp for 15 minutes is the major cause of the opposition.

Questionnaire Conclusions.—Based on the results of the mailed questionnaire, the following conclusions are made concerning the freeway control plan:

1. A majority of the motorists indicated that the controls were effective in reducing travel time. Some additional delay was encountered at the entrance ramps but overall travel time was reduced.
2. The traffic operation on the freeway was improved.
3. The motorists were prepared to accept a freeway control system on a regular basis. Special emphasis should be given to informing motorists on ramps to be closed of the possible alternatives.
4. The returns from ramps S. H. 35, S. H. 225, Woodridge and Mossrose were more favorable toward the control plan.

DISCUSSION

General Control Plan

The control plan as developed and later amended worked very well, as an examination of the total system travel times will reveal. The estimates of demand and capacity were at least fairly accurate and the freeway level of service was greatly improved. The capacities at bottleneck locations were probably overestimated slightly (5 percent or so) for the January-February portion of the control study since the freeway volumes during these months (especially January) are lower than they are during the later (summer) months. This is probably due to the earlier sunrise; during the later months the entire peak period fell during good light conditions. In the January studies darkness prevailed until about 7:15-7:30 a.m., well into the peak period. Except for this minor difficulty, the demand-capacity approach provided a rational means of developing a control plan for a freeway system.

When the demands and capacities are estimated for a control plan, an inherent assumption is that these values will not be changed. However, accidents, adverse weather, etc., can change both the demands and capacities at several locations on the freeway. Because of the fixed-time nature of the control system, it was not flexible enough to handle these unusual situations. Such situations will undoubtedly tax the capabilities of the most sophisticated, traffic-adjusted control system that will be developed. However, the more sophisticated control systems will undoubtedly be much better able to cope with the reduced-capacity occurrences.

The opening time of the Griggs entrance ramp at 7:20 a.m. was probably somewhat early since the ramp vehicles normally precipitated congestion on the freeway there shortly after the ramp was opened. The opening time of 7:20 a.m. was actually a compromise in order to avoid diversion to the street system during its peak period. Perhaps this ramp should have been closed longer or metered for a period after 7:20 a.m.

TABLE 11

SUMMARY OF FREEWAY CONTROL QUESTIONNAIRE SURVEY—NUMBER OF RETURNS

	S.H. 225	S.H. 35	Woodridge	Mossrose	Griggs	Wayside	Telephone	Dumble	Total
NUMBER DISTRIBUTED	355	560	324	442	360	195	251	213	2700
NUMBER RETURNED	60	217	107	110	113	37	76	51	771
PERCENT RETURN	16.5	38.6	33.0	24.7	31.4	18.4	30.3	24.0	28.4
1. Did the Freeway ramp controls affect your trip?									
YES	37	153	75	84	75	20	42	31	517
NO	19	52	28	24	31	17	29	15	215
NO ANSWER	4	12	4	2	7	0	5	5	39
2. Did you avoid using the Gulf Freeway due to the controls?									
YES	1	16	3	3	14	4	7	12	60
NO	56	190	102	104	93	33	62	35	675
NO ANSWER	3	11	2	3	6	0	7	4	36
3. Did you use a different entrance ramp than the one you would normally use?									
YES	7	15	9	23	31	5	14	4	108
NO	48	182	94	80	70	31	52	41	598
NO ANSWER	5	20	4	7	12	1	10	6	65
4. Did you change the starting time of your peak hour trip to avoid the freeway controls?									
YES	4	29	11	14	31	8	14	14	125
NO	51	175	93	94	75	29	54	33	604
NO ANSWER	5	13	3	2	7	0	8	4	42
5. Indicate your impression of the delay encountered at the entrance ramp.									
SLIGHT DELAY	24	86	34	62	46	16	29	20	317
LONG DELAY	2	7	3	5	9	1	5	3	35
NO ANSWER	6	29	5	12	17	5	19	9	102
6. What effect did the freeway control have on the overall travel time?									
REDUCED	39	129	70	67	48	17	25	11	406
INCREASED	5	20	17	10	26	3	11	12	104
NO CHANGE	13	53	17	30	32	14	31	23	213
NO ANSWER	3	15	3	3	7	3	9	5	48
7. What effect do you feel the freeway control had on traffic operation?									
IMPROVED OPERATION	45	158	88	83	59	20	32	21	506
NO CHANGE	10	31	10	21	23	12	28	17	152
MADE CONDITIONS WORSE	2	9	5	2	22	2	5	6	53
NO ANSWER	3	19	4	4	9	3	11	7	60
8. Do you feel that the control plan should be continued?									
YES	48	166	87	82	54	22	29	17	505
NO	2	8	3	14	32	9	18	12	98
NO OPINION	8	21	13	12	16	4	21	17	112
NO ANSWER	2	22	4	2	11	2	8	5	56

TABLE 12

SUMMARY OF FREEWAY CONTROL QUESTIONNAIRE SURVEY—PERCENT OF RETURNS

	S.H. 225	S.H. 35	Woodridge	Mossrose	Griggs	Wayside	Telephone	Dumble	Total
NUMBER DISTRIBUTED	355	560	324	442	360	195	251	213	2700
NUMBER RETURNED	60	217	107	110	113	37	76	51	771
PERCENT RETURN	16.5	38.6	33.0	24.7	31.4	18.4	30.3	24.0	28.4
1. Did the Freeway ramp controls affect your trip?									
YES	61.6	70.5	70.1	76.4	66.4	54.1	55.3	60.8	67.1
NO	31.7	23.9	26.2	21.8	27.4	45.9	38.2	29.4	27.8
NO ANSWER	6.7	5.6	3.7	1.8	6.2	0.0	6.5	9.8	5.1
2. Did you avoid using the Gulf Freeway due to the controls?									
YES	1.7	7.4	2.8	2.7	12.4	10.8	9.2	23.5	7.8
NO	93.3	87.6	95.3	94.6	82.1	89.2	81.6	68.6	87.5
NO ANSWER	5.0	5.0	1.9	2.7	5.5	0.0	9.2	7.9	4.7
3. Did you use a different entrance ramp than the one you would normally use?									
YES	11.7	6.9	8.4	20.9	27.4	13.5	18.4	7.9	14.0
NO	80.0	83.9	87.9	72.7	62.0	83.8	68.4	80.4	77.6
NO ANSWER	8.3	9.2	3.7	6.4	10.6	2.7	13.2	11.7	8.4
4. Did you change the starting time of your peak hour trip to avoid the freeway controls?									
YES	6.7	13.4	10.3	12.7	27.4	21.6	18.4	27.4	16.2
NO	85.0	80.6	86.9	85.5	66.4	78.4	71.1	64.7	78.3
NO ANSWER	8.3	6.0	2.8	1.8	6.2	0.0	10.5	7.9	5.5
5. Indicate your impression of the delay encountered at the entrance ramp.									
NO DELAY	46.7	43.8	60.7	28.2	36.3	40.5	30.3	37.3	41.1
SLIGHT DELAY	40.0	39.7	31.8	56.4	40.7	43.2	38.1	39.2	41.1
LONG DELAY	3.3	3.2	2.8	4.5	8.0	2.7	6.6	5.9	4.6
NO ANSWER	10.0	13.3	4.7	10.9	15.0	13.6	25.0	17.6	13.2
6. What effect did the freeway control have on the overall travel time?									
REDUCED	65.0	59.4	65.4	61.0	42.5	45.9	32.9	21.6	52.7
INCREASED	8.3	9.2	15.9	9.1	23.0	8.1	14.5	23.5	13.5
NO CHANGE	21.7	24.4	15.9	27.2	28.3	37.9	40.8	45.1	27.6
NO ANSWER	5.0	7.0	2.8	2.7	6.2	8.1	11.8	9.8	6.2
7. What effect do you feel the freeway control had on traffic operation?									
IMPROVED OPERATION	75.0	72.8	82.2	75.4	52.2	54.1	42.1	41.1	65.6
NO CHANGE	16.7	14.3	9.3	19.1	20.4	32.4	36.8	33.3	19.7
MADE CONDITIONS WORSE	3.3	4.1	4.7	1.8	19.5	5.4	6.6	11.8	6.9
NO ANSWER	5.0	8.8	3.8	3.7	7.9	8.1	14.5	13.8	7.8
8. Do you feel that the control plan should be continued?									
YES	80.0	76.5	81.3	74.5	47.8	59.5	38.2	33.3	65.5
NO	3.3	3.9	2.8	12.7	28.3	24.3	23.7	23.5	12.7
NO OPINION	13.4	9.5	12.2	11.0	14.2	10.8	27.6	33.3	14.5
NO ANSWER	3.3	10.1	3.7	1.8	9.7	5.4	10.5	9.9	7.3

TABLE 13

TIME INTERVALS BETWEEN SUCCESSIVE VEHICLES, DUMBLE SIGNAL INSTALLATION

Vehicles	No. Observed	Interval (sec)
0-1	147	3. 3
1-2	93	2. 7
2-3	51	2. 4
3-4	5	2. 2

TABLE 14

COMPARISON OF THEORETICAL AND OBSERVED METERING RATES, DUMBLE SIGNAL INTALLATION

Time	Theoretical 5-Min Rate	Observed 5-Min Rate
7:05-7:10	40	35
7:10-7:20	25	24
7:20-7:25	40	35

Operation of the Controls

Two problems did arise with the manual metering operation. One was that the officers were visible from the freeway and the freeway traffic tended to slow down slightly in the vicinity of the metering point. The second was the tendency on the part of some of the policemen to change the metering rates according to their subjective evaluation of the freeway and merging traffic conditions. The tendency which was noted was an attempt to reduce the length of the queue at the metering point when, in some cases, one object of the control may have been to develop a long queue (to discourage the use of that ramp). In some instances the officer's judgment may have been superior to the fixed-time plan (especially in the case of upstream accidents) but their changes in the controls conflicted somewhat with the research objective of evaluating the operation under a particular control plan.

Signal Installation at Dumble.—The starting headways for metered vehicles at the Dumble signal installation are summarized in Table 13. These values are

headways at typical signalized intersections which may be attributed, in part, to the conservative reaction of drivers in an unfamiliar situation.

Table 14 is a comparison of the observed metering rates at Dumble with the desired theoretical rates established by the master freeway control plan. The observed rates are seen to be slightly lower due to the comparative sluggishness in the starting headways as explained above.

Two police officers were stationed at the signal to enforce metering on all but one day. On that day metering was accomplished by the signal alone and out of 123 vehicles metered, only 5 violations were observed. There was no significant difference in the starting headways or the number of violations with or without the policemen.

The effect of the bulk-service technique of ramp metering on the critical gap for merging ramp vehicles is illustrated in Figure 24. The critical gap for bulk service metering is seen to be 3. 1 seconds compared to 2. 5 seconds for normal operation. The reason for this is not clear, but it is suspected that (a) metered vehicles have a greater relative speed and (b) metered drivers are more conscious of the merging maneuver and are therefore more cautious.

The general conclusions of those observing the operation were that if metering is to be accomplished on the frontage road, the amber phase is a necessity. However, the inclusion of an amber phase in each cycle limits the maximum metering rate. In order to obtain higher metering rates using the bulk service technique, the cycle length must be increased allowing several vehicles to arrive at the freeway merging area at one time. This creates a platoon of ramp vehicles and raises the critical acceptance gap which for a given ramp and freeway volume will result in a lower merging level of service than that obtained during normal operation or during single-vehicle metering. For these reasons, it is suggested that in the future the metering station be located on the ramp, and that vehicles be metered one at a time using just the red and green phases.

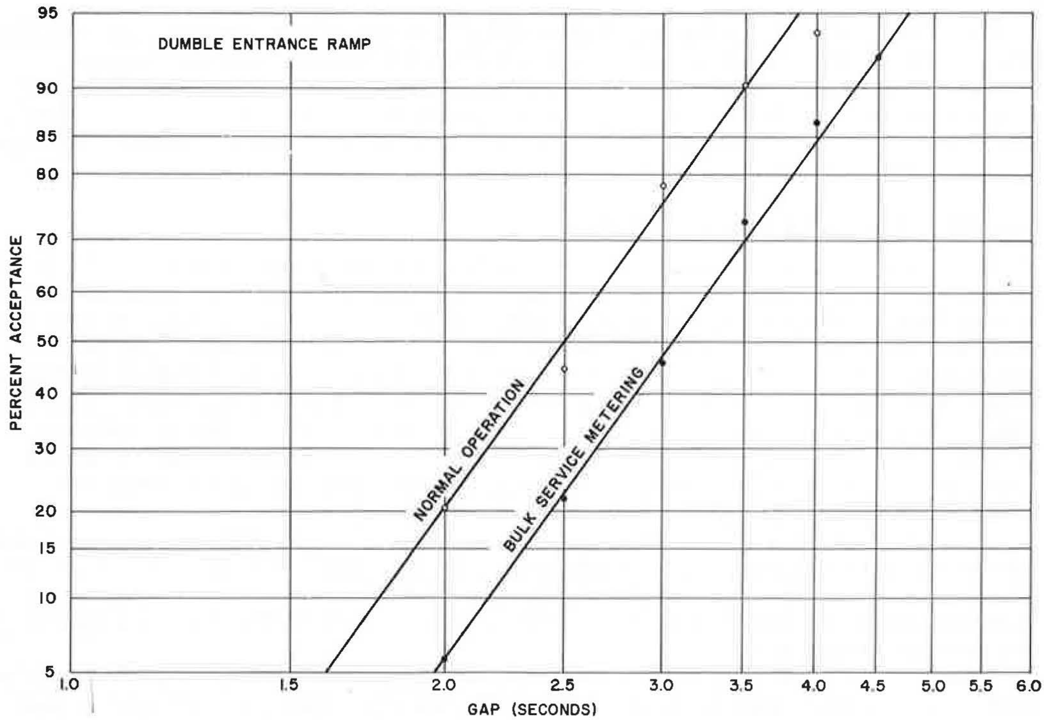


Figure 24. Comparison of percent acceptance for moving merging vehicles during normal and controlled operation.

Ramp Closure.—The personnel responsible for closing and reopening the three entrance ramps (with barricades) were very dependable and the actual closure times were very close to the desired closure times each day. This ramp closure technique, while somewhat crude, worked extremely well, and created no special problems.

Overall Effects of the Controls

The overall effect of the controls was a sizable improvement in the traffic operation on the inbound Gulf Freeway during the morning peak period. The total travel time on the inbound freeway and frontage roads was reduced by about 360 vehicle hours, roughly a 25 percent reduction, while a 48 percent reduction in travel time was accomplished on the freeway between Broadway and Griggs. Most of the additional travel time on the arterial streets naturally appears in the form of delay at the critical intersections. Delays at five critical intersections were studied and a total increase in delay of 23 vehicle-hours was found at these intersections. Thus, the overall decrease in travel time was about 320-330 vehicle-hours which, assuming an average vehicle occupancy of 1. 5 persons per vehicle, represents a savings of about 475-500 man-hours per day.

The observed changes in the travel patterns were not unexpected. Generally the volumes decreased on entrance ramps that were controlled and increased on uncontrolled ramps and the freeway near Broadway. The total volume using the freeway from 7-8 a. m. decreased about 5 percent, although between Broadway and Griggs the total 7-8 a. m. volume using the freeway was virtually unchanged.

Individual vehicles saved a substantial amount of time on inbound freeway trips in the 7-8 a. m. period, especially between Broadway and Dumble Streets. Travel time savings on a trip between Broadway and Dowling were as much as 10 minutes.

The additional delay occurring at the critical arterial street intersections was slight. Thus, a considerable system travel time reduction was produced by the controls.

The questionnaire sent to the ramp motorists revealed that about 70 percent of the ramp motorists felt that the freeway traffic operations were improved by the controls and that the controls should be continued. This indicates the readiness of the motorists for some type of freeway ramp controls.

Direction of Future Ramp Control Work

The two control plans which have been evaluated to date have both been of the fixed-time type. Time alone determined the type and degree of controls which were in effect at each ramp. Historic traffic data obtained from previous studies were used to estimate the demands at the various locations. While this type of demand-capacity analysis provides a rational basis for a control system and was an excellent first step in developing a control system, it is not at all flexible and cannot respond to reduced-capacity occurrences nor even normal fluctuations in demand or daily variations in demand. This fairly simple type of control system was necessary to determine the order of magnitude of the benefits to the peak-period traffic and to determine the responses or reactions of the motorists to these controls.

Thus the study shows that one of the next steps should be research on a control system which will respond to traffic conditions on the freeway. This type of control system can base the individual ramp controls on what the freeway traffic conditions are at the particular instant of time rather than what they were at that time on a typical day several months ago.

This can best be accomplished by immediate installation of ramp control signals at each of the inbound entrance ramps. This will permit studies to determine the best type of ramp signal and the best operation of the signals. The first signal installation should be capable of manual operation and should also have several fixed time settings available. With the signals, the problem of subjective decisions of the police officers

personnel at the ramps will be greatly reduced. The differences in freeway level of service under one-at-a-time or two-at-a-time metering can be tested.

Such a signal system could be operated at one or two locations according to traffic conditions on the freeway as detected manually. Thus the proper variables (gaps, 3-lane volume, 1-lane volume, density, etc.) to be detected and the proper detector location can be determined. This determination is, of course, necessary before a fully automatic control system can be developed.

The final step in an automatic control system is the interconnection of all control locations so that they can truly be operated as a system. Some form of computer or real time control system may be required to accomplish this.

CONCLUSIONS

1. The control plan which was tested was quite successful at reducing the Gulf Freeway congestion during the morning peak period. Overall freeway travel time was reduced about 25 percent.
2. The control plan tested produced little adverse effect on the arterial street system near the Gulf Freeway.
3. The total 7-8 a.m. volume using the inbound Gulf Freeway decreased about 5 percent during the control study.
4. Individual vehicles saved as much as 10 minutes in traveling between Broadway and Dowling during the control study.
5. The demand-capacity technique provides a good method for determining a fixed-time control plan.
6. A fixed-time control plan lacks the flexibility to deal with reduced-capacity situations such as accidents and adverse weather.
7. The motorists complied extremely well to the police-operated metering controls.
8. It is desirable that ramp metering signals release vehicles onto the freeway one at a time rather than allowing multiple entries.

9. Public acceptance of the controls was good and the study indicates that the motorists are prepared to accept a freeway control system on a continuing basis.

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Appendix

A news release (Fig. A-1) was issued to the newspapers and radio and television stations on January 19 by the Texas Highway Department. This release described the control study and listed the times and dates of control for the two-week study. The news articles that appeared in the two Houston newspapers are shown in Figures A-2 and A-3.

At the end of the first week of operation, it was apparent that a longer study period was required to reach a steady-state condition in traffic patterns. The news agencies were informed by phone of the extension of the control study. The resulting news article (Fig. A-4) did not contain specific times on the controls.

The statement by the City Councilman in that article was the only published criticism of the study.

OFFICIAL NEWS RELEASE

The second in a series of freeway control studies, conducted by the Texas Highway Department, City of Houston, and the Texas Transportation Institute of Texas A&M University under the sponsorship of the U. S. Bureau of Public Roads, will be placed in effect on the Gulf Freeway during the morning peak periods from January 26 to February 6, 1965.

The controls will consist of closing three ramps and metering, or limiting the number of vehicles that enter the freeway at the other ramps from State Highway 225 to Dowling Street. The following control plan for the time and type of controls to be placed on each ramp was developed from the results of numerous studies completed during the last twelve months:

Ramp	Type of Control	Time Control Is In Effect
State Highway 35	Meter	6:55 - 7:30 a.m.
Woodridge	Meter	6:55 - 7:30 a.m.
Mossrose	Meter	6:55 - 7:30 a.m.
Griggs	Close	7:05 - 7:20 a.m.
Wayside	Meter	7:00 - 7:30 a.m.
Dumble	Meter	7:05 - 7:30 a.m.
Cullen (South)	Close	7:05 - 7:30 a.m.
Cullen (North)	Close	7:05 - 7:30 a.m.

Previous studies in Houston and other cities have proven the need for some type of traffic control during the periods of peak traffic demand to maintain a high level of efficiency on urban freeways. The objectives of these studies are to provide the information necessary to develop an automatic freeway control system.

The results of the first control study conducted last August and limited to a short section of the Gulf Freeway substantiated the claim that traffic flow can be improved by controlling the critical entrance ramps during the time of peak loading. That study also indicated that the small number of motorists who are diverted from the freeway during the control period can be accommodated on the city street system. The second control system which will include all ramps in the congested area of the Gulf Freeway is expected to produce similar improvements but over a longer section of roadway and for a greater number of motorists.

Figure A-1.

THE HOUSTON CHRONICLE

Tuesday, January 19, 1965

Jan. 26 Until Feb. 6

Close Few Gulf Freeway Ramps

Some ramps for inbound traffic on the Gulf Freeway will be either closed or the amount of traffic regulated by the State Highway Department beginning Jan. 26 and continuing until Feb. 6.

The ramps admitting traffic on the freeway from Reveille, Woodridge and Mossrose will meter traffic from 6:55 a.m. to 7:30 a.m.; the Griggs Rd. ramp will be closed from 7:05 to 7:20 a.m.; the Wayside and Telephone ramps will meter traffic to the freeway from 7 a.m. to 7:30 a.m.; the Dumble ramp will meter traffic from 7:05 to 7:30 a.m., and the Cullen ramp will be closed from 7:05 a.m. to 7:30 a.m.

"Metering" is a process whereby traffic will be allowed on the freeway by state highway engineers as gaps appear in freeway traffic.

Traffic will have the option at those ramps of either waiting or taking alternate routes into the downtown area.

Traffic must use alternate routes where ramps are closed.

District Highway Engineer W. E. Carmichael said that beginning Wednesday, signs will be put up at the inbound ramps affected, giving directions to motorists.

Last August a similar study was made between Wayside and Dowling. Carmichael said this is an expanded study.

Figure A-2.

THE HOUSTON POST
WEDNESDAY, JANUARY 20, 1965

Gulf Freeway Ramps Will Be Closed in Another Test

Nine ramps on the Gulf Freeway will be closed or curtailed during morning rush periods from next Tuesday through Feb 6.

This is the second in a series of tests aimed at relieving rush-hour hardening of the traffic artery.

THE NEW TEST, more extensive than the first, will close or limit traffic on all ramps between Gulfgate Shopping City and Dowling Street downtown.

Motorists who normally use these near-downtown ramps will be diverted to other routes leading downtown. The idea is that everybody will get to town faster if the short-trip freeway drivers are eliminated.

This appeared to be the result in the first freeway closing test last August.

The Texas Highway Department reported recently that freeway traffic got to town 25 per cent faster during the first test, even though the freeway was carrying more traffic.

THESE RAMPS and times are involved in the new test:

State Highway 35, Woodridge and Mossrose—limited traffic from 6:55 to 7:30 AM.

Griggs—closed from 7:05 to 7:20 AM.

Wayside and Telephone—limited traffic from 7 to 7:30 AM.

Dumble—limited traffic from 7:05 to 7:30 AM.

Cullen—both ramps closed from 7:05 to 7:30 AM.

Figure A-3.

THE HOUSTON CHRONICLE

Saturday, February 6, 1965

3 Freeway Ramps To Close an Hour

The State Highway Dept. will close three Gulf Freeway ramps on an "indefinite" basis to facilitate the free flow of downtown traffic between 7 and 8 a.m.

State Dist. Highway Engr. Wiley Carmichael said the ramp at Griggs Rd. and two ramps at Cullen will be closed during the peak periods.

Six other ramps, at Reveille, Woodridge, Moss Rose, Wayside, Telephone and Dumble, will be regulated by traffic patrolmen during the morning rush period. The officers will permit traffic to enter these ramps only when it will not interfere with through traffic.

These ramps were closed in August on a test basis. The test was so successful, said Carmichael, that the ramps will now remain closed on an "indefinite" basis.

City Councilman Bill Elliott protested that city drivers, who pay the bulk of the taxes, are being penalized for the benefit of rural dwellers who work in Houston.

Figure A-4.

Discussion

PATRICK J. ATHOL, Project Supervisor, Expressway Surveillance Project, Oak Park,

periment on the Gulf Freeway. It is refreshing to read of practical experimentation in a field where the theory to date has mushroomed faster than the supply of experimental data.

As the authors point out, this paper represents only one phase of their continuing control effort. The paper limited its ramp control scope to a pretimed system based on historical capacity data. This type of system is excellent for manual techniques where the data sample is selected subsequent to the control experiment. However, there are certain prevailing operational and environmental conditions on most freeways which lead me to think that some adaptive control system will evolve for general application. Such commonplace occurrences as rain, snow, bright sunshine, darkness, accidents and other flow disturbances vary the critical control parameters. These variations require some adaptive control scheme if the system is to prevent congestion consistently.

The evaluation of freeway performance using the measure of vehicle miles of travel and vehicle minutes of travel time is simple, effective and independent of the control scheme. The technique can be readily applied to conventional manual or automatic detection volume records. The authors note a closing error of less than one percent between the input and output volumes in the manual counting procedure. Experience with automatic detection systems (9) shows agreement well within the same error range. However, a one-percent error between input and output volumes, for a total input volume of 10,000 vehicles, amounts to 100 vehicles or more nearly a 14 percent error in the storage calculation of 700 vehicles. Adjustments may be made for systematic errors at the end of the control period, but this limits the evaluation technique as a control parameter. It is not until after the study that one can assess the reliability of the data and the magnitude of the closing error.

The authors at one point summarized their data in terms of speed derived from vehicle miles of travel and vehicle minutes of travel time. Speed would appear to have the advantage of combining total travel with total travel times, thereby differentiating data samples with equal travel time, but varying total travel. The plot of vehicle

vs vehicle minutes for a section of freeway gives a curve similar to the conventional volume density graph.

The results presented clearly show the significant benefits to be derived from control. There appear to be two points which should be carefully considered in future experiments: first, the occurrence of benefits related to the time of control and, second, the selection of comparable data samples. From the report it appears that benefits were ascribed to control at times when no control was exercised. At 7:00 a.m., with control, the data showed about 150 fewer vehicles stored in the system, and yet effective control was only necessary after 7:00 a. m. These benefits should probably be ascribed to other variables not recorded. As a parallel example, a study of freeway operations (9) showed, in the comparison of two samples of 33 days each, at the same location, a significant improvement in operations where no control was exercised in either sample. The changes could only be ascribed to seasonal variations in demand, but these changes could not be detected from the input-output data. If any noneffective control had been used at that time, significant benefits might have been attributed to the control scheme. The evaluation technique of input and output with zero closing error is in no way a control measure in comparing data samples. Under all conditions of weather and freeway incidents there will be a zero closing error with accurate counting. The closure error only reflects the accuracy of the counting procedure. Freeway operations studies are still sadly lacking in measures of experimental control. Data are collected in the same manner that a physicist would measure the volume of a gas oblivious to the effects of temperature and pressure.

The selection of data was based on incident-free days, which are difficult to define, and the data showed considerable daily variation. From a practical standpoint, control will have to survive days with incidents, and indeed the numerical saving may be highest at those times. Perhaps the concept of potential performance may assist in describing data. Results would then be related to potential performance. There may be two potential levels, a higher level with control and another level without control. The range of values for daily performance with or without control will probably overlap. A good day without control would often show an improvement over a bad day with control. Thus, days with incidents need not be simply discarded, but rather related to the potential level. Data samples under varying conditions may then be compared to potential levels without selecting "special days" in a qualitative manner.

The authors have used a test area in which the surface street traffic may readjust with minimal difficulty. There are probably many urban areas where the contiguous streets are less permeable to diverted traffic. Where this exists diverted traffic may radically reduce the time savings benefits to the control system. Indeed there may be areas of negligible system time savings where the improved safety of operation through control will justify the system.

The growth of freeway ramp control must surely increase; improvements have been verified in many areas and the continuing debate will perhaps center around the extent of those benefits. There seems to be a glaring need to establish the causative parameters in freeway operations. The traditional before-and-after studies are limited in value if variations in results go unexplained. Evaluation measures may be adequate in summarizing effects, but are inadequate in pinning down cause-and-effect relationships.

Looking to the future, the astute administrator might reap the greatest harvest by installing control years before congestion develops (10). The system itself would then modify the growth of traffic demand at the various ramp loading points. Prevention is usually better than cure, and the case of congestion may be no exception.

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HUGH C. KENDALL, Director of Research, General Railway Signal Co., Rochester, New York—The traffic corridor linking Houston and its suburbs to the south consists of the Gulf Freeway and parallel surface streets. The authors demonstrated that, by using a simple fixed time control program on nine entrance ramps to the freeway, a substantial improvement in the quality of traffic flow in the corridor could be achieved during the morning peak period. Specifically, 360 vehicle-hours travel time was saved on the freeway due primarily to an increase in speed at approximately the same volume. Travel time on the surface streets was increased by 30 vehicle-hours. Net travel time savings in the corridor amounted to 330 vehicle-hours.

The fixed time control plan was designed using the demand-capacity concept, in which the capacity of each bottleneck area was estimated to be the highest 15-minute volume over a number of days of observation. The estimated demand at each bottleneck area was ascertained from historical data concerning the character of the freeway volume input function, as well as origin-and-destination studies of freeway traffic. The success of the control plan is a tribute to the ingenuity of the authors, and to the cooperation of the 1500 to 2000 motorists which it is estimated were directly affected by the ramp controls each day. In the opinion of this discussor, great good has come from simple means. Possibly five years from now, we will be able to establish the point of diminishing returns between complexity and simplicity in the optimization of corridor operations. The time is not now. Too many hypotheses remain to be proven. I am encouraged by the mounting number of reports covering studies in the field of freeway operations alone.

I would like to commend the authors for their excellent report covering their corridor studies, and spend the remaining moments on the subject of traffic-responsive corridor control. I am not unmindful of the thoughts of the authors in this regard, nor of the suggestions which have been made by others in the field in their reports and discussions at this meeting. Many ideas have been suggested. It may take some time, however, to establish the economic justification of some of these ideas in terms of benefits gained as applied to corridor operation.

speeds are high, they become the backbone of any corridor. Freeways which are exposed to uncontrolled demand frequently shift from high volume-high speed operation to high volume-low speed operation as demand approaches or exceeds practical capacity. It has been shown that practical capacity is a function of adverse weather and prevailing driving conditions. Furthermore, practical capacity can be severely limited by a disabled vehicle or accident. To sustain a high volume-high speed condition on a freeway, therefore, requires an up-to-date knowledge of its practical capacity at all points. The fixed time control program demonstrated by the authors worked well, due primarily to the ability of the program to hold demand as a function of time at some level below the predicted practical capacity in the critical areas of the freeway. In other words, good agreement between the expected and what actually happened was evident.

Improvements to the control system demonstrated by the authors could be reasonably expected by introducing means for determining the up-to-date practical capacity of each critical area. Using this information, demand in these areas could be aligned with practical capacity on a continuously updated, rather than on a historically preselected basis. I do not wish to imply here that means are at hand to reliably accomplish this objective under normal free flow conditions. Many studies concerning the dynamic relationship between volume, speed, density and lane occupancy under typical free flow conditions have been made. These studies have shown wide variations of volume associated with a fixed value of average speed, density or lane occupancy, making the reliable prediction of practical capacity from these variables very difficult. Refinements in instrumentation used to measure these and other variables may well point the way toward better prediction of practical capacity from current traffic stream measurements.

On the other hand, the automatic detection of the occurrence of a temporary capacity restriction on a freeway due to an accident or disabled vehicle appears promising today from what has been learned thus far. Such restrictions are usually accompanied

by a marked discontinuity in either volume or lane occupancy immediately downstream of the restriction.

As a first step toward improved control, the authors might well consider modifying the present Mark I fixed time system to include the relatively simple logic, the necessary interconnect, and a modest number of sampling detectors along the center lane of the freeway to automatically detect such discontinuities. In the opinion of this discussor, the benefit-to-cost ratio would be high. The metering rates of ramps both upstream and downstream of a capacity restriction could be automatically adjusted to take account of the restriction. Metering of traffic on downstream ramps could be temporarily suspended. Metering on upstream ramps could be set to minimum. Motorists approaching the freeway upstream of the restriction could be advised through appropriate signing, to use the surface streets in their own best interest to bypass the restriction. Splits and cycle lengths at critical intersections could be adjusted to accommodate a temporary increase in traffic demand along the corridor. With the removal of the temporary capacity restriction on the freeway, normal ramp metering and operation of the critical intersections could be resumed.

The modified or Mark II control system would operate as a multiple mode fixed time system. A number of modes could be automatically selected by the logic associated with the traffic flow discontinuity detection equipment. Manual overide or mode selection due to other inputs could also be provided. One such input could be the prevailing weather conditions on the freeway.

The strengths in the Mark II system would lie in its simplicity and its predictable behavior under traffic conditions which are relatively simple to measure and observe, yet are extremely important to take into consideration in minimizing overall corridor travel time through control. The weaknesses would lie in the thought of possibly being able to do a better job through more sophisticated traffic-responsive control, in which the measured behavior of traffic at all points in the system is tightly coupled to the control decision-making process. Let us refer to this system as Mark III, and possibly the ultimate.

There are those who advocate a Mark III corridor control system consisting of a large network of vehicle detectors, high-speed digital computer, and staff of programmers as a present-day solution to the problem. As a supplier of systems of transportation control, we find ourselves involved in the design of many types of systems whose operation could be regarded as falling within the framework of Mark I, Mark II or Mark III. We of course are most interested in Mark III systems, since large amounts of equipment are involved which keep our shipping room busy. We are called upon, however, to furnish handbooks of operating instructions for all our systems. We are well along on handbooks covering systems of the Mark I and Mark II variety as applied to the control of vehicular traffic. It is only fair to tell you, however, that we are experiencing some difficulty in the completion of the programming section for our Mark III system. We hope that you will help us write it.

DONALD O. COVAULT, Professor of Civil Engineering, Georgia Institute of Technology, Atlanta—This discussion will not specifically concern itself with the content of the excellent paper presented by Pinnel, Drew, McCasland and Wattleworth, but will be concerned with a philosophical discussion of the decision-making processes which might be involved in the problem of ramp control on a freeway section.

The decision to close or meter a ramp should initially be based on certain objective criteria which can be easily measured. These criteria usually are concerned with the use of travel time and travel distance as measures of effectiveness of freeway and arterial street operation. Of great importance also is the concept of system evaluation in measuring travel time and travel distance; i.e., one must consider the arterial streets and freeway as a system in developing the decision-making processes for ramp control.

The use of subjective criteria for ramp metering and ramp closing must also be considered before such an operational procedure as ramp control is initiated. Included in the subjective criteria would be the need for routes for fire and police vehicles and other emergency vehicles and the compatibility of the closing or metering of a ramp with land use and other usually nonmeasurable aspects of transportation.

Also related to the subjective problem of ramp closing and ramp metering is the problem of street management itself. Some city traffic engineers may look upon the arterial street systems as basically the problem of major concern in street management and that freeway operation is a problem which in many cases is associated with the operation of a system outside their authority; that is, this system "belongs" to the State Highway Department. Consequently the city traffic engineer may concentrate much of his efforts on the movement of vehicles on arterial streets and may mitigate the importance of the freeway itself. The problem of optimization quickly appears when one considers the relationship of street management to the operation and management of the freeways. The question that must be answered here is: Must one close ramps and meter ramps in such a way as to optimize travel or travel time on the freeway or to optimize travel or travel time on the arterial streets or to optimize these parameters on the arterial streets and the freeway system? From studies made on the Atlanta freeway system it was found that simultaneous optimization of both freeway operation and arterial street operation by ramp closing was not possible. In all cases where entrance ramps were closed, the operation of the freeway improved in terms of travel time but rather serious congestion problems were created, mainly through turning movements, on the arterial street system.

The problem of public acceptance of ramp metering and ramp closing must also be considered in the decision to close or not close or to meter a ramp. Although objective criteria may indicate that the optimum way to operate a freeway and arterial street system would be to close or meter ramps, public opinion may prevent this from happening because of preconceived ideas as to ways of driving, vested interests, etc.

[illegible]

entrance ramps, which may be undesirable during peak periods, be permitted during off-peak periods. These ramps may provide for optimum street management which could not otherwise be provided if the ramps were not allowed to operate. Furthermore, when these ramps are permitted to operate normally during the off-peak periods they do not create the problems to freeway traffic flow that occur when traffic flows are approaching breakdown densities.

Because ramp closing and ramp metering may create rather extreme changes in the operation of a freeway and arterial street system, it is highly desirable that the general public have prior knowledge as to the location and timing of ramp metering and ramp closing. The provision for complete automatic control may be too flexible for the driver to accept. That is, he would have no assurance that once he came to a particular ramp that this ramp would be open for him to have use of the freeway; or he may have to wait a considerable length of time in a queue on a metered ramp in order to get on the freeway. Furthermore, it is quite usual for unusual things to happen on the freeway. From this point of view it may be desirable to permit the freeway to operate on a fixed-time control system so that the driver can expect certain ramps to operate in the same manner at a certain time each day.

In conclusion, ramp control appears to be highly feasible. Ramp control, however, should be used with a great deal of discretion and one must be very careful that problems are not overlooked which may be created on the arterial streets associated with the ramp closure or metering when these control measures are adopted.

CHARLES PINNELL, DONALD R. DREW, WILLIAM R. McCASLAND, and JOSEPH A. WATTLEWORTH, Closure—The authors would like to express their thanks to the three discussers for their stimulating and considered comments on the paper.

Mr. Athol expresses a great enthusiasm for the motorist benefits which can be achieved with peak-period freeway control. The authors certainly share this enthusiasm and also share his belief that the peak-period controls should be of the traffic-adjusted type rather than of the fixed-time type reported in this study. The ramp control plan described in this report represents merely one step or phase in our program to develop a final control system. Subsequent to this study several traffic-adjusted metering systems have been tested and others will be tested in further developmental work toward a final control system design.

Like Mr. Athol, we regret that the traffic researcher cannot collect data with the same precision and under the highly controlled conditions as can the scientist in his laboratory. Until it is possible to do this the traffic researcher must use the best techniques at his disposal. The study techniques used in the evaluation of the effects of the controls in this study are admittedly not perfect, but they are believed by the authors to be highly reliable and better than other techniques which were available for this purpose.

Mr. Athol raises the question of the adequacy of the statistical sample sizes of the number of days of data used in the before-and-after evaluations of the effects of the controls. His comments are based on his experience in Chicago in which the operation during some days of control is worse than during some days without control. Our experience on the Gulf Freeway, based both on the study reported here and eight months of subsequent operation of the ramp controls, is somewhat different. The operation was very consistent both before and during the controls; daily variations in each case were small, but seasonal variations were noticed. Barring transient effects when a control study is initiated and barring adverse weather and accidents, the operation on any day during control is better than any day without control. However, this is related to the severity of congestion on the facility before controls were initiated and this experience would vary from one facility to another.

In perhaps a related point Mr. Athol commented on the fact that the data were based on incident-free days, i.e., days on which no accidents, stalled vehicles, adverse weather or other factors affected the traffic flow. Several factors affect the traffic flow, among them: (a) the operation of ramp controls, (b) weather, (c) light conditions, (d) accidents and (e) disabled vehicles. We were trying to isolate the effects of just one of them, namely the operation of the ramp controls, so it seemed appropriate to hold constant the effects of the other factors, and this is the reason for selecting incident-free days for the before-and-after analysis. Most traffic researchers believe that ramp controls can produce the greatest motorist benefits when a severe capacity reduction prevails on the freeway, such as when an accident or adverse weather affects the traffic flow. Thus, the use of incident-free days in the before-and-after evaluation perhaps provides a conservative estimate of the motorist benefits resulting from peak-period ramp controls.

Mr. Kendall raised the very important question of the determination of the point of diminishing returns on investment in control equipment. This is somewhat surprising coming from a manufacturer's representative but is nevertheless a good point.

All the authors can say regarding this point is that we don't know where the point of diminishing returns is but that finding it is an important part of our current research program. Our approach is to determine the magnitude of the benefits to the motorists that can be derived from control systems of varying levels of sophistication. Using Mr. Kendall's terminology, we have at the present time looked at the "Mark Ia" and "Mark Ib" systems and hope to experiment a little with "Mark II" and "Mark III" before deciding where the optimal investment point may be. We are also trying to write our own handbook for the "Mark III" system.

The "horseback" methods which we have tried have yielded good results and it could be that it will be hard to improve on them. In this research stage, however, we feel that more sophisticated methods should be evaluated.

Dr. Covault has rightfully raised some of the very important practical, political considerations which can become very important. Research on traffic control is not an "ivory tower" situation; the very useful, practical, beneficial results must be "sold" to someone, be it the governing political agency or the people themselves.

We were very fortunate in Houston to get almost 100 percent cooperation from the motorists; at least, very few have voiced opposition. We were also fortunate to be working with two extremely cooperative agencies, the Texas Highway Department and the City of Houston, each of which recognizes its own responsibility and each of which has adopted the "systems approach" to the Gulf Freeway problem.

Again the authors would like to thank the three discussers for their thought-provoking comments on this paper.

Merging Behavior at Freeway Entrance Ramps: Some Elementary Empirical Considerations

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This paper discusses an elementary empirical analysis of merging behavior, and in particular of gap acceptance and rejection behavior at a freeway entrance ramp. The tone of the paper is purely empirical. No attempt is made to develop a theory of merging, nor to validate any existing analytical or simulation models of the merging process. Rather, emphasis is placed on the development of an improved analytical methodology and on the collection of basic empirical information of the type which might be utilized either as input to future theoretical studies, or in the design of a real-time freeway control system.

•CONSIDERABLE attention has been devoted in recent years to the analysis of gap acceptance and merging behavior at freeway entrance ramps. A selected bibliography (Appendix A) lists some 62 references to recent research in this or closely related fields.

MERGING BEHAVIOR—AN ANALYTICAL FRAMEWORK

A review of these studies suggests that most empirical research to date has been based on a "static" rather than a "dynamic" interpretation of the merging process. Gap acceptance and rejection has been described in terms of "static" measurements of mainstream time headways made at a single, arbitrarily defined point in space (conventionally, this point has been taken as the nose of the entrance ramp). No real attempt has been made either to take account of the positioning of the ramp vehicle within a given mainstream gap, nor, more importantly, to allow for the relative motions and trajectories of the merging vehicles.[1]

The implications of this comment may perhaps be most clearly illustrated by reference to Figure 1. This figure depicts the time-distance traces of a series of ramp and mainstream vehicles passing through the merge area of a freeway entrance ramp. Time is plotted continuously along the abscissa of the diagram and distance vertically up the ordinate. Mainstream vehicles are denoted by solid lines and ramp vehicles by dashed lines. Two points, X and O, are defined for each ramp vehicle trace: X denotes the point at which the ramp vehicle first encroaches onto the adjacent mainstream lane, O the point at which the vehicle finally enters that lane. The distance X-O therefore constitutes the length of the physical merging maneuver. All traces are defined with respect to the front of the vehicle in question.

[1] There are, as may be expected, a number of exceptions to this generalization. In particular, reference is made to the empirical work of Hurst et al (32), Knox (38) and Vardon (55) and to the related theoretical analyses of Bisbee and Conan (4), Haight (24), Haight et al (25) and Jewell (34). Figure 2 summarizes graphically the results of some typical previous "static" analyses.

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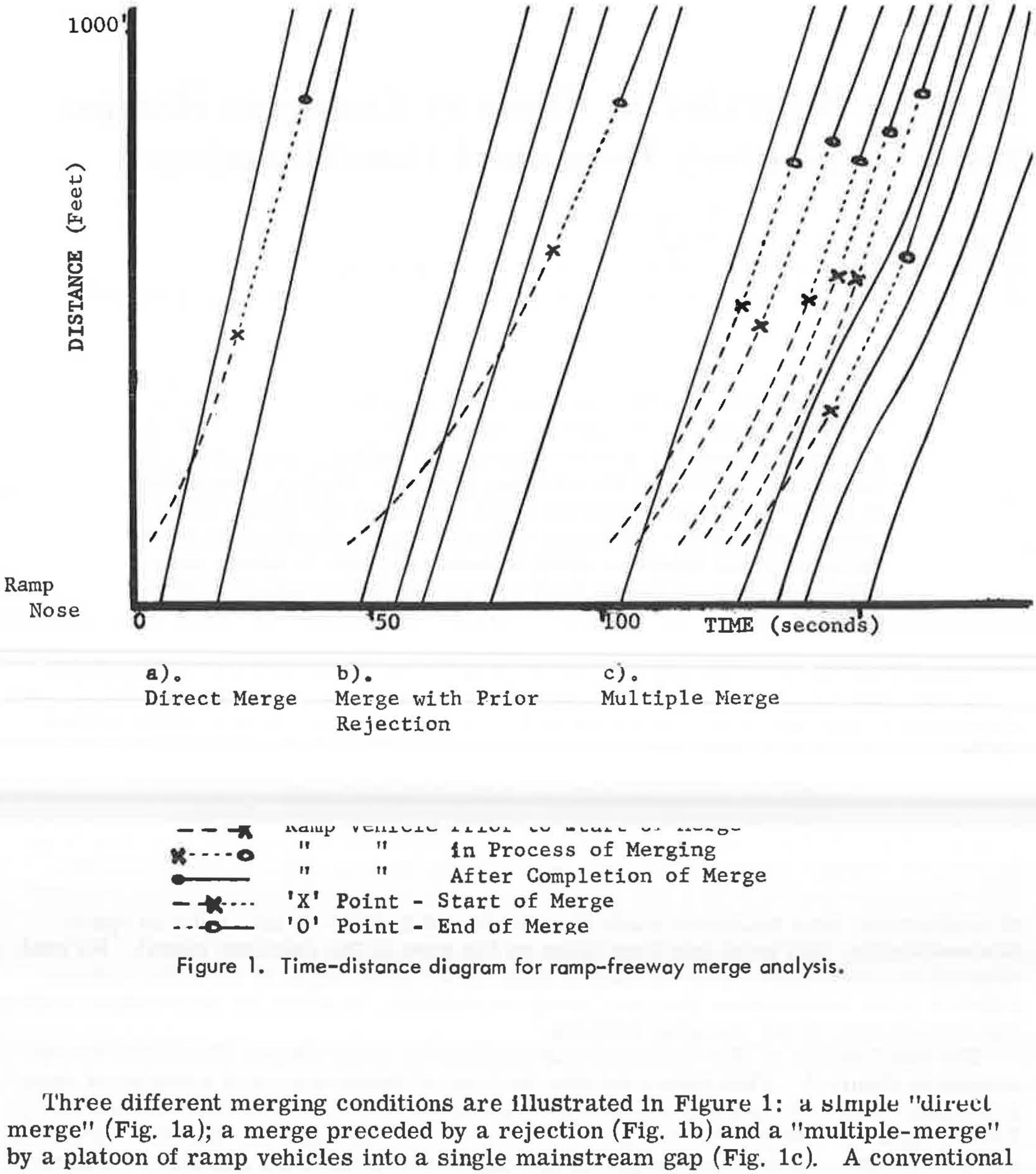
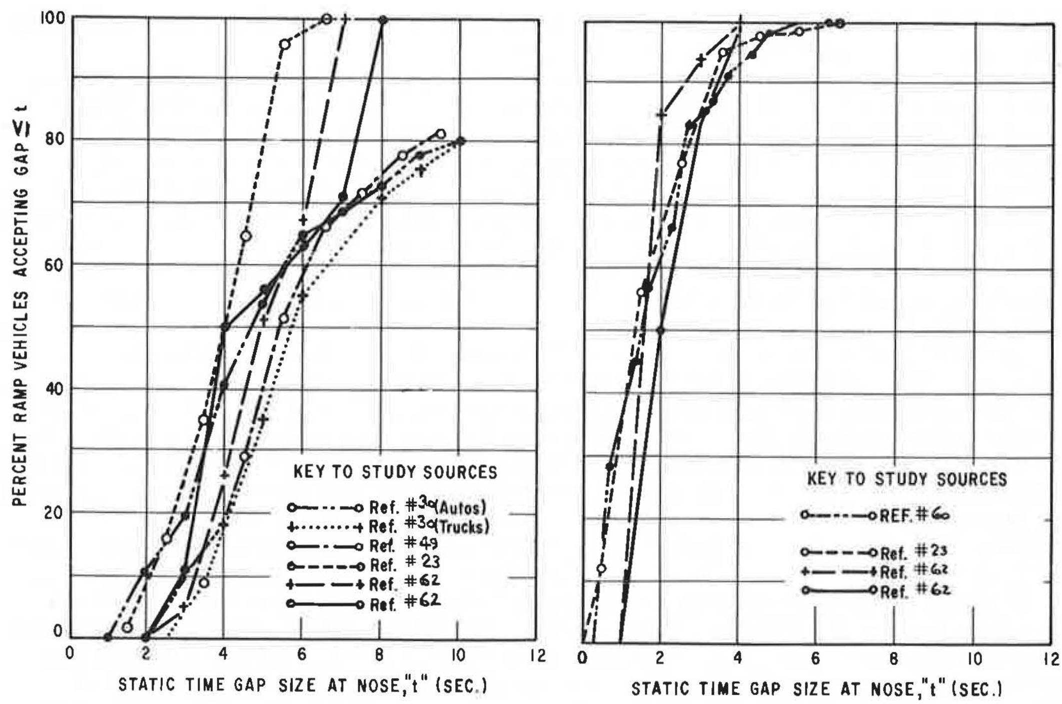


Figure 1. Time-distance diagram for ramp-freeway merge analysis.

Three different merging conditions are illustrated in Figure 1: a simple "direct merge" (Fig. 1a); a merge preceded by a rejection (Fig. 1b) and a "multiple-merge" by a platoon of ramp vehicles into a single mainstream gap (Fig. 1c). A conventional "static" analysis would describe each of these conditions in terms simply of the ramp vehicles' acceptance or rejection of the series of mainstream headways measured statically at the ramp nose. The result would be only a partial and possibly a misleading description of the merging process.

In the case of the multiple merge illustrated in Figure 1c, for example, a total of five vehicles would be recorded as having accepted a gap of 25 seconds; a sixth vehicle would be recorded as having rejected that gap in favor of the subsequent value of 13



a). MERGE FROM STOPPED POSITION
(i.e. FOR RAMPS WITH NO ACCELERATION LANE
OR RAMPS SUBJECT TO STOP-SIGN CONTROL).

b.) CONTINUOUS UNCONTROLLED MERGE
(i.e. FOR RAMPS WITH FULL ACCELERATION LANES).

Figure 2. Summary of results of existing gap acceptance studies.

seconds. This is clearly illogical. The driver of the sixth vehicle, at the instant he reached the ramp nose, was faced, not with a gap of 25 seconds but rather with a "lead-time" to the preceding ramp vehicle of 2 seconds and a "lag-time" to the succeeding mainstream vehicle of approximately 1 second. It was this 2 second-1 second gap structure which the driver rejected rather than the original 25-second gap.[2]

The distinction between "gap size" and "gap structure" is obviously especially important in the case of a multiple merge. It is also applicable, however, to each of the single vehicle merges illustrated in Figure 1. In each case, the definition of effective lead-lag structure reduces considerably the bias introduced into a simple static analysis by the arbitrary arrival rates of the ramp and mainstream vehicles. This bias may be reduced still further by relaxing the restriction that all measurements be made at the ramp nose. It is useful, for example, to consider the values of "dynamic" lead and lag time accepted by a merging driver as he actually starts his merging maneuver (i.e., at the point X in Figure 1). Similarly, the values of accepted or rejected gaps may be evaluated "dynamically" by measuring the headway between two successive mainstream vehicles at the instant the lead vehicle passes or overtakes a ramp vehicle. Such "dynamic" measurements approximate more closely the mechanics of the merging situation in that they are

[2] "Lead-time" is defined as the time-headway between a ramp vehicle and either the immediately preceding mainstream vehicle or the preceding ramp vehicles whichever is the smaller; "lag-time" is defined as the time-headway between a ramp vehicle and the immediately succeeding mainstream vehicle. A glossary of the terms and definitions used in this paper is given in Appendix B.

defined not with respect to an arbitrary, fixed point in space but rather with respect to the observed motions of the merging vehicles.

Depending on the flow conditions in the adjacent mainstream section, dynamic and static analyses may yield similar or considerably different results. In this context, we note with Haight (25) that ramp-freeway merging is characterized in part by the merging driver's ability to adjust his environment according to his desires. By speeding up or slowing down, the driver may adjust his position relative to that of a sequence of mainstream vehicles so as to create a suitable gap for merger. In extreme circumstances he may actually force his way into a gap, causing one or more mainstream vehicles to change speed or direction in order to accommodate him. The implications of this type of behavior have been discussed theoretically by a number of writers (4, 25, 34).

Obviously, many different factors exert considerable influence on the merging process. For example, the relative speeds of the ramp and mainstream vehicles, the spatial as well as temporal dimensions of mainstream headways, the availability of alternate acceptable gaps ahead of and behind the one under consideration, the types of vehicles involved, the characteristics of the merging drivers, the ramp and merge area geometrics, etc., all may affect a given acceptance/rejection decision to a greater or lesser degree.[3] Analysis of such factors, therefore, should desirably form a primary element in any comprehensive empirical analysis of merging behavior. Many of these factors are directly measurable on the time-distance diagram shown in Figure 1.

It should be emphasized here, however, that time-distance trace analysis has its own very serious limitations. It can provide only a description (albeit a very efficient description) of the merging process; the geometrics of a time-distance diagram can tell the analyst nothing of the instant when a driver made his decision to accept or reject a mainstream gap, nor of his perception of the "size" of that gap. Apart from the work of Hurst et al (32), relatively little empiricial (or theoretical either, for that matter) attention has been paid to this aspect of merging behavior. It represents potentially an extremely fruitful field for study.[4]

[illegible] merging maneuver. To be truly "acceptable," however, a merging maneuver should cause neither disruption of the mainstream flows nor undue discomfort to the merging driver. One may thus consider a possible redefinition of the phrase in terms of the delay or speed change imposed on a mainstream vehicle consequent upon completion of a merge or else in terms of the tension induced in a merging driver (measured, say, by GSR techniques) by a given merging situation. In each case an "acceptable gap" might be redefined as one whose acceptance caused neither more than a specified maximal perturbation in the mainstream flow nor more than a maximal tension level in the merging driver.

EMPIRICAL STUDIES

Collection and Analysis of Field Data

A pilot empirical study designed to test the efficacy of the analysis techniques discussed previously was conducted during the summer of 1965 at two freeway entrance ramps

[3]Haight et al (25) have shown the importance of the spatial dimension under conditions of low relative merging velocities, while Knox (38) and Vardon (55) have both analyzed the effects of relative and absolute speeds on merging behavior. Of particular interest to this discussion is a recent analysis reported by Hurst et al (32) in which comparisons are made between alternate indices of acceptable gap size. The results of this latter study suggest that complex indices which combine estimates of relative merging speed with conventional temporal measurements provide a more efficient and more stable basis for predicting gap acceptance/rejection behavior than do simple time-gap measurements alone.

[4]One interesting hypothesis worthy of research is that a merging driver evaluates the "size" of a mainstream gap in terms of the rate of change of the angle subtended at his eye by an approaching mainstream vehicle.

TABLE 1

DESIGN CHARACTERISTICS OF STUDY LOCATIONS

Description	Harlem Avenue Left-Hand On-Ramp Eisenhower Expressway (Eastbound)	Nagle Avenue Right-Hand On-Ramp Kennedy Expressway (Northbound)
(a) Geometric Characteristics		
Mainstream section	Depressed 6-lane Urban Freeway; Level, Tangent Alignment	Depressed 6-lane Urban Freeway; Level, Tangent Alignment
Acceleration lane	Parallel Type Design, 1075 ft long	Parallel Type Design, 800' long
Ramp width at nose	Single Lane Entry, 16 ft wide	Single Lane Entry, 16 ft wide
Ramp grade	-3 percent	-3 percent
(b) Traffic Characteristics		
Ramp ADT	10,700 veh/day	5,600 veh/day
Mainstream ADT (one direction)	64,000 veh/day	55,000 veh/day
Avg. ramp flow during study	575 veh/hr (16 percent trucks)	310 veh/hr (10 percent trucks)
Avg. flow in adjacent mainstream lane during study	1,260 veh/hr	875 veh/hr

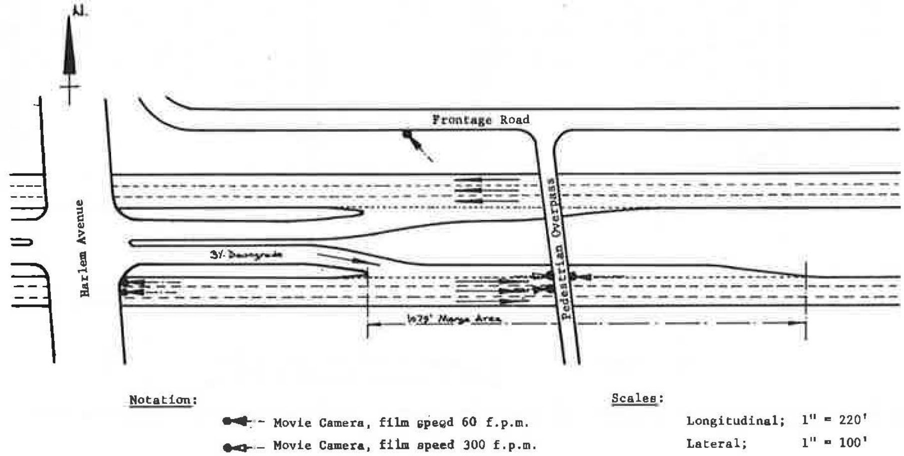


Figure 3. Harlem Ave. (eastbound) left-hand entrance ramp, Eisenhower Expressway, Chicago.

in Chicago. One of the ramps selected was located on the left-hand side of the through pavement, the other on the right. In all other respects the two ramps had similar design and traffic characteristics (see Figs. 3 and 4 and Table 1).

Field data were collected by means of synchronized, time-lapse ground photography, using a system of six time-lapse movie cameras at each ramp. The systems of cameras were synchronized by stopwatch timing, supplemented by timed runs through the study section in a marked vehicle. Figures 3 and 4 indicate the approximate positions and operating speeds of the cameras. Approximately two hours of continuous data were collected at each location, during periods of medium to high flow in the mainstream lanes.

The film data were reduced to yield information on the time-distance trajectories of all ramp and adjacent lane mainstream vehicles passing through the merge areas of the two ramps. Time measurements were made to an accuracy of $\pm \frac{1}{5}$ second and distance measurements to ± 5 feet. At least 10 time-distance observations were made for each individual vehicle. These time-distance data, reduced to digital form for

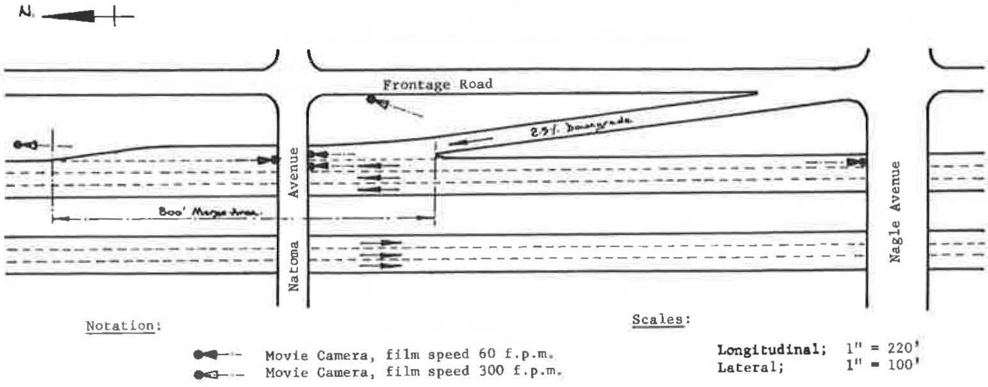


Figure 4. Nagle Ave. (northbound) right-hand entrance ramp, Kennedy Expressway, Chicago.

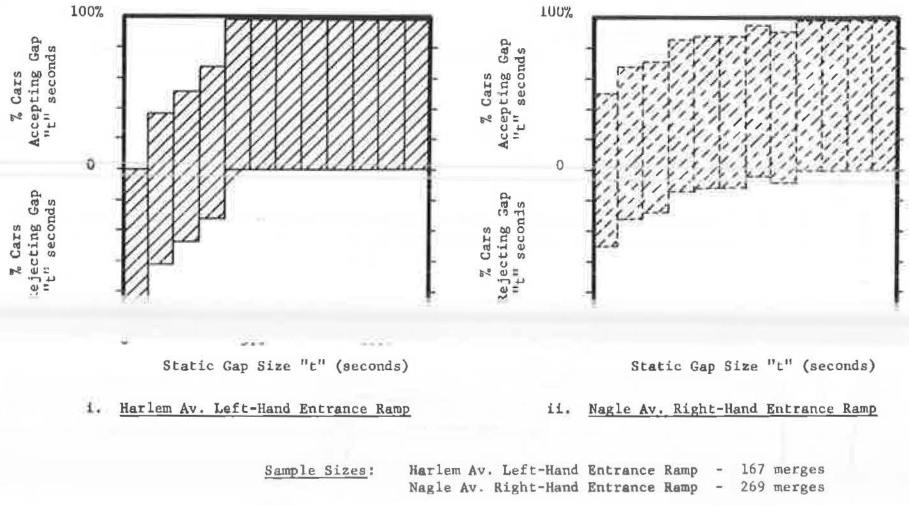


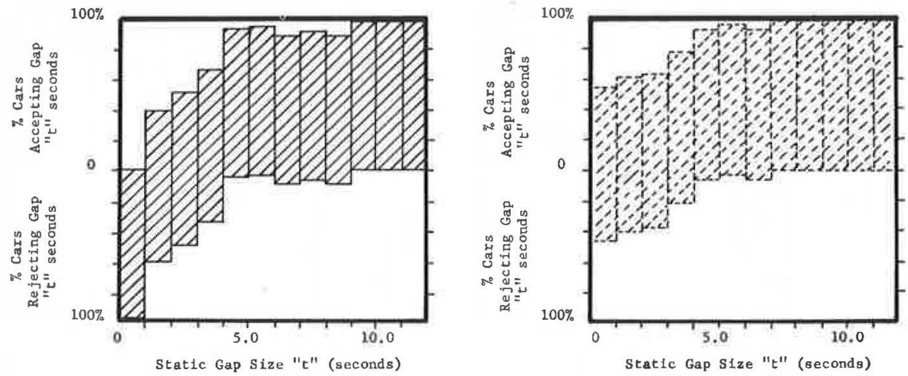
Figure 5. Acceptance and rejection of static time-gaps, single merges only, passenger cars only.

computer analysis, provided the framework for the analytical work discussed in this paper.

Structured Static Analysis

An initial static analysis indicated that there was no significant difference between the behavior of "isolated" ramp vehicles, separated by at least 5 seconds from the nearest preceding or succeeding ramp vehicle, and the behavior of "lead" vehicles in multiple-merges, separated by at least 5 seconds from the preceding ramp vehicle but by less than 5 seconds from the succeeding ramp vehicle.[5] To provide a larger sample for analysis, therefore, these two categories were combined together under the general heading "single-vehicle merges." Trailing vehicles in multiple merges (i.e., the second, third and subsequent vehicles in each platoon of ramp vehicles[5])

[5] A "platoon" of ramp vehicles was defined, arbitrarily, as a succession or group of ramp vehicles of any number greater than two, having a maximum within-group headway of 5 seconds. The efficiency of this definition is discussed in a later section.

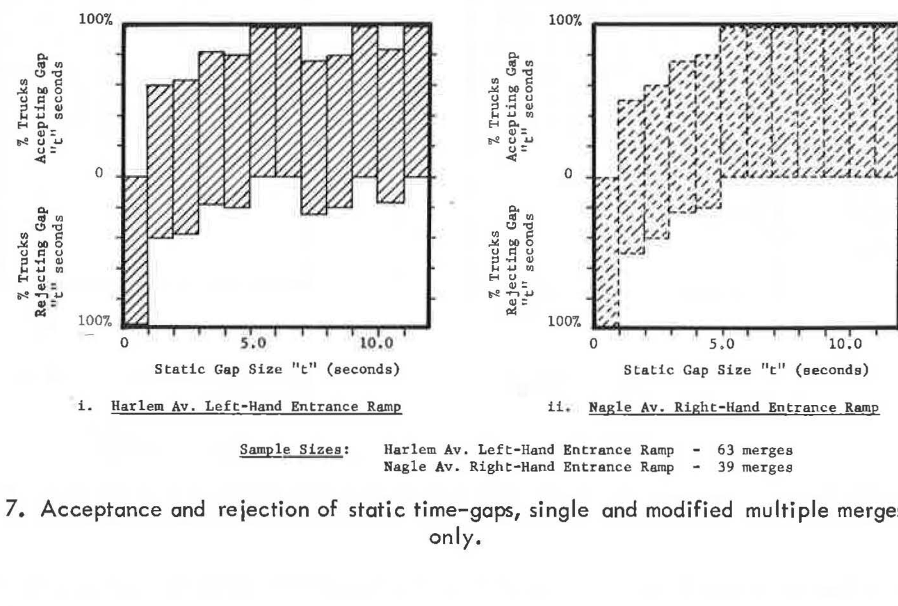


i. Harlem Av. Left-Hand Entrance Ramp

ii. Nagle Av. Right-Hand Entrance Ramp

Sample Sizes: Harlem Av. Left-Hand Entrance Ramp - 399 merges
Nagle Av. Right-Hand Entrance Ramp - 395 merges

Figure 6. Acceptance and rejection of static time-gaps, single and modified multiple merges, passenger cars only.



i. Harlem Av. Left-Hand Entrance Ramp

ii. Nagle Av. Right-Hand Entrance Ramp

Sample Sizes: Harlem Av. Left-Hand Entrance Ramp - 63 merges
Nagle Av. Right-Hand Entrance Ramp - 39 merges

Figure 7. Acceptance and rejection of static time-gaps, single and modified multiple merges, trucks only.

were treated as a separate data sample, for which a "modified-multiple-merge-gap" was defined equal to the sum of the effective lead and lag values defined previously.

Figure 5 illustrates the simple "static" gap-acceptance/rejection distributions determined for the two ramps for single-vehicle merges, passenger cars only. Figures 6 and 7 illustrate the equivalent distributions for the combined samples of single-vehicle and modified-multiple merges involving passenger cars and trucks respectively.

Comparison of Figures 5 and 6 suggests that there is little difference between the results derived for the two different data sets when they are analyzed in purely static terms.[6] In each case it is apparent that fewer small gaps (< 4 seconds) are accepted at the left-hand than at the right-hand ramp. Also, on the basis of Figure 7 it appears that trucks tend to require a slightly larger gap for acceptance than do cars. (This

[6] This point is discussed further in a later section.

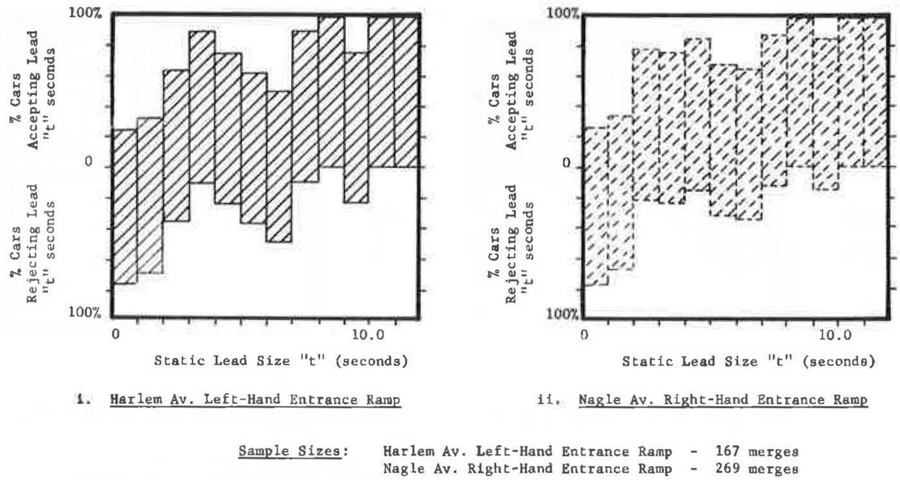


Figure 8. Acceptance and rejection of static lead values, single merges only, passenger cars only.

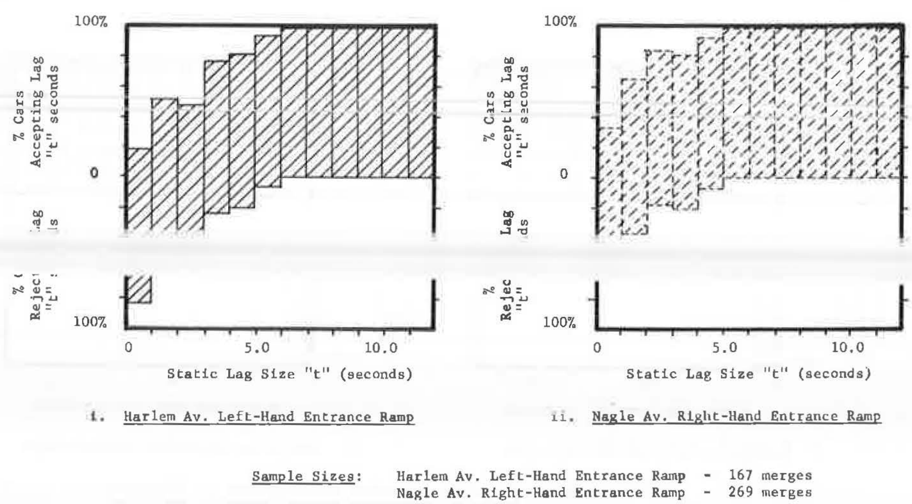
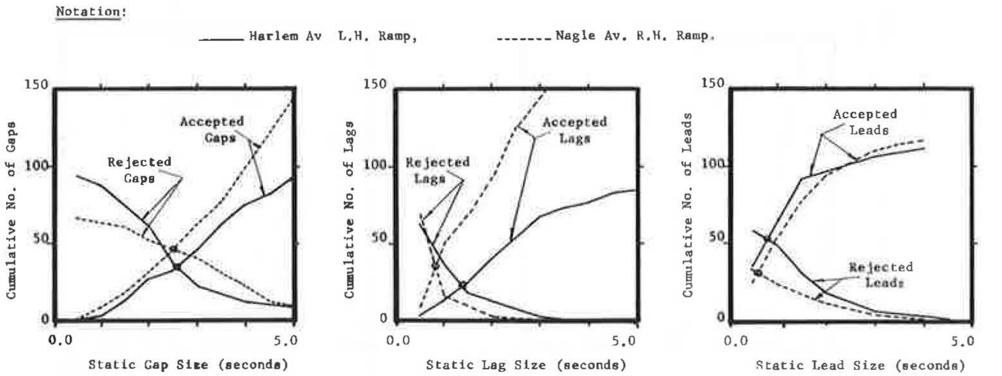


Figure 9. Acceptance and rejection of static lag values, single merges only, passenger cars only.

last conclusion should be interpreted with extreme caution owing to the small sample size.)

Figures 8 and 9 illustrate the static acceptance/rejection distributions for "lead" and "lag" values, again based on single-merge data only. The lead distributions at both ramps are almost rectangular, suggesting a roughly equal probability of acceptance for all values above a minimum of 1 second. The lag distributions, however, exhibit a more exponential form, indicating that the likelihood of acceptance increases as the value of static lag increases. As in the case of the gap size analyses discussed above, there is an evident tendency for small lags to be accepted more readily at the right-hand than at the left-hand ramp.

In any binary response situation, such as that represented by a ramp vehicle's acceptance or rejection of mainstream headways, a number of alternate indexes of "critical response level" may be defined. In the following discussion it is assumed that the "median-acceptance-value" (i.e., that value of lead, lag or gap which has a



i. Estimate of Critical Gap Size — ii. Estimate of Critical Lag Size — iii. Estimate of Critical Lead Size

Critical Acceptance Values:

L.H. Ramp Gap Size = 2.60 sec.	L.H. Ramp Lag Size = 1.38 sec.	L.H. Ramp Lead Size = 0.80 sec.
R.H. Ramp Gap Size = 2.45 sec.	R.H. Ramp Lag Size = 0.80 sec.	R.H. Ramp Lead Size = 0.60 sec.

Figure 10. Raff analysis: Estimation of static gap, lag and lead acceptance values, single merges only, passenger cars only.

TABLE 2

CRITICAL GAP, LEAD AND LAG ACCEPTANCE VALUES, SINGLE VEHICLE MERGES ONLY

Method	Subject	Harlem Avenue Left-Hand On-Ramp	Nagle Avenue Right-Hand On-Ramp
Raff Analysis	Critical Gap	2. 60 sec.	2. 45 sec.
Probit Analysis	Critical Gap 95% Conf. Lim.	2. 63 sec. 2. 28-3. 03 sec.	1. 88 sec. 1. 22-2. 62 sec.
'f Analysis	Critical Lag	1. 38 sec.	0. 88 sec.
[illegible] Analysis	Critical Lag 95% Conf. Lim.	1. 11 sec. 0. 86-1. 33 sec.	0. 71 sec. 0. 47-1. 07 sec.
Raff Analysis	Critical Lead	0. 80 sec.	0. 60 sec.

50 percent probability of acceptance) may be used as a simple, parametric index of critical acceptance behavior.[7]

Two separate empirical techniques were employed to estimate this assumed critical value: a graphical approach due to Raff, and a standard probit analysis.[8] Raff's method

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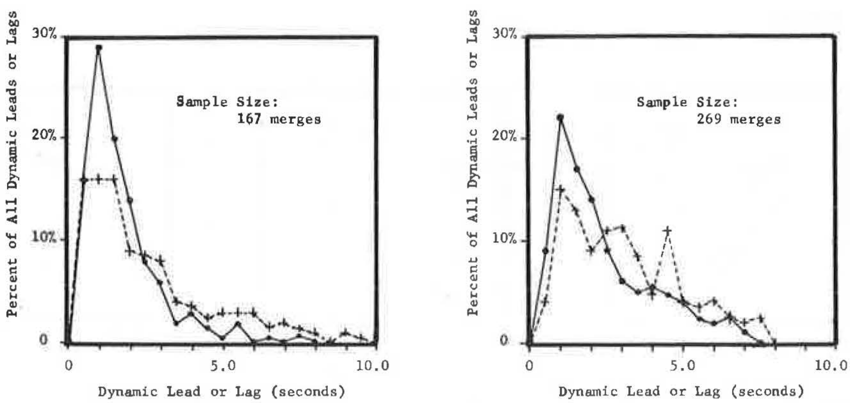
[7]Obviously, the 50 percentile acceptance value implies an equal likelihood of acceptance or rejection. The arbitrary selection of this value may well be criticized on the grounds that a higher probability of acceptance than 50 percent is desirable. More importantly, however, the critical values determined here are general values for the ramps in question, aggregated over all drivers. A more realistic analysis might either develop a critical acceptance function for individual drivers or for statistically significant groupings (in the sense of a maximized ratio of between-group variance to within-group variance) of drivers. Alternatively, the simple analysis discussed here might be stratified according to driver or vehicle characteristics, speed levels, etc. The development of a more realistic empirical acceptance/rejection function is discussed in detail in the final section of this paper.

[8]Raff's method was first developed in: Raff, M. S., and Hart, J. W., A Volume Warrant for Urban Stop Signs, Eno Foundation for Highway Traffic Control, 1950. A simple discussion of probit analysis is given in: Experimental Statistics, Handbook No. 91. U. S. Department of Commerce, National Bureau of Standards, U. S. Government Printing Office, August 1963.



i. Harlem Av. Left-Hand Entrance Ramp   ii. Nagle Av. Right-Hand Entrance Ramp

Notation:

●—● Dynamic Lead Value Accepted at "X" Point of Merging Maneuver

+ – + Dynamic Lag Value Accepted at "X" Point of Merging Maneuver

Figure 11. Distributions of dynamic lead and dynamic lag times at the commencement of the merging maneuver (X point), passenger cars only, single merges only.

involves the plotting of two cumulative curves for each set of data, one indicating the number of accepted gaps (leads, lags) smaller than a set of given values, the other the number of rejected gaps (leads, lags) greater than these values. The intersection of the two cumulative curves then provides an estimate of the median acceptance value as defined above. Probit analysis involves a slightly more sophisticated analytical

objects, $a_1, a_2, \ldots, a_k$, were subjected to a set of K stimuli $X_1, X_2, \ldots, X_k$, yielding K separate binary responses, $r_1, r_2, \ldots, r_k$. In the case of gap acceptance data, the set of stimuli are the available mainstream gaps, leads or lags; the objects are the merging vehicles and the set of binary responses the merging drivers' acceptances or rejections of the given gaps (leads, lags). The value of median critical response is then estimated simply by assuming that the distribution of response levels follows a normal distribution. In the case of gap and lag observations a close approximation to normality may be achieved by transforming the time scale logarithmically; lead values, however, cannot be so transformed. Probit analyses therefore were performed only on the samples of gap and lag observations.

Figure 10 and Table 2 summarize the results of probit and Raff analyses applied to the samples of single-vehicle merges, passenger cars only. Note that in each case the critical acceptance values for the left-hand ramp are higher than the equivalent right-hand ramp values. Note also that the critical lead values obtained from the Raff analysis are lower at both ramps than the equivalent critical lag values. Probit analysis yields estimates of both the critical median acceptance value and the critical response variance. The values of the latter (see Table 2) are virtually the same in all four cases, suggesting that though the level of response varies vetween a left- and right-hand ramp and between gap and lag values, the nature of the response mechanism is essentially the same.

Both analyses yield consistently higher values for critical acceptable gap size than for critical lag size. The sum of critical lead and critical lag values, however, exceeds the critical gap value at both study locations, suggesting that though a driver may accept critical values of either lead or lag time, he is unlikely to accept both simultaneously. Finally, it is relevant to note that the estimates of median acceptance value derived from the graphical analyses are in all cases contained within the 95 percent confidence limits for the equivalent probit analysis estimates.

A Pseudodynamic Analysis

The preceding discussion has been based entirely on "static" measurements made at the ramp nose. Figure 11 in contrast, illustrates the distributions of accepted lead and lag values measured "dynamically" at the start of the merging maneuver (i.e., at the point X in Figure 1). Again the analysis is based on data for single-vehicle, passenger car merges only.

The marked skewness and clearly defined modal values of the lead distributions illustrated in Figure 11 suggest that one may interpret these values as approximate estimates of the minimum dynamic lead time required by a merging driver.[9] Equivalent interpretation of the modal lag values is clearly not valid.

Figure 12 illustrates the set of cumulative frequency distributions derived from the curves in Figure 11, determined in this case for both dynamic and static observations. In the case of the accepted lag curves, the distributions of static and dynamic observations are significantly different from each other (at the 5 percent level) for each of the two ramps. The equivalent pairs of lead distributions, however, are not significantly different.

The difference between the sets of lag distributions suggests that the static lag values (which are largely dependent on the arbitrary arrival rates of the ramp vehicles) cannot be used to predict the size of acceptable dynamic lags at the actual point of merger. This point is perhaps emphasized more clearly by Figure 13. Note in this figure that the standard error of estimate of "Y" (dynamic lag) on "X" (static lag) is 1.68 seconds, indicating a considerable degree of instability between individual static and dynamic accepted-lag measurements. Note also that although the general tendency is for lag values at the "X" point to be smaller than those measured at the ramp nose, this tendency is reversed at extremely low values.

All of the data in Figure 13 were collected during periods of free flow on the mainstream section, when mainstream speeds exceeded ramp vehicle speeds. In a more congested situation where the reverse speed condition obtains, a totally different relation might apply, with lead rather than lag acceptance critical. It should also be noted that the poor correlation between static and dynamic lag values does not necessarily imply a serious instability of mainstream gap size over the length of the merge area. Rather, it implies an adjustment of gap structure (i.e., lead and lag values) which is suppressed in the case of the lead measurements by definitional bias.

In all cases accepted lead and lag values, both static and dynamic, tend to be highe at the left-hand than at the right-hand ramp. In considering this result, it should be borne in mind that an average of 52 lane changes per hour was observed at the righ'-hand ramp in which vehicles moved from the right-hand lane into the center lane. At the left-hand ramp, the equivalent figure was 91 lane changes per hour from the left lane into the center lane. This lane changing was frequently associated with the acceptance by a merging vehicle of a relatively small static lag value.

Speed and Volume Effects, Merge Length Analysis

A crude estimate of the influence of merging speed on gap acceptance behavior may be obtained simply by stratifying the data of Figure 12 according to a number of preselected relative speed groupings.[10] The results of such an analysis are illustrated

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[9]This in turn suggests the existence of a minimal or "buffer" lead-time between two merging vehicles, which may take on slightly different values depending on the characteristics of the drivers. Such a buffer is implicit in the familiar car-following models of stream flow.

[10]The analysis is crude in the sense that it incorporates only the coarsest levels of stratification with no regard to the absolute speeds of the vehicles involved. It is also based, necessarily, on observations of acceptance and rejection behavior which lie within rather than on the critical speed/acceptance envelope. A more meaningful approach, precluded in this case due to lack of data, might have been to perform critical response level analyses for each of a set of ramp/mainstream speed groupings.

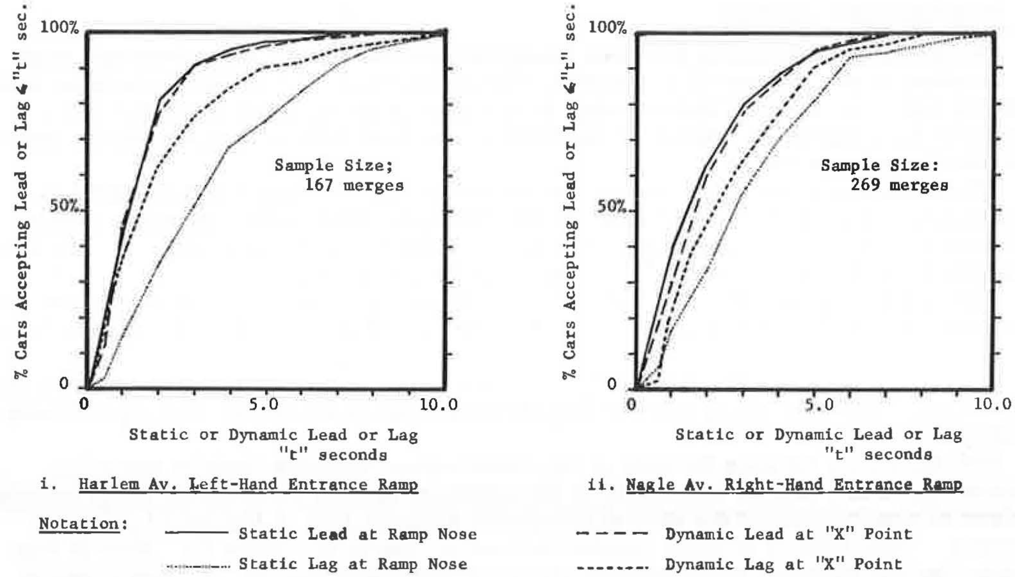
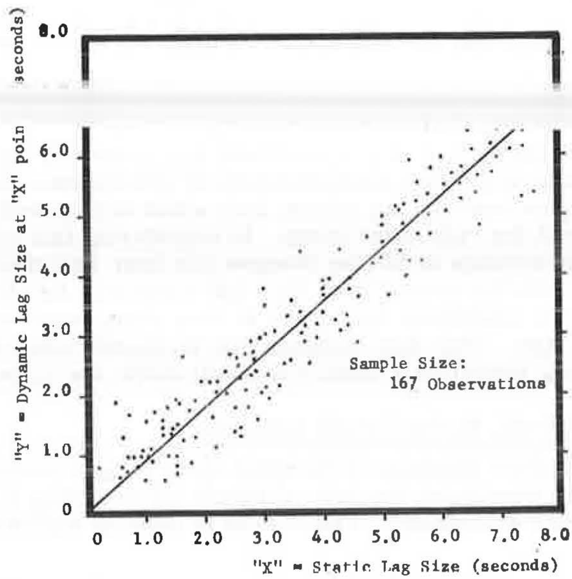


Figure 12. Cumulative distributions of static and dynamic lead and lag values, single merges only, passenger cars only.



Regression Equation: Y = 0.056 + 0.873 X

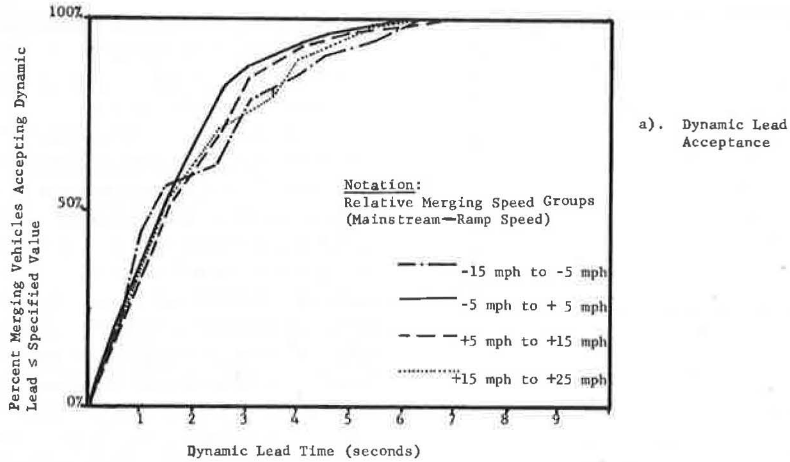
(R^2 = 0.88, Std. Error of Est. = 1.68 sec.)

Where: X = Value of Static Lag Accepted by merging vehicle, measured at the ramp nose

Y = Value of Dynamic Lag Accepted by the same merging vehicle, measured at the "X" point

All data for Nagle Av. Right-Hand Entrance Ramp, Kennedy Expressway, Chicago

Figure 13. Stability of static vs dynamic lag acceptance, single merges only, passenger cars only.



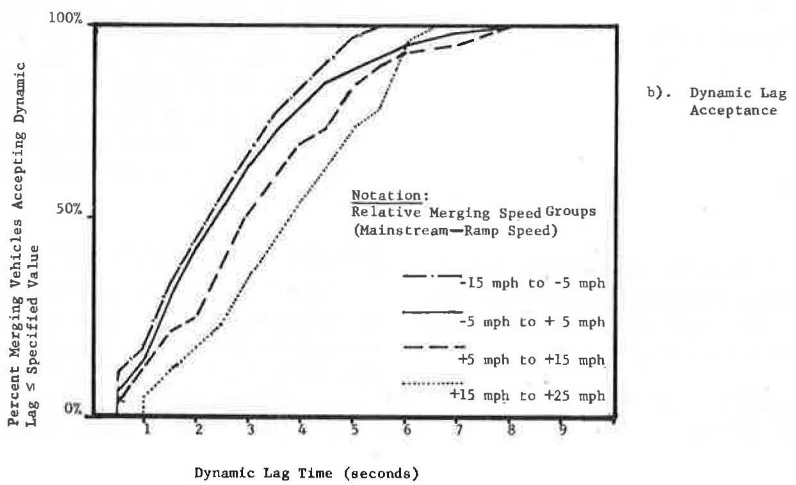
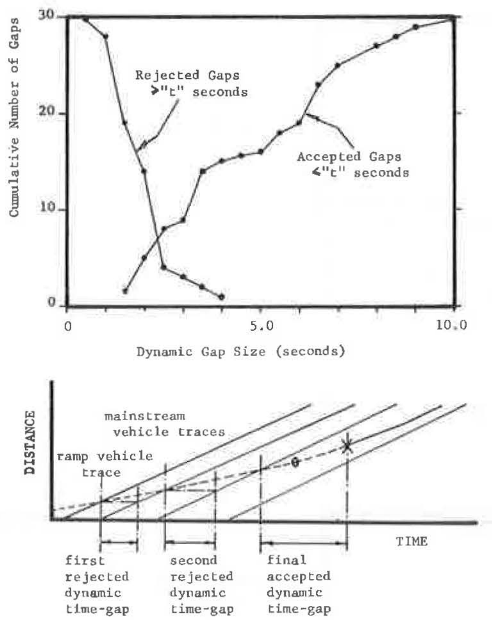


Figure 14. Effect of relative merging speed on dynamic lead and lag acceptance (all data for right-hand ramp only).

graphically in Figure 14. It is immediately apparent that relative merging speed exerts a considerable influence on the distribution of accepted dynamic lag values, the proportion of vehicles accepting small lag values decreasing rapidly as the relative speed of the mainstream to the ramp vehicle increases. The equivalent effect in the case of dynamic lead values is much less pronounced. In neither case does allowance for mainstream speed level, again by 10 mph increments, exert any significant influence on the results. This may well, however, be due to the extremely small sample size rather than to any basic speed/acceptance relationship.

Table 3 illustrates the relatively insignificant influence of mainstream volume, expressed in this case as a minute flow rate in the adjacent lane, on merging behavior.[11] Note that merge length and point of merger are virtually independent both of each other and also of relative merging speed. The slightly higher average merge length at the left-hand ramp may be a reflection of a longer acceleration lane. The higher standard deviation of merge lengths at that ramp, however, suggests a basic difference in merging behavior, due possibly to driver hesitancy at the left-hand terminal or to the

[11] An equivalent stratification of the curves in Figure 12 according to three levels of mainstream flow also indicates that volume exerts no significant influence on the merging process.



Note: Dynamic Gap Size defined as the time-gap between a ramp vehicle and the next successive mainstream vehicle, measured at the instant the ramp vehicle is overtaken by a preceding mainstream vehicle.

Figure 15. Raft analysis: Acceptance and rejection of dynamic time-gaps, single merges only, passenger cars only (all data for Harlem Ave.

tendency noted in a previous paper (61) of some drivers to travel to the end of a left-hand ramp acceleration lane before completing their merge.

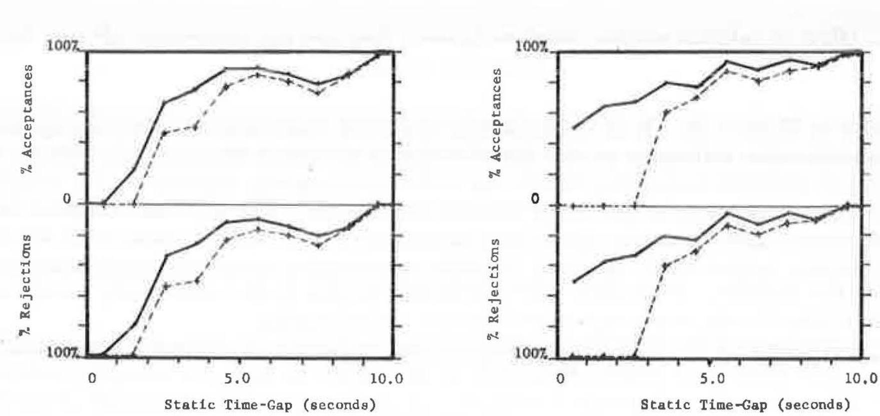
By definition, it is not possible to consider dynamic rejection behavior with reference to measurements made at the X and O points. If, however, one redefines a dynamic time-gap as suggested earlier as the time-headway between a ramp vehicle and the next, succeeding mainstream vehicle measured at the instant when the ramp vehicle is overtaken by a leading mainstream vehicle (Fig. 15), then a modified dynamic form of acceptance/rejection analysis is possible. Figure 15 illustrates a Raff analysis of a sample of 55 dynamic gap acceptance and rejection observations at the Harlem Avenue left-hand ramp. Based on this extremely small sample of data it appears that the critical dynamic acceptance value at that ramp is approximately 2. 25 seconds, compared with an equivalent static value of 2. 60 seconds.

Analysis of Multiple Vehicle Merges

Approximately 33 percent of all merges observed at the right-hand ramp and 57 per-

category of multiple-vehicle merges.

Figure 16 depicts a straightforward static analysis of these data, based on the technique developed by Pearson and Ferreri (49). Note that, apart from a marked tendency for



i. Harlem Av. Left-Hand Entrance Ramp

ii. Nagle Av. Right-Hand Entrance Ramp

Notation:

Single Vehicle Merges, i.e. one ramp vehicle only entering the mainstream

Two Vehicle Merges, i.e. two ramp vehicles accepting a single mainstream gap

Figure 16. Multiple merge analysis: Acceptance and rejection of static time-gaps by multiple vehicles.

TABLE 3

CORRELATION OF MERGE LENGTH AND POINT

Category	Values of Simple Correlation Coefficient	
	R. H. Ramp	L. H. Ramp
Merge length vs relative merging speed	0. 01	0. 08
Merge length vs through volume	0. 10	0. 09
Merge length vs point of merge	0. 03	0. 06
Point of merge vs through volume	0. 06	-0. 10
Point of merge vs relative merging speed	-0. 08	-0. 02
Average merge length	138 ft	179 ft
Standard deviation of merge length	59 ft	115 ft

single vehicles to accept smaller time-gaps than pairs of successive vehicles, there is relatively little pattern in the data.

Figures 17 and 18 illustrate a simple dynamic analysis of multiple vehicle merges for the Harlem Avenue left-hand ramp. Insufficient data were available to duplicate this analysis for the right-hand ramp. The broken lines in Figure 17 represent the distributions of modified-static leads accepted by the first, second, third and fourth vehicles respectively. In this case, all of the lead values were measured statically at the ramp nose. For all but the first vehicle in a platoon, these lead values represent also the headways between successive ramp vehicles as they pass the ramp nose. Figure 18, in contrast, compares the acceptance of dynamic lead values in single-vehicle merges with that for the trailing vehicles in multiple merges. All measurements in this case were made at the X point.

The distributions of both static and dynamic values accepted by the lead vehicle in a multiple merge (Figs. 17 and 18) are significantly lower than the equivalent acceptance distributions for successive trailing vehicles. There is no significant difference, however, among the acceptance distributions for the second, third and fourth vehicles in line in a multiple merge. These results have certain implications in terms of the comparative efficiency of isolated and multiple vehicle merges. In particular, if one defines "merge-efficiency" as the average time required per vehicle to merge into the mainstream, either singly or in a platoon, and one accepts the values for critical lead

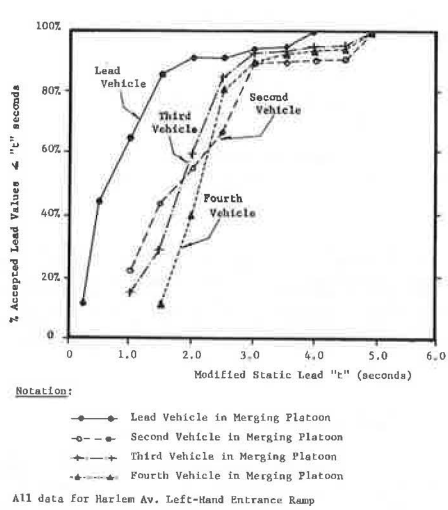


Figure 17. Multiple merge analysis: Cumulative distributions of accepted lead values by successive vehicles in a merging platoon.

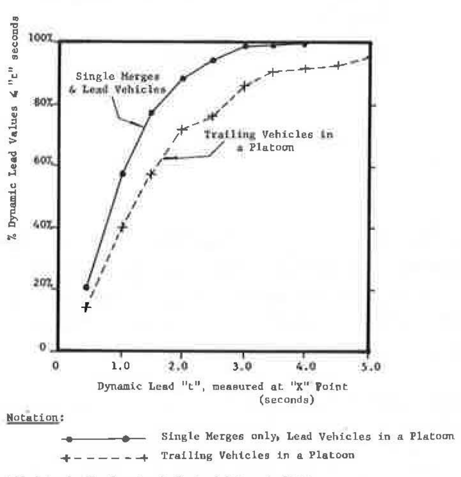


Figure 18. Multiple merge analysis: Cumulative distributions of dynamic lead values for single merges and trailing vehicles in a multiple merge.

TABLE 4

INPUT VARIABLES FOR PRINCIPAL COMPONENTS ANALYSES

Analysis Groups I & II	Analysis Groups III & IV
Group I—Nagle Ave., Direct Acceptance Group II—Harlem Ave., Direct Acceptance	Group III—Nagle Ave., Rejections Group IV—Harlem Ave., Rejections
1. Vehicle type of leading vehicle	1. Vehicle type of leading vehicle
2. Vehicle type of merging vehicle	2. Vehicle type of merging vehicle
3. Vehicle type of lagging vehicle	3. Vehicle type of lagging vehicle
4. Mainstream 1-minute volume	4. Ramp 1-minute volume
5. Ramp 1-minute volume	5. Ramp 1-minute volume
6. Accepted gap at nose	6. Accepted gap at nose
7. Accepted lead at nose	7. Absolute speed of merging vehicle at nose
8. Accepted lag at nose	8. Relative speed of merging vehicle to leading vehicle at nose
9. Absolute speed of merging vehicle at nose	9. Relative speed of merging vehicle to lagging vehicle at nose
10. Relative speed of merging vehicle to leading vehicle at nose	10. Number of rejected gaps at nose
11. Relative speed of merging vehicle to lagging vehicle at nose	11. Rejected gap at nose
12. Second previous mainstream gap	12. Rejected lead at nose
13. First previous mainstream gap	13. Rejected lag at nose
14. First following mainstream gap (excluding accepted gap)	14. Second previous mainstream gap
15. Second following mainstream gap	15. First previous mainstream gap
16. Third following mainstream gap	16. First following mainstream gap (excluding rejected gap)
[illegible]	[illegible]
18. Accepted lead at X-point	18. Third following mainstream gap
19. Accepted lag at X-point	19. Accepted gap at X-point
20. Absolute speed of merging vehicle at X-point	20. Accepted lead at X-point
21. Distance nose to X-point	21. Accepted lag at X-point
	22. Absolute speed of merging vehicle at X-point
	23. Distance nose to X-point

and lag time derived earlier, then it appears that a multiple merge is slightly, but only slightly, more efficient than an equivalent series of isolated merges. This conclusion, of course, takes no account of the disruptive effect of a sequence of multiple merges on traffic flow in the mainstream, a factor of considerable importance in comparing the two types of merging.

A Multivariate Approach to the Analysis of Merging Behavior

If nothing else the preceding discussion has at least emphasized the essential multivariate character of merging behavior. A broad range of variables has been defined, all of which may exert a potentially significant influence on the merging process. Some of these variables may be determined from a simple time-distance trace analysis, others may not.

In an attempt to reduce the dimensionality of the analytical problem and at the same time to identify some of the basic collinearities within the data, a varimax rotated

TABLE 5

CORRELATION MATRIX, MEANS AND STANDARD DEVIATIONS FOR SINGLE MERGES WITH DIRECT ACCEPTANCE
NAGLE AVENUE, RIGHT-HAND ON-RAMP, KENNEDY EXPRESSWAY (NORTHBOUND)

Variable		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	Mean	Std.Dev.
VEH TYPE/LDG.	1	1.0	.00	.12	.05	-.01	-.07	-.04	-.07	-.02	-.02	-.08	-.06	.09	.09	.06	-.11	-.06	-.02	-.06	-.04	-.05	0.20	0.40
VEH TYPE/MRG.	2		1.0	.04	-.16	.06	.06	-.02	.08	-.13	.09	.08	.01	.04	.03	.00	.17	.06	.06	.04	.15	.03	0.13	0.34
VEH TYPE/LAG.	3			1.0	.00	.09	-.08	-.06	-.07	.04	-.07	-.07	.01	-.05	-.01	-.11	-.03	-.07	-.07	-.05	.06	.00	0.20	0.40
THRU. VOL.	4				1.0	.12	-.40	-.22	-.34	.12	-.11	-.18	-.14	-.29	-.44	-.25	-.32	-.40	-.25	-.33	.05	.06	14.7	3.5
RAMP VOL.	5					1.0	-.02	-.10	.04	-.12	.03	-.04	-.02	.00	-.03	-.19	-.03	-.05	-.09	.00	-.16	.12	6.1	2.3
ACC. GAP NOSE	6						1.0	.59	.83	-.16	.31	.14	.00	.13	.12	-.08	.13	.98	.61	.82	-.07	-.24	6.69	4.53
ACC. LEAD NOSE	7							1.0	.04	-.06	.26	.11	.00	.18	-.04	-.07	.09	.59	.94	.07	.04	-.14	2.42	2.56
ACC. LAG. NOSE	8								1.0	-.15	.21	.09	.00	.05	.18	-.06	.10	.81	.10	.96	-.13	.21	4.32	3.63
ABS. SP. NOSE	9									1.0	-.64	-.62	-.01	-.12	.07	.07	.01	-.16	-.13	-.11	.42	.05	72.3	13.4
REL. SP. LDG NOSE	10										1.0	.50	.04	.25	-.06	-.05	-.03	.32	.30	.19	-.23	-.05	11.8	16.1
REL. SP. LAG NOSE	11											1.0	.01	.13	.01	.00	.16	.12	.14	.05	-.20	-.08	8.3	16.9
PREC. GAP #2	12												1.0	.12	-.08	-.07	-.09	.02	-.01	.02	-.02	-.04	4.11	3.28
PREC. GAP #1	13													1.0	.07	-.09	.01	.15	.17	.06	.01	.00	4.40	3.71
SUCC. GAP #1	14														1.0	.11	-.06	.12	.01	.15	-.04	-.10	4.58	3.98
SUCC. GAP #2	15															1.0	.08	-.10	-.09	-.06	.03	-.07	4.09	3.63
SUCC. GAP #3	16																1.0	.14	0.9	.12	-.01	-.06	3.95	3.30
ACC. GAP X-PT.	17																	1.0	.63	.82	-.07	-.24	6.87	4.53
ACC. LEAD X-PT.	18																		1.0	.07	-.08	-.10	2.91	2.59
ACC. LAG. X-PT.	19																			1.0	-.03	-.23	3.97	3.53
ABS. SP. X-PT.	20																				1.0	.37	74.1	14.9
DIST. N - X	21																					1.0	261.	106.

T/ 6

CORRELATION MATRIX, MEANS AND STANDARD DEVI ONS FOR SINGLE MERGES WITH DIRECT ACCEPTANCE
HARLEM AVENUE, LEFT-HAND ON-RAM ISENHOWER EXPRESSWAY (EASTBOUND)

Variable		1	2	3	4	5	6	7	8	9	10	12	13	14	15	16	17	18	19	20	21	Mean	Std.Dev.
VEH TYPE/LDG.	1	1.0	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	0.00	0.00
VEH TYPE/MRG.	2		1.0	-.05	-.08	.01	.18	.13	.15	.21	.04	-.03	.08	.05	.00	-.12	.35	.27	.28	-.24	.05	0.19	0.39
VEH TYPE/LAG.	3			1.0	-.12	.04	.07	-.08	.11	-.03	.06	.01	-.03	.13	.16	.07	.09	-.05	.14	-.12	-.06	0.01	.09
THRU. VOL.	4				1.0	-.26	-.21	-.14	-.18	-.06	.03	-.19	-.22	-.29	-.33	-.15	-.17	-.11	-.14	-.10	-.10	18.3	4.1
RAMP VOL.	5					1.0	.09	-.10	.15	-.13	.04	.04	.32	.08	-.05	.07	-.11	-.12	-.07	-.22	.01	12.1	4.2
ACC. GAP NOSE	6						1.0	.48	.92	-.13	-.04	.02	.02	-.03	-.10	-.07	.69	.51	.56	-.17	.05	6.60	4.90
ACC. LEAD NOSE	7							1.0	.09	.07	-.08	-.06	-.02	.08	.02	.04	.57	.91	.19	.06	.09	1.51	1.93
ACC. LAG. NOSE	8								1.0	-.18	-.01	.04	.03	-.07	-.12	-.10	.52	.17	.55	-.22	.01	5.09	4.33
ABS. SP. NOSE	9									1.0	-.48	.07	-.10	-.09	.12	-.01	-.16	-.12	-.13	.58	.12	51.9	7.30
REL. SP. LDG NOSE	10										1.0	-.13	.07	.16	-.03	.00	.00	.00	.00	-.19	.01	23.1	12.0
REL. SP. LAG NOSE	11											.13	.05	.21	.01	.02	.05	.02	.05	-.27	-.02	23.7	12.5
PREC. GAP #2	12											1.0	.10	-.09	-.11	.05	.05	-.08	.11	.12	.02	3.10	3.31
PREC. GAP #1	13												1.0	-.01	.01	.05	.00	.02	0.02	-.07	.01	3.71	3.68
SUCC. GAP #1	14													1.0	.06	-.18	.18	.06	.19	-.04	.11	3.93	4.43
SUCC. GAP #2	15														1.0	-.09	-.04	.04	-.08	.16	-.10	3.96	4.69
SUCC. GAP #3	16															1.0	-.01	.05	-.05	-.09	-.10	3.39	2.93
ACC. GAP X-PT.	17																1.0	.60	.89	-.20	-.13	5.99	4.73
ACC. LEAD X-PT.	18																	1.0	.18	-.14	.10	2.42	2.17
ACC. LAG. X-PT.	19																		1.0	-.18	-.22	3.57	3.83
ABS. SP. X-PT.	20																			1.0	.19	61.8	9.70
DIST. N - X	21																				1.0	309.	167.

TABLE 7

CORRELATION MATRIX, MEANS AND STANDARD DEVIATIONS FOR SINGLE MERGES WITH REJECTIONS
NAGLE AVENUE, RIGHT-HAND ON-RAMP, KENNEDY EXPRESSWAY (NORTHBOUND)

Variable		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	Means	Std Dev.
VEH TYPE/LDG.	1	1.0	-.27	.21	.11	.02	-.04	.08	-.21	-.01	-.19	.26	.28	-.14	.23	.32	.02	-.10	-.06	-.04	-.29	.00	.38	-.17	0.24	0.44
VEH TYPE/MRG.	2		1.0	-.17	-.05	.07	-.01	.21	-.04	.04	-.16	.10	.11	-.10	.23	-.07	.08	-.18	-.16	.01	.33	-.04	-.22	-06	0.19	0.40
VEH TYPE/LAG.	3			1.0	-.01	.03	-.16	-.01	-.16	-.03	.10	-.03	.02	-.22	-.01	.27	-.11	-.18	-.19	-.16	.04	-.17	.23	.10	0.11	0.31
THRU VOL.	4				1.0	.23	-.41	-.09	-.04	.01	.09	-.25	-.23	-.08	-.05	-.16	-.25	-.51	-.38	-.47	-.41	-.43	-.12	-.14	15.6	3.9
RAMP VOL.	5					1.0	.07	.03	-.05	-.09	.00	.03	-.03	.29	-.17	.03	-.12	-.07	.07	.00	-.28	.04	-.02	-.12	6.8	2.6
ACC. GAP (NOSE)	6						1.0	-.05	.03	.02	.05	.00	-.08	.37	.17	.16	.74	.36	.18	.97	.34	.97	.31	.01	4.61	3.78
ABS. SPEED	7							1.0	-.68	-.65	-.30	.09	.15	-.32	.34	-.19	.08	.06	.13	.00	.02	-.01	.22	.25	64.4	9.0
REL. SP. LDG.	8								1.0	.53	.13	-.06	-.09	.15	-.18	.01	-.02	.04	-.12	.01	-.02	.01	-.24	-.29	18.6	13.8
REL. SP. LAG.	9									1.0	.05	-.08	-.13	.26	.03	.10	.01	-.23	-.14	.00	.11	-.02	-.25	-.33	22.0	13.0
NO. REJ. GAPS	10										1.0	-.07	-.12	.23	-.17	-.03	-.27	-.04	.02	.02	.05	.02	-.05	.20	1.14	0.42
REJ. GAP (NOSE)	11											1.0	.98	-.03	.05	.06	-.09	.09	.10	.12	-.08	.14	.08	-.22	3.40	2.36
REJ. LEAD (NOSE)	12												1.0	-.24	.09	-.02	-.07	.02	.02	.04	-.07	.05	.08	-.21	2.66	2.43
REJ. LAG (NOSE)	13													1.0	-.17	.37	-.06	.33	.39	.37	-.05	.39	.01	.01	0.74	0.51
PREC. GAP #2	14														1.0	-.10	.29	-.09	.07	.20	.20	.18	.38	.13	3.19	2.23
PREC. GAP #1	15															1.0	.00	.21	.05	.17	.10	.16	.24	.16	3.37	2.37
SUCC. GAP #1	16																1.0	.05	-.12	.71	.22	.71	.26	.06	4.22	3.18
SUCC. GAP #2	17																	1.0	.48	.40	.24	.38	.28	.09	5.12	4.19
SUCC. GAP #3	18																		1.0	.23	.17	.21	.09	.07	5.19	4.48
ACC. GAP X-PT.	19																			1.0	.36	.99	.37	.03	4.86	3.71
ACC. LEAD X-PT.	20																				1.0	.23	.02	.41	0.97	0.52
ACC. LAG. X-PT.	21																					1.0	.38	-.03	3.89	3.55
ABS. SP. X-PT.	22																						1.0	.33	64.9	12.5
DIST. N - X	23																							1.0	334.	103.

TA 8

CORRELATION MATRIX, MEANS AND STANDARD I IATIONS FOR SINGLE MERGES WITH REJECTIONS
HARLEM AVENUE, LEFT-HAND ON-RAM ISENHOWER EXPRESSWAY (EASTBOUND)

Variable		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	Means	Std.Dev.
VEH TYPE/LDG.	1	1.0	-.06	.00	.12	-.13	-.01	.02	-.09	-.03	.13	.	.08	.07	-.07	.01	-.10	-.03	.06	.06	.10	.04	-.15	.27	0.02	0.13
VEH TYPE/MRG.	2		1.0	.00	-.02	.04	.09	-.15	.15	.15	-.11	.	.39	.30	-.09	-.02	.12	-.18	-.11	-.11	.01	-.11	-.28	-.02	0.16	0.37
VEH TYPE/LAG.	3			1.0	.00	.00	.00	.00	.00	.00	.00	.	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	0.00	0.00
THRU VOL.	4				1.0	-.25	-.03	.04	-.17	-.25	.19	-.	.31	.11	-.40	-.08	-.21	-.15	-.04	.02	-.31	.10	-.09	-.04	17.8	3.8
RAMP VOL.	5					1.0	-.07	-.24	.29	.26	-.34	.	.06	.20	.17	.05	.23	.16	-.16	-.17	-.03	-.17	-.09	-.07	13.8	4.1
ACC. GAP (NOSE)	6						1.0	-.21	.14	.17	.35	.	.01	.06	.11	.01	.68	-.06	-.08	.63	.28	.58	.09	.06	5.02	3.62
ABS. SPEED	7							1.0	-.46	-.46	-.30	-.	.03	-.28	-.26	.08	-.15	-.02	.30	-.11	-.32	-.03	.27	-.08	49.2	5.4
REL. SP. LDG.	8								1.0	.83	-.01	.	.13	.09	.19	-.08	.28	.00	-.21	.07	.12	.04	-.24	-.32	24.8	12.3
REL. SP. LAG.	9									1.0	-.13	.	.30	.01	.14	-.03	.34	.22	-.22	.15	.07	.13	-.20	-.34	25.8	12.5
NO. REJ. GAPS	10										1.0	.	.06	.20	-.13	-.10	-.25	-.17	-.05	.29	.40	.19	-.13	.43	1.49	1.54
REJ. GAP (NOSE)	11											1	.92	.42	-.03	-.10	-.07	.04	-.11	.11	.12	.08	-.21	.20	2.43	1.75
REJ. LEAD (NOSE)	12												.0	.03	-.06	-.07	-.08	.07	-.08	.18	.13	.16	-.11	.09	1.28	1.59
REJ. LAG (NOSE)	13													1.0	.03	-.07	.01	-.04	-.13	-.13	.03	-.15	-.28	.33	1.14	0.70
PREC. GAP #2	14														1.0	-.08	.30	.08	.00	.00	.32	-.08	.00	-.01	4.36	4.71
PREC. GAP #1	15															1.0	-.02	-.06	-.15	-.05	-.12	-.02	.16	-.06	3.54	3.94
SUCC. GAP #1	16																1.0	.11	-.09	.32	.13	.29	.05	-.28	3.78	3.31
SUCC. GAP #2	17																	1.0	-.20	-.11	.02	-.12	.04	-.12	3.32	2.86
SUCC. GAP #3	18																		1.0	-.12	-.10	-.10	.03	-.09	3.51	2.82
ACC. GAP X-PT.	19																			1.0	.26	.97	.09	.01	3.92	2.44
ACC. LEAD X-PT.	20																				1.0	.01	-.25	.32	1.05	0.62
ACC. LAG. X-PT.	21																					1.0	.15	-.07	2.87	2.36
ABS. SP. X-PT.	22																						1.0	.07	57.8	12.1
DIST. N - X	23																							1.0	503.	212.

TABLE 9

VARIANCE ACCOUNTED FOR BY UNROTATED FACTORS

Factor No.	Direct Acceptance Merges			Merges With Rejections		
	Eigen Value	Percent Variance Accounted For		Eigen Value	Percent Variance Accounted For	
		Individual	Cum.		Individual	Cum.
1	3.985	19.9	19.9	3.653	16.6	16.6
2	2.566	12.8	32.8	2.854	13.0	29.6
3	1.741	8.7	41.5	2.603	11.8	41.4
4	1.699	8.5	50.0	1.974	9.0	50.4
5	1.444	7.2	57.2	1.676	7.6	58.0
6	1.294	6.5	63.6	1.314	6.0	64.0
7	1.108	5.5	69.2	1.276	5.8	69.8
8	1.071	5.4	74.5	1.045	4.7	74.5
9	0.918	4.6	79.1	0.980	4.5	79.0
10	0.863	4.3	83.4	0.855	3.9	82.9
11	0.753	3.8	87.2	0.716	3.3	86.1
12	0.704	3.5	90.7	0.673	3.1	89.2
13	0.546	2.7	93.5	0.613	2.8	92.0
14	0.406	2.0	95.5	0.453	2.1	94.0

factor analysis was applied to the sets of time-distance information collected at each of the two entrance ramps.[12] To avoid problems due to unequal sample sizes, separate analyses were performed for the samples of direct acceptance merges and for merges involving one or more rejections. Twenty-one pertinent variables were identified for analysis in the case of the direct acceptance samples, and 23 for the equivalent sets of rejection observations. The variables selected are listed in Table 4. Total samples of 224 merges (67 rejections, 157 direct acceptances), and 169 merges (57 rejections, 112 direct acceptances) were analyzed at the right- and left-hand ramps respectively.

Tables 5 through 8 show the matrices of simple-correlation-coefficients derived for each of the four basic analysis groups—right-hand ramp, direct acceptances; right-hand ramp, rejections; left-hand ramp, direct acceptances and left-hand ramp, rejections. Based on these correlation matrices, four rotated factor analyses were performed, yielding four reduced sets of orthogonal variates or "factors." For the sake of brevity only the results of the left-hand ramp analyses will be discussed here.

Table 9 indicates the amount of variance accounted for in each of the two left-hand ramp analyses by the first eight and the first fourteen orthogonal factors respectively. Note that in each case the first eight factors explain approximately three-quarters of the total variance. Hence, one may realistically consider replacing the original sets of 21 and 23 variables by revised, orthogonal sets of 8 variates.

[12]A detailed discussion of factor analysis would be out of place in this paper. For such a discussion the reader is referred to: M. G. Kendall, A Course in Multivariate Statistics, Griffin's Statistical Monograph Series, London, 1957; and W. E. Cooley and P. R. Lohnes, Multivariate Procedures for the Behavioral Sciences, Wiley, New York, 1962. Essentially, factor analysis permits one to identify collinearities within a multidimensional data set, and then to reduce the basic dimensions of the set by defining a new set of orthogonal or independent factors, based upon the original set of variables but fewer in number. These factors are chosen so that each in turn explains a maximal amount of the total residual variance. The character of the new set of orthogonal factors—that is, the parameters represented by each factor—may be evaluated by examining the loading of each new factor on the original set of variables. The process of rotation merely improves the efficiency of this interpretation; it does not affect the basic factor structure.

TA… E 10

ROTATED FACTOR LOAD…S—DIRECT ACCEPTANCES

SINGLE MERGES WITH DIRECT ACCEPTED GAPS AT HARLEM …ENUE

ROTATED FACTOR MATRIX

VARIABLE NO.	NAME	COMMUNALITY 8 FACTORS	1	2	3	4	5	6	7	8
FACTOR NUMBER			1	2		4	5	6	7	8
SUM SQUARES OVER VARIABLES			2.893	2.380	…51	1.578	1.595	1.475	1.316	1.219
1	ICY	0.000	0.000	0.000	…00	0.000	0.000	0.000	0.000	0.000
2	IC	0.627	0.095	0.047	-…65	-0.075	-0.205	0.251	0.659	0.017
3	ICZ	0.487	0.253	0.094	…70	0.530	0.065	0.168	-0.254	-0.091
4	VS	0.740	-0.136	0.036	…64	-0.601	0.455	0.149	0.042	-0.317
5	VR	0.653	0.140	0.090	…77	0.055	-0.755	-0.078	-0.080	-0.091
6	GN	0.950	0.896	-0.028	-…61	-0.025	-0.084	-0.078	0.005	-0.053
7	HDN	0.916	0.149	-0.082	-…31	0.037	0.071	-0.118	-0.015	-0.022
8	HGN	0.919	0.947	0.004	…06	-0.044	-0.127	-0.038	0.013	-0.051
9	SN	0.692	-0.141	-0.682	…31	0.183	0.210	-0.311	-0.130	0.124
10	SRDN	0.784	-0.048	0.879	…20	0.022	0.025	-0.072	-0.036	-0.027
11	SRGN	0.799	-0.008	0.889	…22	0.077	-0.018	-0.018	-0.026	-0.029
12	GN1	0.682	0.054	-0.130	…99	-0.114	-0.129	-0.012	-0.087	0.784
13	GN2	0.552	-0.084	0.026	-…57	0.003	-0.708	0.039	0.089	0.173
14	GNL1	0.593	-0.010	0.342	-…45	0.476	0.061	-0.250	0.368	0.212
15	GNL2	0.682	-0.187	-0.138	-…56	0.742	-0.035	0.102	0.075	-0.237
16	GNL3	0.722	-0.127	0.050	-…16	-0.029	-0.173	0.299	-0.703	0.204
17	GX	0.941	0.641	0.086	-…53	0.104	0.139	0.259	0.237	0.252
18	HDX	0.938	0.179	0.046	-…44	-0.012	-0.004	-0.019	0.044	-0.102
19	HGX	0.871	0.690	0.080	-…48	0.135	0.174	0.331	0.268	0.369
20	SX	0.690	-0.239	-0.390	…18	0.223	0.288	-0.512	-0.069	0.283
21	DNX	0.670	0.006	0.026	-…03	-0.118	-0.112	-0.792	0.058	-0.036

TABLE 11

ROTATED FACTOR LOADINGS—MERGES WITH REJECTIONS

SINGLE MERGES WITH REJECTIONS AT HARLEM AVENUE

ROTATED FACTOR MATRIX

		FACTOR NUMBER	1	2	3	4	5	6	7	8
SUM SQUARES		OVER VARIABLES	2.570	2.909	2.383	2.311	1.945	1.737	1.312	1.228
VARIABLE NO.	NAME	COMMUNALITY 8 FACTORS								
1	ICY	0.246	-0.074	-0.004	0.114	0.375	-0.135	-0.009	0.134	0.225
2	IC	0.708	0.153	-0.024	0.499	-0.202	-0.067	0.487	-0.391	0.013
3	ICZ	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4	VS	0.800	-0.044	0.060	-0.335	0.109	-0.783	0.153	-0.087	0.164
5	VR	0.553	0.248	-0.134	-0.078	-0.327	0.270	0.419	0.291	-0.166
6	GN	0.836	0.066	0.858	-0.069	0.074	0.184	0.201	-0.105	-0.004
7	SN	0.710	-0.669	-0.094	0.121	-0.310	-0.178	-0.324	-0.013	0.076
8	SRDN	0.815	0.875	0.069	0.077	-0.149	0.124	0.011	-0.011	-0.003
9	SRGN	0.865	0.827	0.150	0.247	-0.211	0.107	-0.069	0.193	-0.007
10	KRG	0.783	0.098	0.219	-0.115	0.813	-0.103	0.031	-0.198	0.022
11	RGN	0.946	0.088	0.031	0.928	0.109	0.028	0.244	0.030	0.057
12	RLDN	0.950	0.079	0.069	0.962	0.042	0.054	-0.082	0.048	0.016
13	RLGN	0.757	0.054	-0.066	0.136	0.195	-0.067	0.826	-0.014	0.073
14	GN1	0.629	0.119	0.032	-0.136	-0.042	0.759	0.065	0.044	0.105
15	GN2	0.678	-0.054	-0.044	-0.031	-0.094	-0.022	-0.059	-0.137	-0.801
16	GNL1	0.844	0.155	0.622	-0.130	-0.448	0.363	0.266	0.071	0.087
17	GNL2	0.762	0.053	-0.072	0.037	-0.089	0.086	-0.029	0.853	0.099
18	GNL3	0.656	-0.317	-0.122	-0.056	-0.160	0.138	-0.267	-0.406	0.507
19	GX	0.899	0.068	0.892	0.118	0.203	-0.040	-0.204	-0.006	-0.013
20	HDX	0.724	0.190	0.148	0.055	0.571	0.570	0.001	-0.054	0.099
21	HGX	0.883	0.021	0.886	0.108	0.060	-0.192	-0.212	0.008	-0.040
22	SX	0.573	-0.474	0.205	-0.118	-0.177	0.085	-0.250	0.189	-0.395
23	DNX	0.779	-0.373	-0.046	0.124	0.695	0.145	0.320	0.047	-0.113

TABLE 12

INTERPRETATION OF MAJOR FACTORS

Factor	Direct Acceptance Merges	Merge With Rejections
1	Accepted Lag/Gap	Speed Index
2	Speed Index	Accepted Lag/Gap
3	Accepted Lead/Gap	Rejected Lead/Gap
4	Mainstream Flow Condition—Availability of Alternate Gaps	Previously Rejected Gaps
5	Mainstream/Ramp Volume	Mainstream Flow Condition
6	Point of Merge	Rejected Lag
7	Ramp Vehicle Type/Alternate Gaps	Ramp Vehicle Type/Alternate Gaps
8	Mainstream Flow Condition	Mainstream Flow Condition

Tables 10 and 11 summarize the rotated factor loadings for the first eight orthogonal factors for each of the two data groups. The first row in each table indicates the percentage of the total variance accounted for by each individual rotated factor; the remaining rows indicate the individual factor loadings. Based upon these loadings certain tentative interpretations may be placed on the major factors.

In the case of the direct-acceptance analysis, factor 1 clearly represents a combined static and dynamic "gap/lag size" index, factor 2 a "speed" index and factor 3 a "gap/lead size" index. Factors 4, 5 and 8 each reflect the general flow conditions and availability of gaps during the merge. Factor 6 loads heavily on the point of merger and factor 7 represents an interaction between the type of merging vehicle and the availability of mainstream gaps.

less clear. In this case factors 1 and 2 are reversed; the former now represents a speed index, the latter a combined accepted gap/accepted lag measure. Factors 3, 4 and 6 all reflect the merging vehicle's rejection behavior; factor 3 is loaded heavily by rejected gap/rejected lead values, factor 4 by the number of previously rejected gaps and factor 6 by rejected gap/rejected lag values. Finally, factors 5, 7 and 8 all reflect the mainstream flow conditions pertaining during the merge and also, in the case of factor 7, the influence of ramp vehicle type. These factor interpretations are summarized in Table 12.

A note of caution is appropriate here. The factors listed in Table 12 do not necessarily represent the major "traffic factors" influencing the merging process; they are merely statistical aggregations of the original set of variables which were themselves postulated to bear some relation to gap acceptance behavior. The process of factor analysis simply identifies the major correlations within the sets of variable observations and places them in groupings which are statistically independent of each other. The results of the analysis are obviously very much dependent on the selection of the original sets of variables.

The degree of collinearity within the data is not particularly high—eight independent factors are required to explain 75 percent of the total variance. Furthermore, much of the collinearity is somewhat artificial, resulting from dual observations of essentially the same parameter at slightly different points in time. If these artificial correlations are ignored it becomes apparent that there is but little interdependence within the data, the values of accepted gap size or gap structure, merging speed, number of rejected gaps, gaps, mainstream flow condition and vehicle type as evidenced by this sample data set being virtually independent. In particular, it should be noted that although accepted gap, lag and lead values load heavily on several factors, they do not load significantly on those factors reflecting merging speeds, mainstream flow conditions or previous rejection behavior. The reasons for this are perhaps not to hard to discern. As noted

in a previous section the data are derived primarily from an analysis of "noncritical" merging conditions, away from the critical acceptance/rejection envelope. It is to be expected therefore that merging speeds and accepted gap size will not be subject to high intercorrelations. The possibly surprising fact is that the inter-correlations are so low as to suggest almost total independence, implying that an original variable itself may in many cases serve as an efficient proxy for an orthogonal factor.

Finally, undue importance should not be placed on the relative ordering of the factors and on the magnitudes of the eigenvalues. These reflect more than anything else the inbuilt bias referred to previously resulting from overmeasurement of single parameters. Redefinition of the original variable set would quite probably alter the ordering of the factors considerably.

TOWARD AN EMPIRICAL MODEL OF GAP ACCEPTANCE

A number of alternative techniques suggest themselves as appropriate to the formulation of an empirical model of gap acceptance behavior. Hurst et al, for example, in a report referred to previously (32), have suggested the use of relatively complex "gap-size" indexes, incorporating estimates of relative merging speed, spatial headways and desirable safe headway in addition to the standard time-gap measurements; Haight et al (25) have developed a simple, minimum safe-headway model based on the principles of elementary dynamics; Worrall (60) and Vardon (55) have both formulated empirical descriptions of acceptance/rejection behavior as a function of relative merging speed. In this final section of the paper attention is given to a slightly different type of model, in which a linear function is developed to estimate the probability of a gap of a given "character" being accepted or rejected. Two specific formulations are discussed, one based on a simple multiple regression analysis with dummy variables, the other on multivariate discriminant analysis.

Multiple Regression Analysis with Dummy Variables

A regression function is postulated of the form:

$$P(A) = A_1X_1 + A_2X_2 \ldots A_iX_i + \ldots + A_mX_m \tag{1}$$

where

$$P(A) = \text{probability of a driver accepting a gap} \begin{cases} P(A) = 1 \text{ for an acceptance} \\ P(A) = 0 \text{ for a rejection} \end{cases}$$

$X_1, X_2, \ldots, X_m$ = gap characteristics (e.g., available lag size, relative merging speed, ramp vehicle type)

$A_1, A_2, \ldots, A_m$ = Regression coefficients

The function differs from a normal regression equation in that the gap characteristics X_i are expressed as "dummy" variables rather than as continuous variables, i.e., each X_i is a vector of dichotomous variables $(x_{i1}, \ldots, x_{ij}, \ldots, x_{in})$, with each element x_{ij} taking on a value of either 1 or 0 depending on the observed value of X_i, and each A_i is a corresponding vector of constants $(a_{i1}, \ldots, a_{ij}, \ldots, a_{in})$. In expanded form, therefore, Eq. 1 may be rewritten:

$$\begin{aligned} P(A) = {} & (a_{11}x_{11} + a_{12}x_{12} + \ldots + a_{1k}) + (a_{21}x_{21} + a_{22}x_{22} + \ldots + a_{2e}x_{2e}) \\ & + \ldots + (a_{m1}x_{m1} + a_{m2}x_{m2} + \ldots + a_{mn}x_{mn}) \end{aligned} \tag{2}$$

Consider an empirical example where X_1 = static lag, X_2 = relative merging speed and X_3 = ramp vehicle type, with the ranges of the dichotomous variables X_1, X_2 and X_3 defined (for illustrative purposes) as:

$x_{11} \rightarrow$ 0-0.99 seconds; $x_{21} \rightarrow$ -10 to -5 mph; $x_{31} \rightarrow$ Passenger Car
$x_{12} \rightarrow$ 1.00-1.99 seconds; $x_{22} \rightarrow$ -5 to 0 mph;
$x_{13} \rightarrow$ 2.00-2.99 seconds; $x_{23} \rightarrow$ 0 to +5 mph;
$x_{24} \rightarrow$ +5 to +10 mph;

A situation in which a passenger car accepts a static lag of 2.5 seconds at a relative merging speed of +2 mph yields an empirical "dummy" observation of:

$$
\begin{aligned}
1 &= (a_{11} \cdot 0 + a_{12} \cdot 0 + a_{13} \cdot 1) + (a_{21} \cdot 0 + a_{22} \cdot 0 + a_{23} \cdot 1 + a_{24} \cdot 0) \\
&= a_{13} \cdot 1 + a_{23} \cdot 1 + a_{31} \cdot 1
\end{aligned}
$$

Similarly, a rejection by a truck of a lag of 0.2 seconds at a relative merging speed of -9 mph yields an observation:

$$0 = a_{11} \cdot 1 + a_{21} \cdot 1 + a_{31} \cdot 0$$

Inserting a set of such dummy observations as input to a multiple regression analysis yields estimates $A_1^*, A_2^*, \ldots, A_m^*$ for the vectors $A_1, A_2, \ldots, A_m$, the individual elements $a_{11}, a_{12}, \ldots; a_{21}, a_{22}$, etc. of which may be interpreted as the partial conditional probabilities of gap acceptance, given the occurrence of a gap having the particular set of characteristics, $x_{11}, x_{12}, \ldots; x_{21}, x_{22}, \ldots$, etc.[13] For example, in the illustration above, the car driver faced with a lag of 2.5 seconds at a relative speed of +2 mph would have, based on this analysis, an average probability of accepting the gap equal to $(a_{13}^* + a_{23}^* + a_{31}^*)$, where a_{13}^*, a_{23}^* and a_{31}^* are the estimates for the elements a_{13}, a_{23} and a_{31} of the vectors A_1^*, A_2^* and A_3^* as determined from the regression equation.

The value of $P(A)^* = (a_{1j}^* + a_{2k}^* + a_{3e}^*)$ may be constrained within the range $0 \leq P(A)^* \leq 1.0$ either directly by modifying the normal regression equations or by means of an arbitrary correction due to Orcutt.[13] The simple linear form of the acceptance [illegible] achieved by using as input the set of factors derived from an orthogonal factor analysis. Of necessity the application of this method requires a relatively large and uniform data set. The small sample of empirical data available for this study precludes inclusion of a worthwhile empirical example.

Discriminant Analysis

Consider a multivariate data set consisting of m observations on n variables, with the set of observations classified into two partially overlapping groups (e.g., accepted and rejected mainstream gaps). A linear discriminant analysis then defines, first, that linear combination of the original variables which discriminates "best" (in the sense of minimizing the ambiguity of group membership) between the two observation groups and, second, provides an estimate of the probability that a given observation most properly belongs in group 1 or group 2. In the case of a gap acceptance analysis, these latter values may be interpreted directly as estimates of the probabilities that a gap having a given set of characteristics will be accepted or rejected.[14]

As in the case of the dummy variable regression approach, the analysis presumes a simple linear statistical model—in this case the linear discriminant function

[13] A detailed exposition of the dummy variable technique is given in: Johnston, J., Econometric Methods, McGraw-Hill, New York, 1960. In essence the values a_{ij}^* correspond to estimates of cell means in an analysis of variance. The problem of heteroscadasticity arising from the dichotomous nature of the dependent variable may be at least partially corrected for by a technique due to Goldberger.

[14] The discussion and application of discriminant analysis given here is based on: Cooley and Lohnes, op. cit.

$$Z = K_1x_1 + K_2x_2 + \ldots + K_ix_i + \ldots + k_mx_m$$

where x_1, x_2, x_i, x_m are the original characteristic variables expressed in standardized form (not, in this case, as dummy variables) and $K_1, K_2, \ldots, K_i, \ldots, K_m$ are the discriminant function coefficients. Validity of the linear form may again be approximated by using as input to the discriminant analysis standardized, orthogonal factor-scores derived from a factor analysis. The probabilities of group inclusion are then computed by evaluating the multivariate distributions of group observations as defined in the discriminant space (see Cooley and Lohnes, Chapter 7).

In contrast to the dummy variable regression technique, it is possible in this case to give a simple illustration of the analysis method based on the sample of data available for this study. It was noted earlier that relatively little collinearity existed within the original factor-analysis data set. In particular, the following 9 characteristic variables were seen to be virtually independent of one another:

Variables*	Variable Name
X_1	Ramp Vehicle Type
X_2	Ramp Vehicle Speed
X_3	Relative Speed Ramp--Leading Mainstream Vehicle
X_4	Relative Speed Ramp—Lagging Mainstream Vehicle
X_5	Static Lead Time at Nose
X_6	Static Lag Time at Nose
X_7	Size of First Succeeding Mainstream Gap at Nose
X_8	Size of Second Succeeding Mainstream Gap at Nose
X_9	Size of Third Succeeding Mainstream Gap at Nose

These variables (based on groups of 112 acceptances and 57 rejections at the left-hand ramp) were used as input into a linear discriminant analysis, yielding the linear discriminant function (LDF)

$$Z = 0.479\ X_2^* + 0.947\ X_6^*$$

where X_2^*, X_6^* are standardized values of X_2, X_6, and a level of significance of 0.05 defines the limit for variable inclusion in the LDF.

Only the values of static lag time and ramp vehicle speed contribute significantly to the power of the discriminant function in distinguishing between the acceptance and rejection groups; the remaining seven variables yield no significant increase in the discriminatory power of the function. The relative contributions of variables X_2 and X_6 to the discriminant function may be assessed by comparing their corresponding scale-vector values: $X_2' = 6.114$, $X_6' = 10.879$ (i.e., lag time is roughly twice as important, in this case, as merging speed in distinguishing between the gap acceptance and rejection groups).

Calculation of the group inclusion probabilities indicated that considerable overlap existed between the two data sets, with the rejection group lying almost entirely within the limits of the acceptance group. In particular, 30 of the 112 acceptance observations and 7 of the 57 rejection observations had associated with them probabilities which indicated that the values of their characteristic vectors (i.e., lag size, speed, etc.) were associated more closely with the complementary group. The two group centroids had associated with them complementary membership probabilities of 0.0140 (acceptances) and 0.2807 (rejections) (i.e., the centroid of the group of rejection observations, on the basis of the discriminant analysis, had a probability of 0.3696 of being associated not with a gap rejection but rather with an acceptance). This suggests either that the original 9 characteristic variables provide only an inadequate description of acceptance/rejection behavior or else that the dispersion matrices for the two observation groups are too widely divergent. The results are sufficiently intriguing, however, to warrant further extension of the simple illustrative analysis described here.

CONCLUSIONS

This paper has touched briefly on a number of elementary points relating to the analysis of gap acceptance behavior at freeway entrance ramps. In many cases the analyses have been little more than exploratory illustrations of what might be done with a larger data sample. It has been shown, however, that the development of a more detailed empirical description of the gap acceptance/rejection process is both feasible and also desirable. The ability to distinguish between those factors which exert a significant influence on merging behavior and which in some sense are susceptible to external control has considerable implications both for the design and operation of future freeway control systems and also for the formulation of future theoretical models. An initial specification of these factors represents the major contribution of this paper.

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Appendix A

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Appendix B

DEFINITIONS

1. Time-Distance Trace—The trajectory of a vehicle defined in time and space as it passes through the merge area of an entrance ramp.
2. Time-Gap—The time headway between successive vehicles, measured with respect to the front of each vehicle.
3. Lead-Time—The time headway between a merging vehicle and the preceding mainstream vehicle.
4. Lag-Time—The time headway between a merging vehicle and the succeeding mainstream vehicle.
5. Static Analysis—An analysis based entirely on time-gap, lead and lag measurements made at the entrance ramp nose.

6. Dynamic Analysis—An analysis based on measurements of gap, lead and lag size made at varying points within the merge area, and defined in terms of the motion of the merging vehicle.
7. "X" Point—The commencement of the physical merging maneuver, the point at which a ramp vehicle first encroaches onto the mainstream lanes.
8. Dynamic Gap—The time headway between a ramp vehicle and the succeeding mainstream vehicle measured at the instant the ramp vehicle is overtaken by the immediately preceding mainstream vehicle.
9. Isolated Merge—A merge by a ramp vehicle separated by at least 10 seconds from the nearest preceding or succeeding ramp vehicle.
10. Multiple Merge—A merge by a platoon of two or more vehicles into a single mainstream gap.
11. Single Merge—A merge by either an isolated ramp vehicle or by a lead vehicle in a multiple merge; includes all instances in which one vehicle merges into a single mainstream gap plus all lead vehicles in multiple merges.
12. Modified Multiple Merge—A measure of the effective gap size for trailing vehicles in a multiple merge, defined as either the time headway between the preceding ramp vehicle in a platoon and the immediately succeeding mainstream vehicle or the time headway between the preceding and succeeding mainstream vehicles, whichever is the lesser.
13. "O" Point—The termination of the physical merging maneuver, the point at which the ramp vehicle finally enters completely into the mainstream lanes.
14. Critical Acceptance Value—The median or 50 percent response value derived from a probit or Raff analysis; that size of gap, lead or lag which is accepted 50 percent of the time.
15. Forced Merge—A merge in which the ramp vehicle forces its way into a small mainstream gap, causing one or more of the mainstream vehicles to change its speed or direction of travel.
16. Gap Stability—The variation of mainstream gap size over the length of the merge area.

Gap Acceptance Characteristics for Ramp-Freeway Surveillance and Control

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The purpose of this investigation was to determine gap acceptance characteristics and merging delay characteristics for 6 inbound entrance ramps on the Gulf Freeway Surveillance and Control Project. A 20-pen graphic recorder operated by 4 men stationed adjacent to the merging area between the freeway and the frontage road was used in collecting the data.

Merging vehicles were divided into 2 groups: those in which the driver rejected gaps before finally accepting a gap and those in which the driver of a ramp vehicle accepted the first gap. The former was referred to as "stopped" vehicles and the latter as "moving" vehicles. The critical gap for stopped vehicles was found to be about 20 percent higher than for moving vehicles. In addition, it was concluded that the critical gap (median) for the merging maneuver from an entrance ramp is independent of the freeway volume but is apparently affected by ramp geometrics and ramp controls.

A distribution of critical gaps was formed and fitted to a gamma distribution. Merging delay values calculated using this distribution were shown to be higher than those calculated assuming that all drivers have [illegible]

If the distribution of time spent by the merging vehicle at the head of the queue is approximated by a gamma distribution, the entrance ramp merging operation may be considered within the context of classical queuing theory. Based on this queuing model, a ramp metering technique was developed which takes into account the individuality of entrance ramps. Finally, it is shown how the need exists for an automatic ramp control technique combining the microscopic approach developed in this paper with the systems or macroscopic approach which has been used as the basis for the past manual ramp metering experiments on the Gulf Freeway.

THE Gulf Freeway Surveillance and Control Project sponsored by the Texas Highway Department and U.S. Bureau of Public Roads has the basic objective of developing criteria for the design and operation of automatic surveillance and control systems which would permit the attainment of acceptable levels of service on urban freeways during periods of peak traffic demand.

A traffic surveillance system should involve the continuous sampling of basic traffic characteristics for interpretation by established control parameters, to provide a quantitative knowledge of operating conditions necessary for immediate rational control and future design. Figure 1 shows some relationships between many of these control parameters, the basic traffic characteristic, and their ultimate applications. Most of

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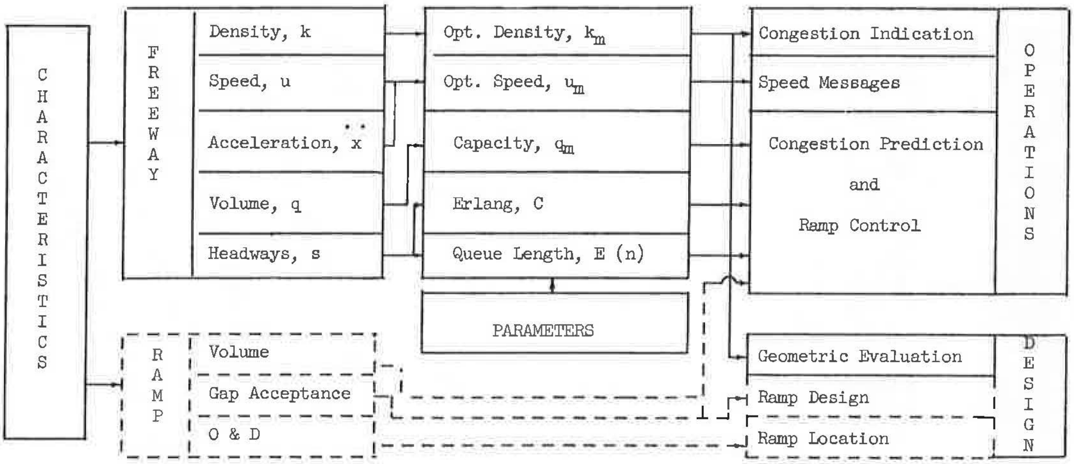


Figure 1. Analysis of freeway traffic congestion.

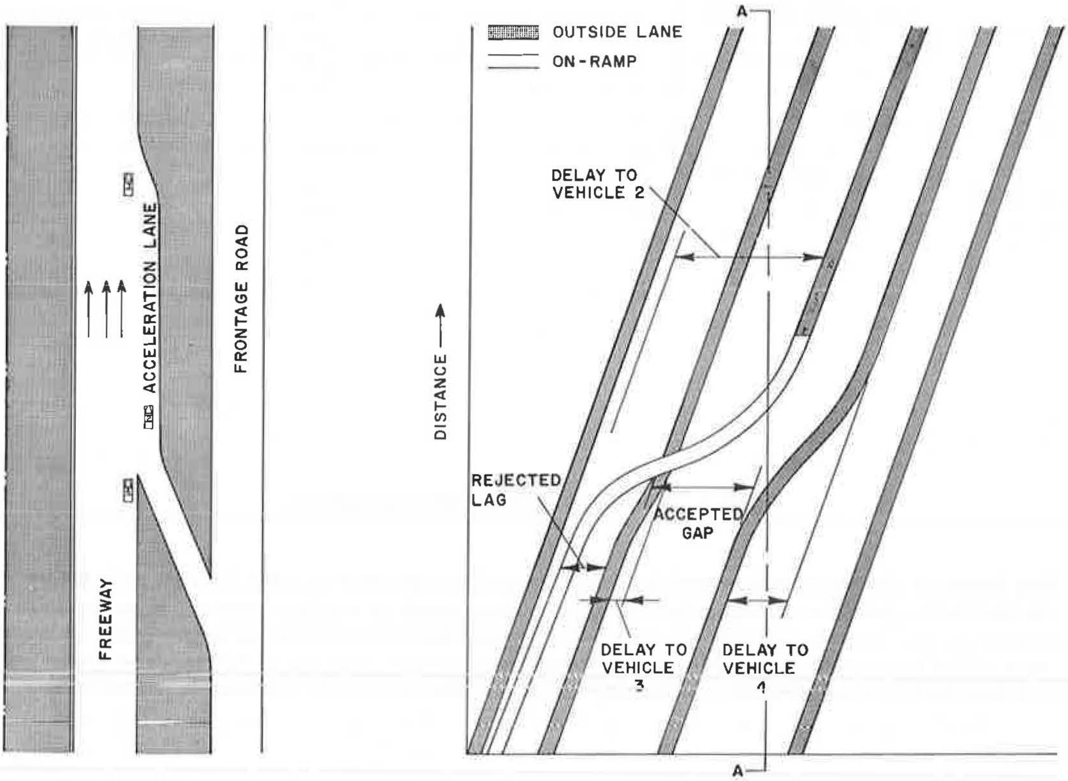
the freeway characteristics and freeway control parameters shown in Figure 1 were evaluated in previous studies and have been discussed in various project research reports (1, 2). These operational studies also pointed to the need for some form of freeway ramp control. Utilizing a comprehensive systems plan for metering and controlling the inbound entrance ramps, a significant improvement in the inbound level of service was obtained (3). As control techniques become more sophisticated, it is felt that greater reliance will be placed on ramp metering and less on complete ramp closure, making it imperative that the characteristics associated with both normal and controlled freeway ramp operation be appreciated.

FREEWAY MERGING MANEUVER

One of the most important elements of freeway operation is the merging maneuver from an entrance ramp onto the shoulder lane of a freeway. Merging, in this case, is simply the absorption of the stream of ramp traffic. A vehicle is said to merge when it moves from the acceleration lane to the shoulder lane. The driver in the merging vehicle must, of course, select the proper moment to execute the merging maneuver based on his judgment as to whether or not a gap which he can enter is large enough for a safe merge.

A merging gap is the interval of time between the arrival of two successive shoulder lane vehicles at a point in the merging area. The concept of gap acceptance is important in describing the interaction of the freeway and ramp streams. It is assumed that the ramp driver measures each gap t in the shoulder lane and either merges (accepts the gap if $t>T$) or waits (rejects the gap if $t<T$) where T may be assumed to be the driver's critical gap for that decision (Fig. 2).

Most of the work on gap acceptance applies to the crossing maneuver at intersections. Greenshields (4) defined the "acceptable average-minimum time gap" as a gap accepted by half of the drivers. Raff (5) used a slightly different parameter, the "critical lag." A lag is the interval from the arrival of a minor stream vehicle at the point of conflict to the arrival of the next major stream arrival (Fig. 2). The critical lag is the size lag which has the property that the number of accepted lags shorter than the critical lag is equal to the number of rejected lags longer than the critical lag. For the data presented in Raff's report, the Greenshields gap quantity averages 0.2 sec longer than Raff's critical lag. The principal use of both parameters, or any such parameter, is to simplify the computation of the number of cars delayed by permitting the assumption that all intervals shorter than the critical value (lag or gap) are rejected whereas all longer intervals are accepted.



SPACE DIAGRAM FOR TIME A-A

The speed of the merging vehicle is important in considering the distribution of gaps acceptable to the merging driver at a freeway entrance ramp. By changing the speed of his own car, a ramp driver also changes his time criterion (critical gap) for a safe merge. In simulating entrance ramp operation, the Midwest Research Institute (6) used a distribution of gaps for nonstop merges with a range of values roughly equal to half those values used for vehicles merging from a stop. Actually, because of the scarcity of published characteristics on entrance ramp operation, the exact relationship between relative speed and gap acceptance is still one of conjecture.

Although numerous data are available regarding merging at an intersection, this information would probably not be applicable as a model for describing ramp merging. The availability of an acceleration lane not only increases the ability of the merging vehicle to minimize its relative speed, but provides the driver with a limited opportunity to inspect different parts of the freeway stream for places to merge. Such considerations as a driver's desire to merge as safely, as quickly, or as directly as possible make the problem a question of policy and complicate the exact formulation of a mathematical model (7).

PURPOSE AND OBJECTIVES

The purpose of this investigation is to determine the gap acceptance characteristics for entrance ramps on the Gulf Freeway Surveillance and Control Project. This information will be used to calibrate some of the ramp metering models slated for future use on this project (1, 2). It is also believed that this research will help to fulfill the very definite need for gap acceptance data for ramp simulation inputs (6), as well as the formulation and testing of ramp-freeway merging models (8).

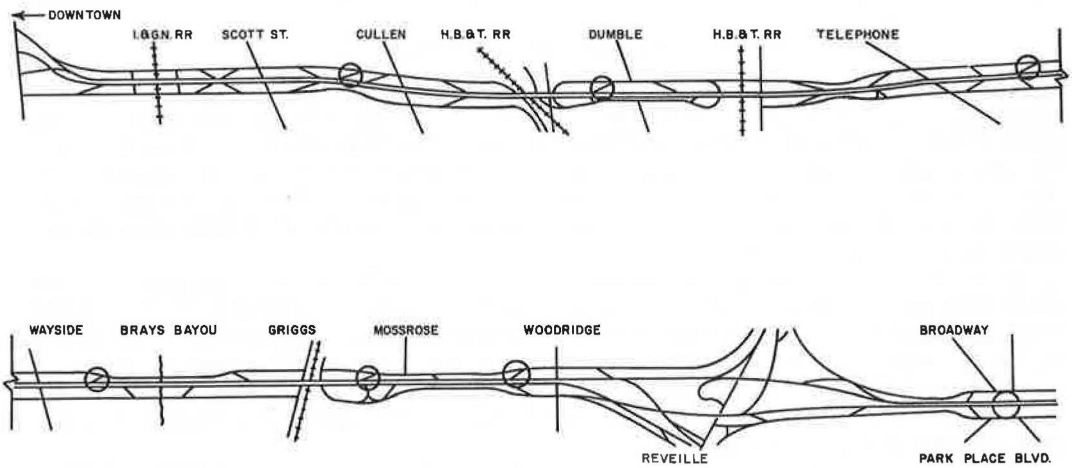


Figure 3. Location of study ramps on Gulf Freeway Surveillance Project.

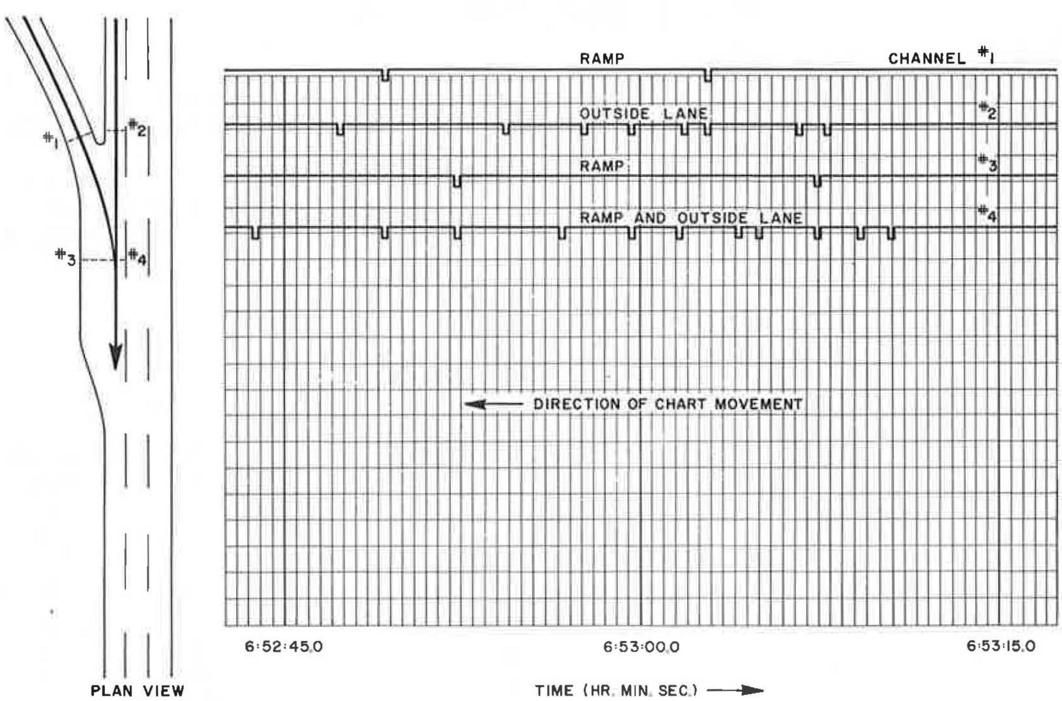


Figure 4. Section of recording chart showing data collection procedure.

Specific objectives of this investigation are (a) to measure gap acceptance characteristics; (b) to determine delay characteristics at the merging area; and (c) to relate these characteristics to freeway operations and design.

STUDY PROCEDURE

All study sites were located on the inbound entrance ramps of the Gulf Freeway in Houston, Texas (Fig. 3). This facility has three 12-ft lanes in each direction. The through lanes of the Gulf Freeway overpass the intersecting roadways producing a "roller-coaster" effect. This design seems to compromise the merging maneuver to the extent that where ramps enter before the overpass some of the acceleration lanes terminate at the overpass structures, and where ramps enter after the overpass the sight distance may be reduced.

A 20-pen graphic recorder operated by four men stationed adjacent to the merging area between the freeway and the frontage road was used in collecting the data. The code for recording traffic behavior is summarized in Figure 4. One pen was assigned to the decision point on the entrance ramp and one pen to a similar point opposite the ramp nose on the outside lane of the freeway. The other two pens were assigned to a point in the merging area, one pen recording both outside lane and merging vehicles and one pen merging vehicles only.

The studies were planned so as to be conducted during periods of congestion-free operation, yet during periods when the ramp demand was high enough to reduce study duration.

The clock times for the four channels shown in Figure 4 were placed on punch cards. A program was written for the IBM 7094 which would output the ramp vehicle arrival time, outside lane arrival time, and time all vehicles passed the designated point in the merging area. The output also included all the gaps rejected by a ramp vehicle plus the gap finally accepted. The study included over 12,000 gaps for 6 entrance ramps.

GAP ACCEPTANCE CHARACTERISTICS

Most studies of gap acceptance have been concerned with the determination of the critical gap, a time gap just as likely to be accepted as it is to be rejected. The probability that it will be accepted is thus equal to one-half.

In evaluating the critical gap, a given gap must be either accepted or rejected by a given driver and as such is binomial in nature. Yet each driver can accept only one gap while he can reject several of them. This means that if all rejected gaps are given

TABLE 1

ACCEPTED AND REJECTED GAPS AT DUMBLE RAMP[a]

Length of Gap t (sec)	Stopped Vehicles		Moving Vehicles		All Vehicles	
	No. Accepted Gaps<t	No. Rejected Gaps>t	No. Accepted Gaps<t	No. Rejected Gaps>t	No. Accepted Gaps<t	No. Rejected Gaps>t
0.0	0	100	0	89	0	180
Δt = 0.5	0	100	0	89	0	189
1.0	0	95	0	80	0	175
1.5	0	71	1	52	1	123
2.0	2	49	7	27	9	76
2.5	11	34	a = 13	c = 16	a = 24	c = 50
3.0	a = 15	c = 20	b = 26	d = 7	b = 41	d = 27
3.5	b = 23	d = 10	38	4	61	14
4.0	32	5	46	3	78	8
4.5	41	4	55	3	96	7
5.0	48	2	63	2	111	4
5.5	57	0	70	1	127	1
10.0	100	0	106	0	206	0

[a]Critical Gap, $T = t + \frac{(c-a)\,\Delta t}{(b+c)-(a+d)}$; T (stopped) = 3.1; T (moving) = 2.5; and T (all) = 2.8

the same weight as accepted gaps, then the percentage of intervals accepted for a particular size will not be a true measure of the percentage of drivers who find such an interval acceptable. If the percentage of intervals accepted is to be used to determine the percentage of drivers who are willing to accept them, then the same number of intervals must be counted for each driver. Raff (5) accomplished this by counting only lags and ignoring the gaps.

For this investigation, merging vehicles were divided into two groups, those in which the driver rejected gaps before finally accepting a gap and those in which the driver of a ramp vehicle accepted the first gap. The former is referred to as "stopped" vehicles and the latter "moving" vehicles, the implication being that if a ramp vehicle rejects the first gap he must stop. Theoretically, this would not have to be true where there is an acceleration lane present. Yet, on the Gulf Freeway, when traffic is heavy, ramp vehicles tend to shy away from using the acceleration lane, perhaps for fear of being "trapped" at the end while other ramp vehicles behind them accept the available gaps.

This arbitrary classification of stopped and moving merges afforded the means of eliminating any bias due to the inclusion of all rejected gaps. For example, in the case of stopped vehicles only two gaps were considered for each vehicle, the largest rejected gap and the gap finally accepted. To evaluate the gap acceptance characteristics for moving vehicles only the first gap available to those ramp vehicles not delayed by previous ramp vehicles was considered, much in the same manner that Raff treated lags.

The number of gaps accepted and rejected at the Dumble entrance ramp is given in cumulative form in Table 1 for stopped and moving vehicles, and then for all vehicles. The critical gap may be determined algebraically as indicated in Table 1 or graphically as shown in Figure 5, in which two cumulative distribution curves depict the number of accepted gaps shorter than t and the number of rejected gaps longer than t. The value of t for which these two curves intersect is the critical gap.

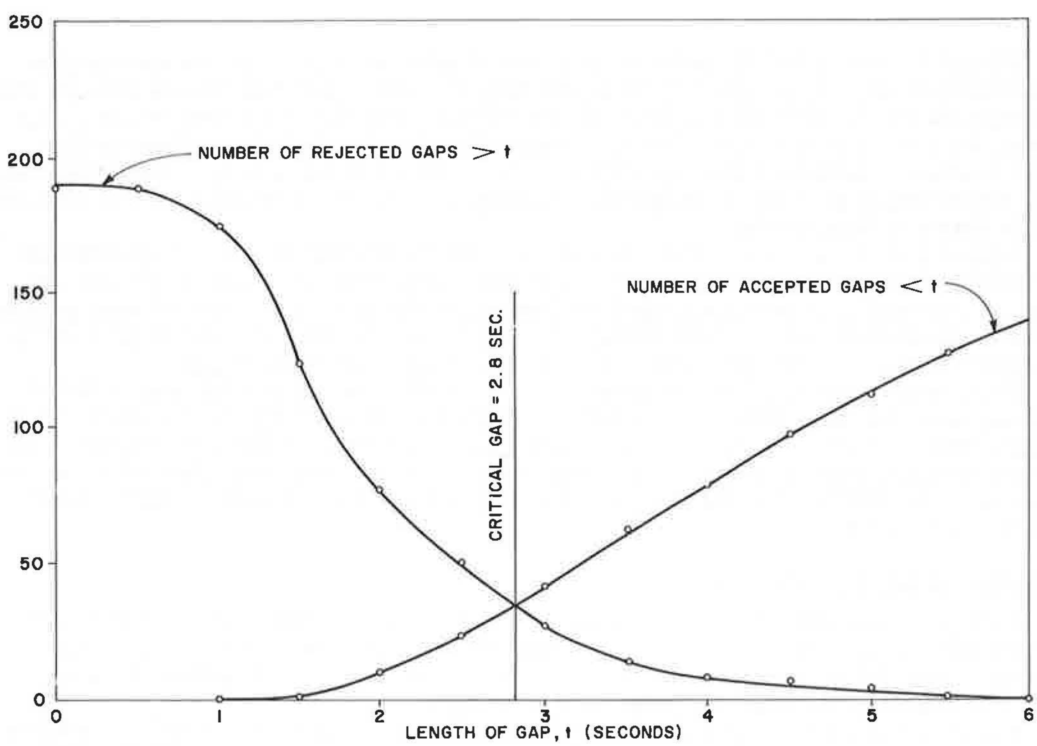


Figure 5. Accepted and rejected gaps for all vehicles at Dumble ramp.

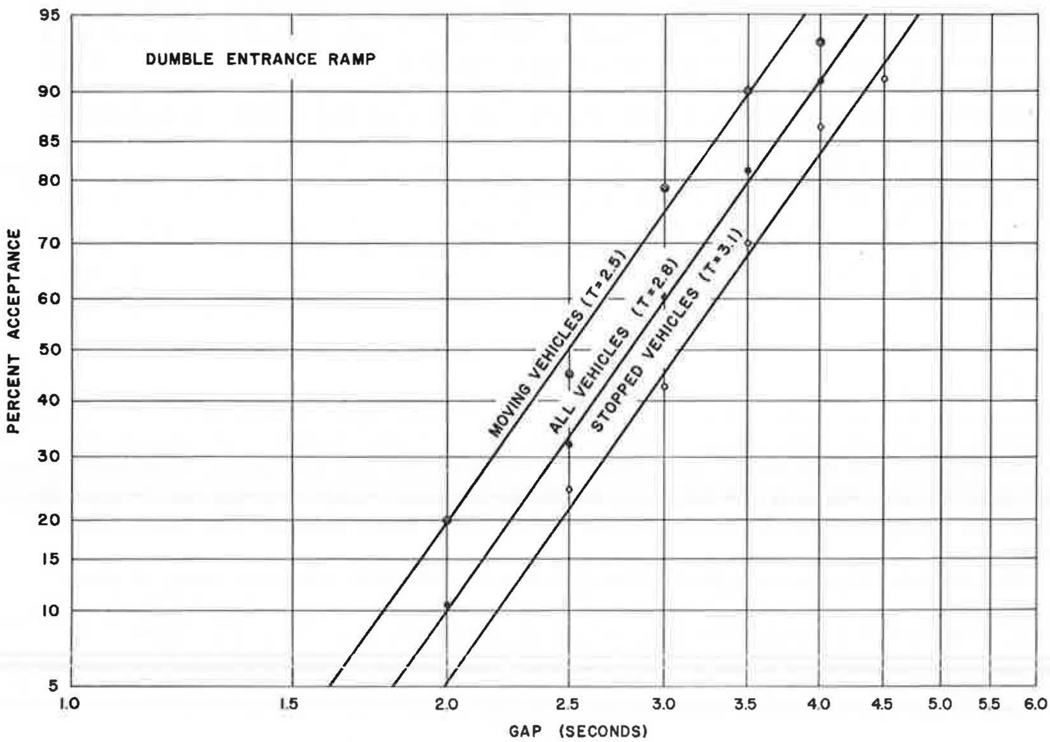


Figure 6. Percent acceptance for merging vehicles.

Bissell (9) calculated the percentage of acceptance observed at an intersection for each time group, plotted the data on log probability paper, and then drew a straight line through the point. Solberg and Oppenlander (10) also calculated the percentage acceptance, but converted the probability scale to probits and then fitted a regression line with confidence limits through the points. The critical gap was taken as the median gap, the 50 percentile in the case of Bissell, the probit of acceptance equal to 5.0 in the case of Solberg and Oppenlander.

Figure 6 shows a procedure combining some of the features of the Raff and Bissell methods. Using Table 1, the number of accepted gaps less than t was divided by the sum of the number of accepted gaps less than t plus the number of rejected gaps greater than t. A straight line is drawn through these points on the log-probability graph (Fig. 6) with the 50 percentile or median gap used to establish the critical gap.

The reason for using this new procedure for establishing the critical gap is that it serves to define the parameter in a way relating directly to the manner in which it is used. The probability of accepting a smaller gap than the cirtical gap is exactly equal to the probability of rejecting a longer gap, thus equating the critical gap to the median gap for a distribution of critical gaps. This concept will be discussed in more detail later in the paper.

Factors Affecting Critical Gap

Raff (5) investigated factors influencing the critical lag at urban intersections and concluded that the critical lag (a) varies directly with main street speeds and main street width; (b) is independent of the main street volume; and (c) is influenced by sight obstructions and the side-street traffic pattern.

The effect of relative speed on the critical gap for merging ramp vehicles is evident in Figure 6. At the Dumble entrance ramp, the critical gap for moving vehicles (based

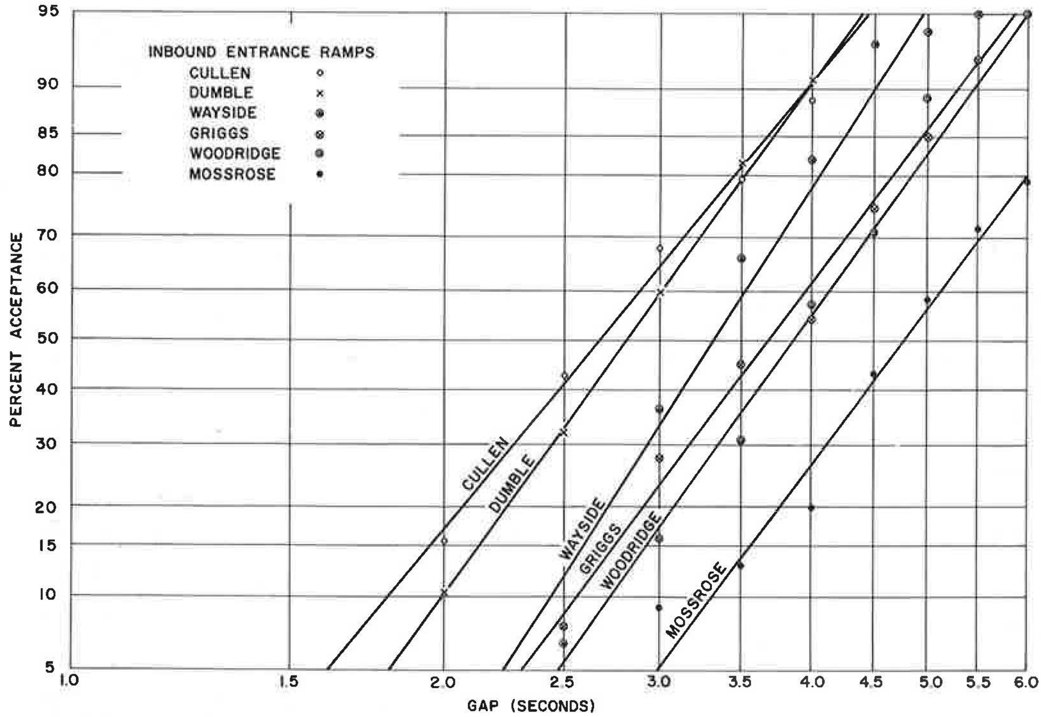


Figure 7. Variations in percent acceptance for merging vehicles at different ramps.

TABLE 2

RELATIONSHIP BETWEEN CRITICAL GAP AND OTHER ENTRANCE RAMP ELEMENTS

Element	Cullen	Dumble	Wayside	Griggs	Mossrose	Woodridge
Critical gap (sec)	2.7	2.8	3.3	3.7	4.7	3.8
Avg. freeway volume outside lane during period of observ. (veh/hr)	1,350	1,240	1,100	700	1,060	970
Percent grade on freeway	0.0	+0.5	-1.5	+1.5	+1.0	-3.0
Ramp length (ft)	320	320	330	230	190	300
Ramp width (ft)	20	20	20	20	15	20
Angle of entry (deg)	12	12	11	11	10	12
Acceleration lane length (ft)	–	330	–	310[a]	250[a]	370
Auxiliary lane length (ft)	350	–	510	–	–	–

[a]Taper-type acceleration lane terminating at overpass structure.

on the first gap evaluated) is seen to be 2.5 sec as against a critical gap of 3.1 sec for stopped vehicles (vehicles rejecting at least one gap before finally accepting a gap).

The graph of the percent acceptance for merging vehicles at different ramps (Fig. 7) indicates that the critical gaps were not the same, but varied from ramp to ramp as they do among intersections. To determine to what extent this variability was due to ramp geometrics, the relationship between critical gap and entrance ramp elements is given in Table 2. First, the table indicates that the assumption that the critical gap for a ramp is independent of the freeway outside lane volume is justifiable. Second, analyzing the table objectively, there is no significant correlation between any single

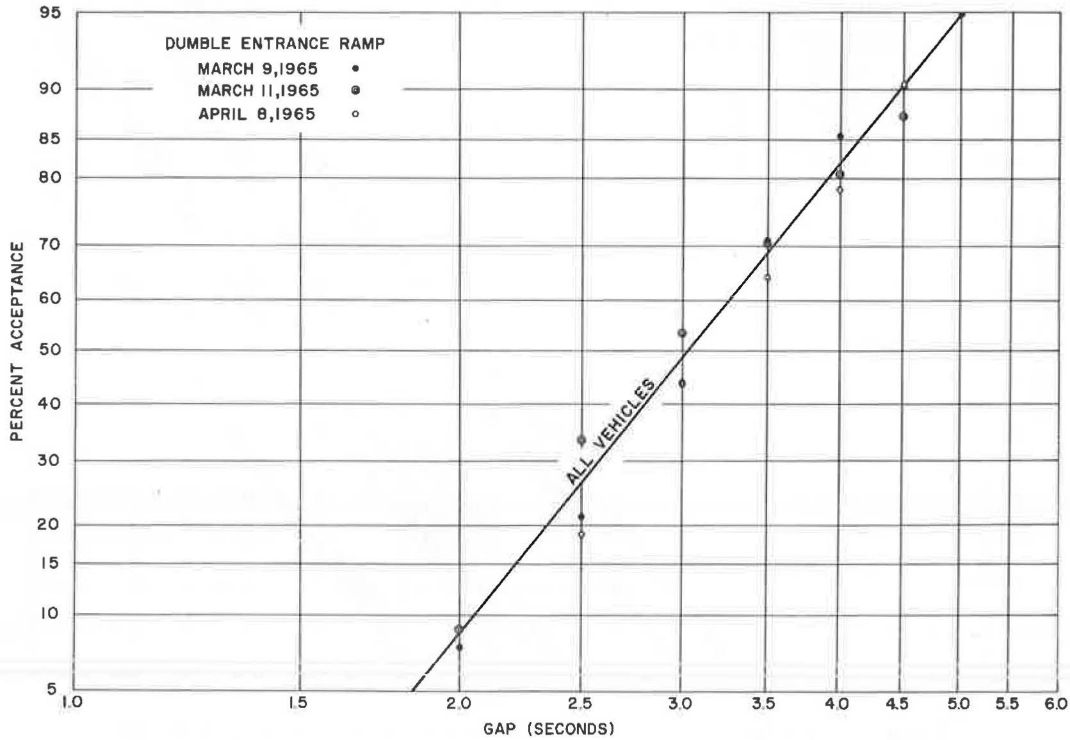


Figure 8. Variations in percent acceptance for merging vehicles at same ramp.

element and the critical gap. Subjectively, however, to the person familiar with the study ramps, it was not surprising that the Cullen, Dumble and Wayside ramps exhibited lower critical gaps than the Griggs, Mossrose and Woodridge ramps. The Griggs and Mossrose ramps have taper-type acceleration lanes which terminate abruptly at overpass structures, whereas operation at the Woodridge ramp is complicated by a high grade differential between the ramp (upgrade) and freeway (downgrade).

To be certain that the variability in the critical gaps between ramps was due to differences in the ramps rather than random fluctuations in the critical gap parameter, the gap acceptance studies were repeated at a single ramp at different times. Figure 8 shows that the critical gap is a stable measure of the operational performance of a given ramp. The parameter, like capacity, is a measure of the ability of a traffic stream to absorb merging vehicles from a given ramp.

In the case of intersections, the side street traffic pattern (i.e., whether the side streets carried one-way or two-way traffic) seemed to affect the critical gap appreciably. Of course, ramps are limited to one-way operation, but ramp metering is a form of operational control which could change the minor street traffic pattern. One objective of this study was to observe some of the characteristics associated with semiautomatic metering to determine equipment requirements for future automatic ramp control systems. This phase of the study was conducted at the Dumble on-ramp. A post-mounted traffic signal with red, amber, and green lenses was installed on the frontage road at the P.C. of the on-ramp. Overhead signals mounted over each lane of the frontage road were employed to separate the two movements (ramp usage and frontage road usage). The signal phasing was designed for bulk-service metering utilizing a three-dial pre-timed controller with a 30-sec cycle length. The three dials were set to give $10\frac{1}{2}$, 8 and $13\frac{1}{2}$ sec of green with a constant amber of $2\frac{1}{2}$ sec, capable of dispatching platoons of 4, 3, and 5 vehicles per cycle.

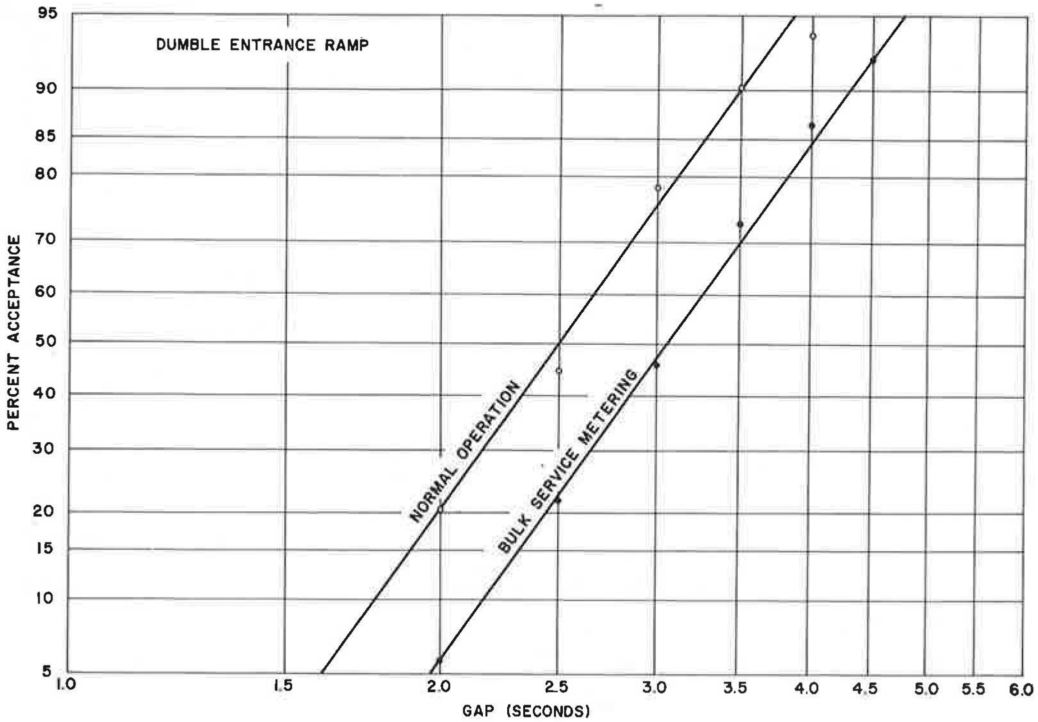


Figure 9. Comparison of percent acceptance for moving merging vehicles during normal and controlled operation.

Figure 9 shows the effect of this technique of ramp metering on the critical gap for moving vehicles. The critical gap for bulk service metering is seen to be 3.1 sec as against 2.5 sec for normal operation. The reason for this is not clear, but it is suspected that (a) metered vehicles have a greater relative speed, and (b) metered drivers are more conscious of the merging maneuver and are therefore more cautious.

Distribution of Critical Gaps

The critical gap for merging indicates how large a time interval is required for the typical ramp driver to enter a freeway. The use of a single typical figure rather than the whole range of observed human behavior has the advantage of simplicity in such advanced stages of the analysis of ramp behavior as delay calculations. In spite of the importance of the critical gap concept, very little consideration has been given to the properties of the parameter. Bissell (9) estimated the standard deviation of the gap acceptance distribution by taking the difference between the median acceptance value and the time corresponding to an acceptance of 15.9 percent. Yet, it is a well-known fact that few traffic phenomena can be described by a single measure of central tendency and dispersion, but can be described by a frequency distribution.

One technique explored in this investigation to obtain a frequency distribution for critical gaps is shown in Figure 10. Only drivers who rejected at least one gap before merging were considered. Assuming that a driver who accepts a given gap size at a certain ramp can be expected to accept any gaps of greater length and that a driver who rejects a given gap size can in a similar manner be expected to reject all gaps of shorter length, it is then evident that a driver's critical gap must lie somewhere in the range between the largest gap he rejected and the gap finally accepted. The number of times the critical gap range included a certain gap interval divided by the total number of gap intervals establishes the observed frequency.

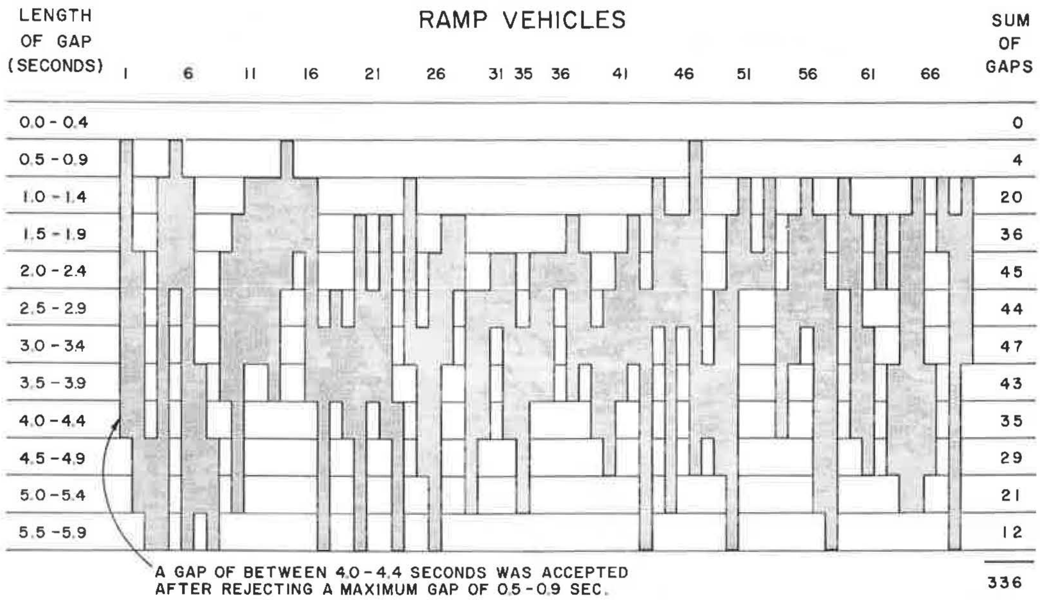


Figure 10. Limits of critical gaps for stopped vehicles at Dumble inbound ramp.

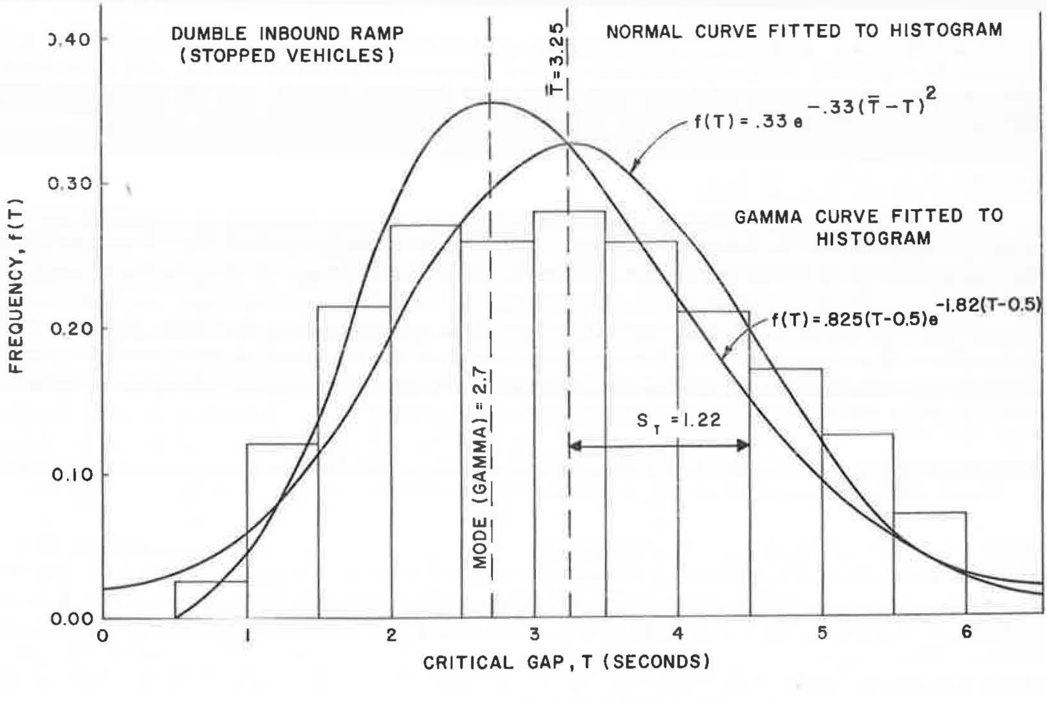


Figure 11. Comparison of observed and theoretical critical gap distributions.

The histogram of observed frequencies obtained from Figure 10 is shown in Figure 11. Since the task of gap selection is primarily a function of the human component of the composite driver-vehicle population, a logical choice in fitting a probability distribution to the observed data is the normal distribution. Since the normal distribution is completely determined by its mean and standard deviation and these quantities are rather accurately estimated from the histogram, a normal curve was easily fitted to the observed data (Fig. 11).

Although there is reasonably good agreement between the normal curve and the data, there are certain theoretical, not to mention practical, considerations which suggest a less restrictive distribution. The principal characteristics of the normal distribution are (a) it is symmetrical, (b) it assigns a finite probability to every finite deviation, and (c) the mode or most probable result is equal to the mean. Of course, possession of these three requisites is by no means general in considering vehicular traffic characteristics. It is unrealistic to think of a driver with a negative critical gap. Yet, if critical gaps were distributed normally, the second property of the distribution would assign a finite probability a negative critical gap.

Among the families of distributions formulated for the purpose of enabling the statistician to deal with a wide variety of data are those developed by Karl Pearson. Pearson (12) noted that the binomial and Poisson distributions satisfy the differential equation

$$\frac{dP}{dx} = \frac{(a + x)\,P}{b + cx + dx^2}$$

for some set of values of the constants a, b, c and d. Certain solutions of this equation have been sorted out, largely because of their algebraic simplicity, to form the Pearson frequency distributions.

Because of the non-negative restriction, we are looking for a curve limited at one end. The Pearson Type III distribution is of this type (see Appendix). Because this distribution uses the gamma function it is also known as the gamma distribution. The element of randomness inherent in this distribution, as exemplified by its formulation from the Poisson distribution, gives it some conceptional appeal for describing the critical gap phenomenon. In Figure 11, the Pearson Type III (Gamma) curve has been superimposed on the histogram along with the normal curve.

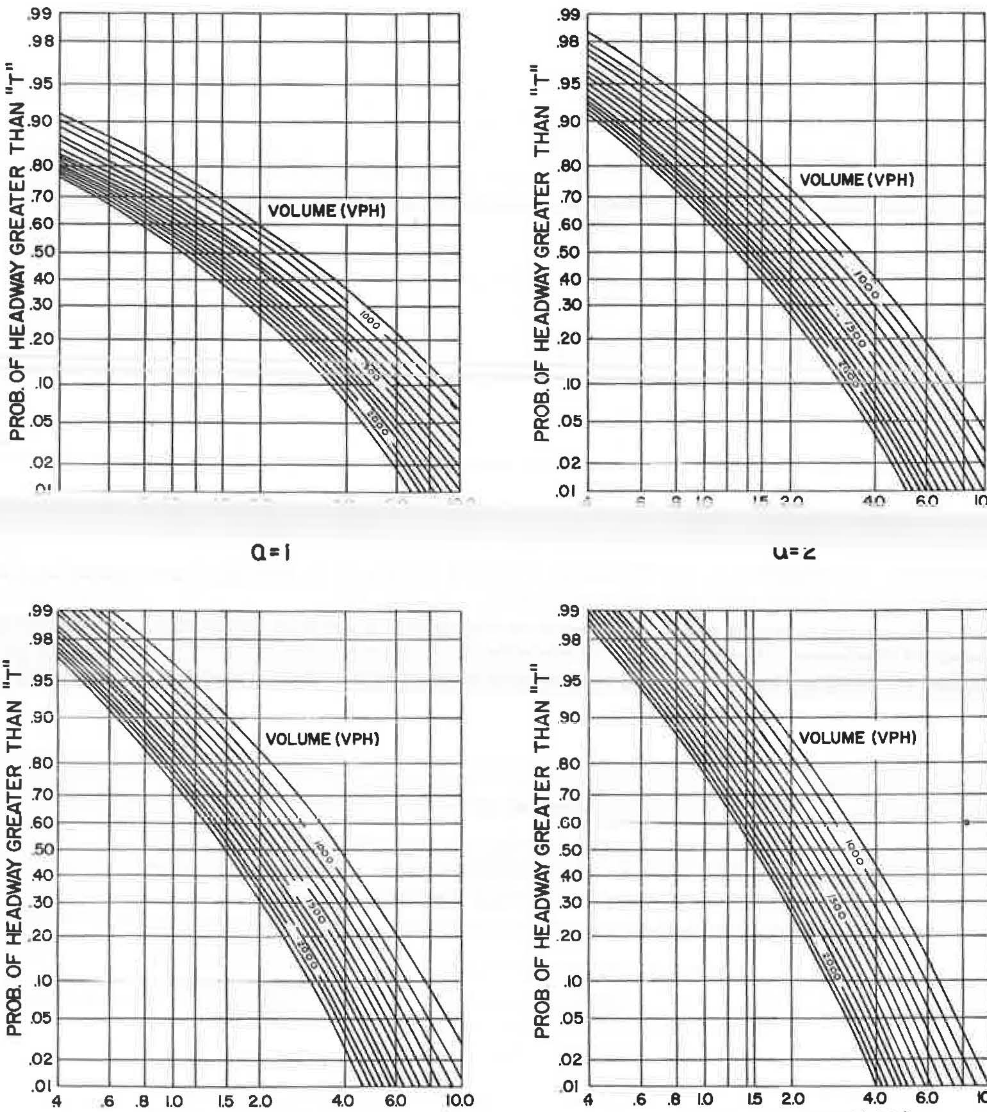
The cumulative type curves of Figures 6 to 9 suggest a second procedure for obtaining the desired frequency distribution. A distribution of minimum or critical gaps can be estimated by finding the proportion of drivers who accept a certain sized gap and the

TABLE 3

SUMMARY OF CRITICAL GAP MOMENTS AND PARAMETERS OF THE PEARSON TYPE III DISTRIBUTION APPROXIMATING OBSERVED CRITICAL GAP DISTRIBUTION

Ramp	Critical Gap Parameters (sec)			Pearson Type III Parameters (sec)		
	Median	Mean	Std. Dev.	a	b	c
Cullen	2.7	3.1	0.98	4.5	2.2	1.0
Dumble	2.8	3.2	0.85	6.6	3.0	1.0
Wayside	3.3	3.6	0.94	7.9	3.0	1.0
Griggs	3.7	4.1	1.29	4.1	1.6	1.5
Mossrose	4.7	5.2	1.57	5.5	1.5	1.5
Woodridge	3.8	4.2	1.18	9.7	2.6	0.5

proportion who accept a slightly longer gap. The difference in these proportions then gives an estimate of the proportion of drivers who will accept the slightly longer gap but not the shorter one, i.e., the proportion of drivers whose critical gap is the slightly longer gap. A summary of the critical gap distribution moments obtained using this second method is given in Table 3. The Pearson Type III distribution was fitted to these frequency distributions; the estimates of the parameters of the theoretical distribution are also summarized in Table 3.



$$\int_T^{\infty} \frac{(q\,a)^a}{(a-1)!} t^{a-1} e^{-aqt}\,dt \quad \text{WHERE } q = \text{FLOW (VPS)}$$

Figure 12. Cumulative probability curves for the Erlang distribution.

There is a subtle yet distinct difference between the frequency distributions obtained by the two methods. The frequency distribution obtained using the method of Figures 10 and 11 enables one to predict the probability of a driver having a certain critical gap. The area under the frequency distribution obtained from the cumulative distribution gives the probability of a driver accepting a certain size gap.

What is desired is a description of the critical gaps among a population of drivers. The mode, mean, and variance of the critical gap distribution add significantly to this description and therefore must be considered in the applications along with the median critical gap, as shall be shown.

DELAY CHARACTERISTICS

Gap Availability

In using mathematics to estimate delay due to merging, it is necessary to have a description of the distribution of time headways or gaps in the outside freeway lane as well as a description of the critical gap.

The Erlang frequency distribution, a special case of the Pearson Type III distribution in which the parameter, a, must be a positive integer, combines many of the advantages of the parent distribution plus simplicity in application. The cumulative form of the Pearson Type III distribution must be evaluated by either numerical methods or tables of the incomplete gamma function unless a is a positive integer, making it difficult to determine the probability of a ramp driver finding a gap larger than the critical gap, for example.

The cumulative frequency curves (Fig. 12) show the theoretical relationship between gap availability and the critical gap, assuming that the distribution of headways on the outside lane of the freeway may be described by an Erlang distribution. Although use of the Erlang distribution affords the opportunity of considering the distribution of freeway vehicles for all cases from randomness ($a = 1$) to complete uniformity ($a = \infty$), experience (2) has shown that the first four curves ($a = 1, 2, 3, 4$) are adequate for the description of vehicular headways in a traffic stream.

Determination of Merging Delays

In the theoretical approach to describing a system composed of two interacting streams, it is necessary to determine the average time for vehicles in the minor stream to merge. It may be assumed that the waiting driver measures each time gap, t, in the traffic on the major highway until he finds an acceptable gap, T, which he believes to be of sufficient length to permit his safe entry. If he accepts the first gap ($t > T$), his waiting time is zero. If he rejects the first gap ($t < T$), but accepts the second gap, his expected waiting time would be one interval. If we assume that the driver's gap acceptance policy does not change with time, then by induction, the individual waiting periods form a geometric distribution and the probability, P_n, of any driver having to wait for n intervals each less than T sec before merging is

$$P_n = p^n (1 - p), \quad n = 1, 2, \ldots, \tag{1}$$

where

$$p = P(t<T) = \int_0^T f(t)dt \tag{2}$$

and f(t) is the distribution of gaps in the major stream. The expected number of intervals for which a driver has to wait is given by

$$E(n) = \frac{p}{1-p} = \frac{\int_0^T f(t)dt}{\int_T^{\infty} f(t)dt} \tag{3}$$

The average time for a ramp vehicle which is in position to merge to find an acceptable gap in the freeway traffic stream will be the product of the expected number of intervals less than T, E(n), and the average length of interval. The average length of interval less than T is, in turn, equal to the total time less than T sec divided by the number of intervals less than T sec:

$$\text{Avg length of intervals} < T = \frac{q\int_0^T t\, f(t)dt}{q\int_0^T f(t)dt} \tag{4}$$

where q is the rate of flow. Multiplying Eqs. 3 and 4 yields the average waiting time for a ramp vehicle in position to merge:

$$d = \frac{\int_0^T t\, f(t)dt}{\int^{\infty}} \tag{5}$$

Recalling that the proportion of ramp vehicles actually delayed is given by Eq. 2, it is apparent that the average waiting time of those who incur delay is

$$d' = d \Big/ \int_0^T f(t)dt \tag{6}$$

The delays expressed in Eqs. 5 and 6 are for single vehicles approaching the merging area.

Theoretical solutions (13, 14) to the determination of delays for vehicles attempting crossing or merging maneuvers have been based on the assumption that the main stream of traffic is exponentially distributed,

$$f(t) = qe^{-qt} \tag{7}$$

Substituting Eq. 7 in Eq. 5, the mean delay d for a critical gap T and freeway lane flow q is

$$d = q^{-1} (e^{qT} - qT - 1) \tag{8}$$

If the more general Erlang distribution is used to describe the distribution of main stream gaps, then

$$f(t) = \frac{(aq)^a}{(a-1)!} t^{a-1} e^{-aqt}, \quad (a = 1, 2, \ldots) \tag{9}$$

and the mean delay for a = 2, 3 and 4 is

$$d_{a=2} = \frac{e^{2qT} - 2(qT)^2 - 2qT - 1}{q(2qT + 1)} \tag{10}$$

$$d_{a=3} = \frac{e^{3qT} - 4.5\,(qT)^3 - 4.5\,(qT)^2 - 3qT - 1}{q[4.5\,(qT)^2 + 3qT + 1]} \tag{11}$$

$$d_{a=4} = \frac{e^{4qT} - 10.67\,(qT)^4 - 10.67\,(qT)^3 - (8qT)^2 - 4qT - 1}{q[10.67\,(qT)^3 + 8(qT)^2 + 4qT + 1]} \tag{12}$$

Since the negative exponential is a special case of the Erlang distribution, $d_{a=1}$ is given by Eq. 8. Eqs. 8, 10, 11 and 12 are plotted in Figure 13.

It must be remembered that the theoretical delay values are based on a fixed critical gap for all drivers. Blunden (11) correctly suggests that a more complete description of delays to traffic would be given if the fixed critical gap were replaced with a distribution of critical gaps, g(T). If it is assumed that the critical gap distribution is of the gamma type

$$g(T) = \frac{b^a}{(a-1)!} T^{a-1} e^{-bT} \tag{13}$$

a reasonable assumption as seen by Figure 11, then the mean delay D is readily obtainable from

$$D = \int_0^\infty d(T)\; g(T)dT \tag{14}$$

Substituting the delay function given by Eq. 8 and the critical gap distribution given by Eq. 13 in Eq. 14 yields

$$D = q^{-1} \int_0^\infty e^{qT} g(T)dT - \int_0^\infty Tg(T)dT - q^{-1} \int_0^\infty g(T)dT \tag{15}$$

Realizing that the second term is the form for the mean of a distribution and the integral in the last term must equal unity, gives

$$D = q^{-1} \int_0^\infty e^{qT} g(T)dT - \bar{T} - q^{-1} \tag{16}$$

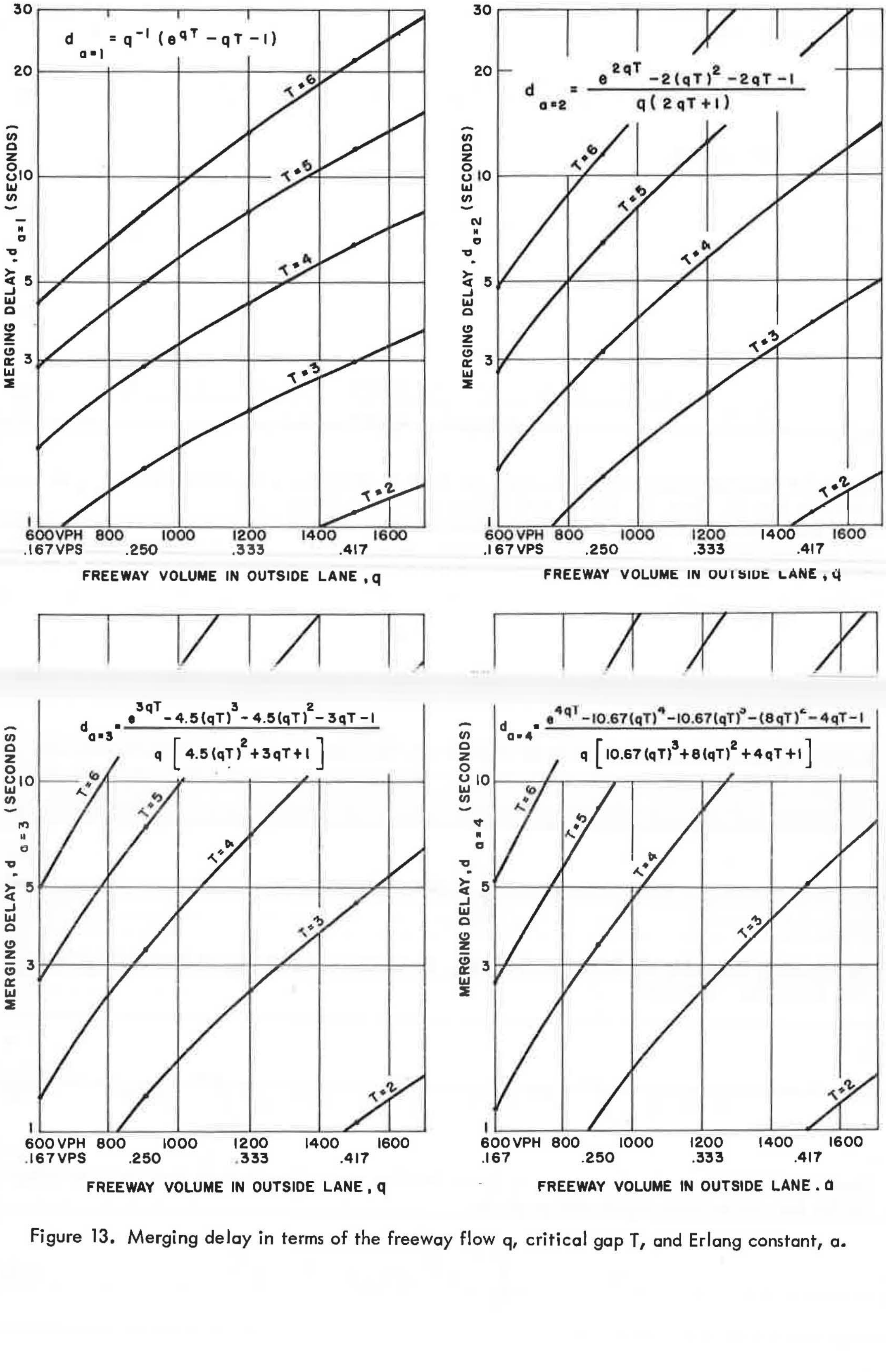


Figure 13. Merging delay in terms of the freeway flow q, critical gap T, and Erlang constant, a.

Letting b = a/T, the first term may be put in the form

$$\frac{(a/\bar{T})^a}{q\left(\frac{a-q\bar{T}}{\bar{T}}\right)^a} \int_0^{\infty} \left(\frac{a-q\bar{T}}{\bar{T}}\right)^a \frac{T^{a-1}}{(a-1)!} e^{-(a-q\bar{T})\,T/\bar{T}}\, dT \tag{17}$$

$$= \frac{1}{q}\left(\frac{a}{a-q\bar{T}}\right)^a \tag{18}$$

provided that $a > q\bar{T}$, since the integral represents the area under a gamma distribution with parameters a and $(a - q\bar{T})/\bar{T}$. Gathering terms, the general expression for delay to a merging vehicle is

$$D = \frac{1}{q}\left(\frac{a}{a-q\bar{T}}\right)^a - \bar{T} - \frac{1}{q} \tag{19}$$

Equation 8 can be verified as the special case of Eq. 19 in which the critical gap is taken as a fixed value equal to the mean critical gap. For $a = \infty$, we know that the variance of the gamma distribution is zero; so we may interpret $g(T; u, \infty)$ as the

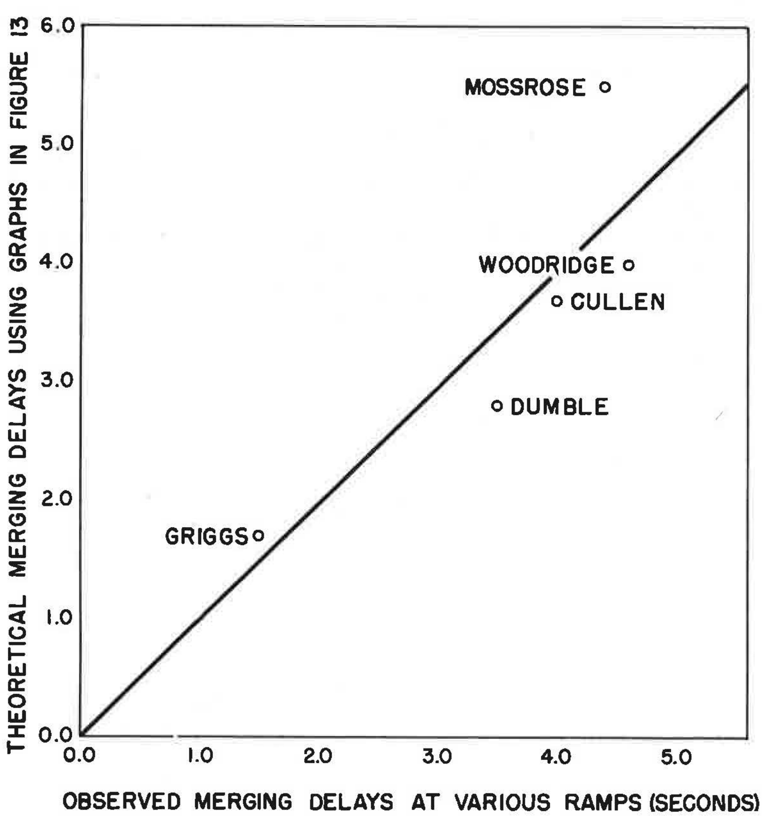


Figure 14. Comparison of observed and theoretical merging delays at study ramps.

family of situations for which the critical gap is constant and has the value $\bar{T}$. From the definition of e it is apparent that

$$\lim_{a \to \infty} \left(1 - \frac{q\bar{T}}{a}\right)^{-a} = e^{q\bar{T}} \tag{20}$$

Substitution of Eq. 20 in Eq. 19 gives Eq. 8.

Delay values calculated from Eq. 19 using finite values of a will always be higher than those obtained from Eq. 8 assuming a fixed critical gap. Moreover, the fact that the delay obtained using the fixed value is the special case of the delay obtained using the distribution for a variance of zero proves that the mean of the critical gap distribution, not the median, should be used in delay computations.

Observed average merging delays were compared to the average merging delays predicted by the equations, using the mean critical gap (Fig. 14). The value of a for the Erlang distribution used to enter the graphs in Figure 13 was calculated from the time headways in the outside lane using the method of moments in which a is the integer closest to the quotient obtained by dividing the mean headway squared by the variance in the headways. The merging delay concepts discussed in this section become important as inputs in a queuing model developed in the next section.

Queuing Considerations

A queuing system is composed of three elements: (a) a demand or flow of arrival requiring service, (b) some restriction on the availability of service, and (c) irregularity in either the demand or in the servicing operation or in both.

Consider vehicles on an entrance ramp arriving at the merging area at a rate described by f (q_r) where q_r is the average rate of arrivals. These vehicles are obliged to yield to the freeway traffic, thus forming a single lane waiting for successive vehicles at the head of the line to merge. If the distribution of time spent by vehicles at [illegible]

[illegible] dall's (15) approach for a queuing system with random inputs and arbitrary service times, formulas are developed for the mean queue length and mean waiting time for a queue of ramp vehicles waiting to merge.

Let n_0, n_1 be the ramp queue lengths immediately after two successive ramp vehicles C_0, C_1 have merged, t be the service time of C_1, r be the number of ramp vehicles arriving while C_1 is being served. If a random variable δ is introduced such that $\delta = 1$ if $n_0 = 0$ and $\delta = 0$ if $n_0 \neq 0$, then it follows that

$$n_1 = n_0 + r - 1 + \delta \tag{21}$$

The definition of δ shows that

$$\delta^2 = \delta$$

and

$$n_0 \delta = 0,$$

and hence, from Eq. 21, on taking expected values,

$$E(n_1) = E(n_0) + E(r) - 1 + E(\delta). \tag{22}$$

If the system is assumed to be in a state of statistical equilibrium, $E(n_1) = E(n_0)$ and

$$E(r) = q_r E(t) = q_r/\mu = \rho$$

Thus substituting in Eq. 22

$$E(\delta) = 1 - \rho$$

Squaring both sides of Eq. 21 and taking expected values as before,

$$E\,(r-1)^2 + E(\delta^2) + 2E\,[n_0\,(r-1)] + 2E\,[\delta\,(r-1)] = 0$$

which reduces to

$$E(n_0) = \rho + \frac{E(r^2) - \rho}{2(1-\rho)} \tag{23}$$

It is now necessary to calculate $E\,(r^2)$, the second moment of the number of arrivals in the service time T, making use of its relationship to the mean and variance in arrivals. Assuming that ramp arrivals are Poisson and remembering that averaging here must be carried out with respect to both r and the service time t,

$$E(r^2) = q_r\,E(t) + q_r^2\,E\,(t^2)$$

Since $E(t) = \mu^{-1}$ and

$$E\,(t^2) = \sigma^2 + E\,(t)^2$$

then

$$E\,(r^2) = \rho + \rho^2 + q_r^2\,\sigma^2 \tag{24}$$

Substituting Eq. 24 in Eq. 23 gives the expected queue length on the ramp as

$$E\,(n)_r = \frac{q_r}{\mu} + \frac{\mu q_r^2\,(\mu^{-2} + \sigma^2)}{2(\mu - q_r)} \tag{25}$$

If w is the waiting time (before merging) of C_1, then n_1 ramp vehicles arrive in time t + w. Thus since the mean arrival rate is q_r,

$$E(n)_r = q_r\,E\,(t + w)$$

TABLE 4

SUMMARY OF SERVICE TIME MOMENTS AND PEARSON TYPE III PARAMETER "a"

Ramp	Service Time Moments		Type III "a" Value
	Mean	Std. Dev.	
Cullen	4.0	5.2	0.6
Dumble	3.5	5.5	0.4
Wayside	6.2	8.8	0.5
Griggs	1.5	2.8	0.3
Mossrose	4.4	6.2	0.5
Woodridge	4.6	6.0	0.6

It follows that the average waiting time for a ramp vehicle before merging is

$$E(w) = [E(n)_r/q_r] - \mu^{-1} \tag{26}$$

and the mean wait in the system for a ramp vehicle is

$$E(v) = E(n)_r/q_r \tag{27}$$

Major and Buckley (16) have interpreted the service time for the queue μ^{-1} as identical to the summation of the rejected gaps for a ramp vehicle in position to merge (what we have referred to as the merging delay d in this paper). The Pearson Type III distribution was fitted to the observed distribution of delays at each ramp and the results are given in Table 4 for all ramps in the study. Based on the assumptions that the average service time is equal to the average merging delay and the distribution of service times is of the form of the Pearson Type III distribution, we obtain from Eq. 25

$$E(n)_r = dq_r + \frac{(dq_r)^2 (1 + a^{-1})}{2(1 - dq_r)} \tag{28}$$

where d is given by Eq. 8, 10, 11 or 12 depending on the distribution of freeway headways and a is the parameter describing the distribution of service times.

If the a value in Eq. 28 calculated in Table 4 is rounded off to unity, Eqs. 25-28 take the form of the conventional, Poisson-negative exponential, queuing formulas.

APPLICATIONS

Ramp Control

Almost any engineering problem may be described as a systematic attempt to re-

build enough strength into the materials of a layered pavement system, for example, to withstand shear stresses due to anticipated loads. However, the mere fact that the strength (capacity) exceeds the load stresses (demand) does not guarantee an acceptable level of service. The deflection, smoothness, texture and color contrast also affect the driver's ride and, as such, are level of service factors that must be considered.

Traffic engineering is the science of measuring traffic characteristics and the application of this information to the design and operation of traffic systems. The traffic engineer's basic problem of resolving a capacity-demand relationship is similar to that of any other engineer: he must be able either to measure the parameters defining capacity and demand very accurately, or to control them. Returning to the pavement design analogy: although the strength of the materials in a pavement can probably not be estimated as accurately as the capacity of a freeway lane, the pavement designer knows that the loads (demand) on the facility are controlled, and in most states limited by law. If urban freeways are to operate at the levels of service for which they were designed, the demand on these facilities must also be regulated.

Because the control of vehicles entering the freeway, as against the control of vehicles already on the freeway, offers a more positive means of preventing congestion, considerable emphasis is being placed on the technique of ramp metering. Any scheme for controlling the rate of flow of vehicles from an entrance ramp onto the freeway should be based on, and be capable of, reacting to operation on the freeway lanes. Chicago (17) utilizes an occupancy (density) measurement in the middle freeway lane in advance of the merging area. Wattleworth (18) proposes a systems approach in which the capacity of a freeway system is limited to the section of smallest capacity (bottleneck). If the bottleneck capacity is known, all ramps in the system can be metered according to the differential between the upstream freeway demand and bottleneck capacity.

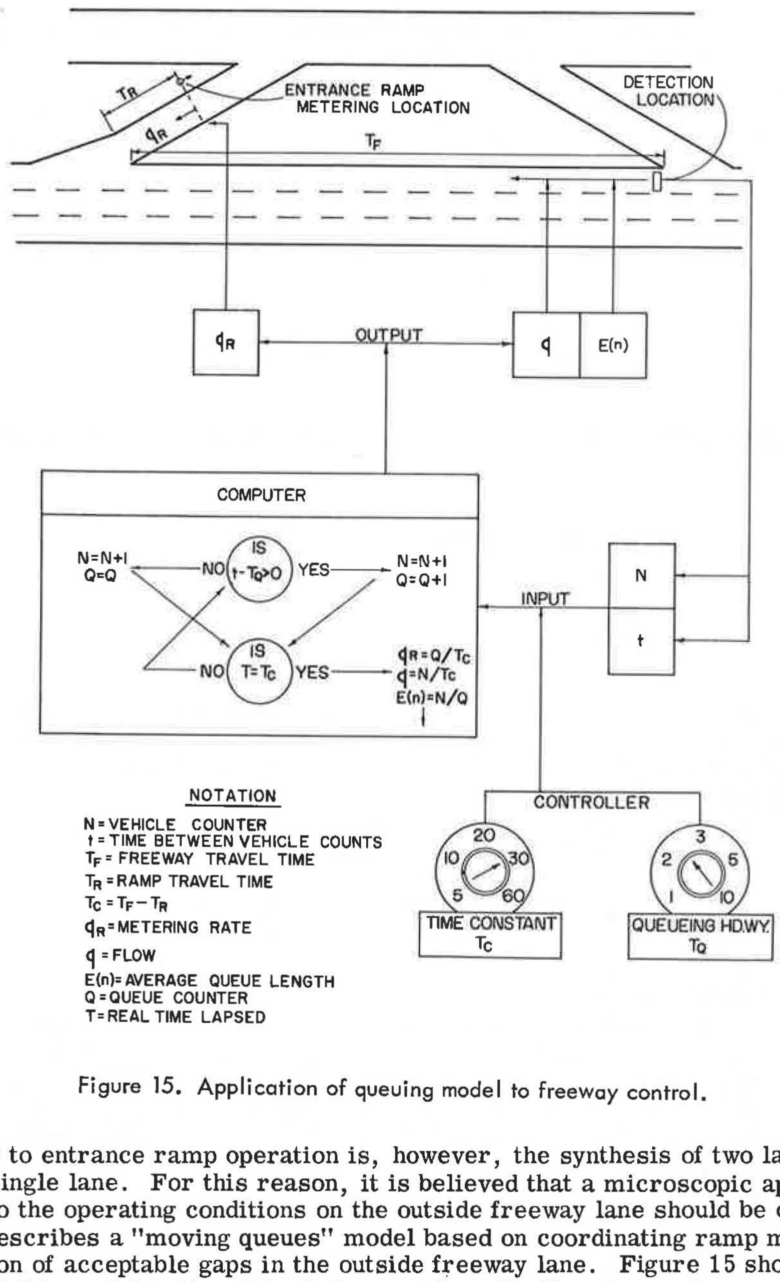


Figure 15. Application of queuing model to freeway control.

The key to entrance ramp operation is, however, the synthesis of two lanes of traffic into a single lane. For this reason, it is believed that a microscopic approach sensitive to the operating conditions on the outside freeway lane should be considered. Drew (2) describes a "moving queues" model based on coordinating ramp metering with the detection of acceptable gaps in the outside freeway lane. Figure 15 shows the application of this model to the control of a merging situation at an entrance ramp. Thus, a vehicle in the outside lane of the freeway with a time headway less than the arbitrary queuing headway T_Q is considered to be queued to the preceding vehicle. The control system illustrated consists of the flow of information from a detector located on the outside freeway lane to a computer and then to the metering signal on the ramp. For the closed loop system pictured, either a digital or analog computing device could be utilized. However, use of the former would necessitate a reduction in the "time constant" (time over which traffic conditions are averaged) by an interval equal to the time necessary for computation.

In considering an example, suppose the travel time during the peak period from detector to merging area is T_F = 35 sec and from the metering station to the merging

area is $T_R = 5$ sec. The critical gap (that headway in the outside freeway lane for which an equal percentage of ramp traffic will accept a smaller headway as will reject a larger one) is assumed to be 2.5 sec. If control adjustment is to be made during the same period as detection, the time constant T_c cannot be greater than $T_F - T_R$. Moreover, if the arbitrary queuing headway T_Q is equated to the critical gap, the number of platoons or moving queues Q will equal the number of critical gaps. The latter determines q_r, the number of ramp vehicles that can merge during T_c. Thus, in the example the dials on the controller would be set to $T_c = 30$ sec and $T_Q = 2.5$ sec. If during T_c, $N = 10$ vehicles were detected and $Q = 5$ of the headways were greater than the queuing headway on the dial ($t > T_Q$), the metering rate during T_c would be:

$$q_r = \frac{Q}{T_c} = \frac{5 \text{ veh}}{30 \text{ sec}} = 1 \text{ veh every 6 sec}$$

The rate of flow and congestion index at the detection station during T_c would be:

$$q = \frac{N}{T_c} = \frac{10 \text{ veh}}{30 \text{ sec}} = 1 \text{ veh every 3 sec}$$

$$E(n) = \frac{N}{Q} = \frac{10 \text{ veh}}{5 \text{ queues}} = 2 \text{ veh/queue}$$

The variability of the critical gap among entrance ramps (Fig. 7) shows that controller settings would vary from entrance ramp to entrance ramp. The geometrics of the freeway would also affect the detector location and hence the time constant.

The significance of the control parameter E(n) is twofold: (a) by definition, it is [illegible] approach infinity); and (b) the moving queue length, E(n), was formulated in such a way that it is the reciprocal of the probability of getting a gap larger than the queuing criteria, and therefore is actually a measure of gap availability.

Comparison Ramp Metering Criteria

An important contribution of this microscopic approach to ramp control is that the individuality of entrance ramps is taken into consideration. Figure 16 shows the relationship between the freeway outside lane volume, q, the headway distribution of the freeway volume, a, the critical gap, T, and the ramp volume, q_r. Consider an entrance ramp operating with a critical gap of 4.0 sec and with the distribution of freeway traffic conforming to an Erlang distribution with $a = 2$. The sum of the coordinates of any point on the line $T = 4$ in the graph $a = 2$ describes the merging service volume for that ramp. For example, the point described by $q = 1,500$ and $q_r = 120$ tells us that the merging service volume is 1,620, and that under these operating characteristics, a ramp arrival has a 67 percent chance of finding the ramp empty or a 33 percent chance of finding a vehicle ahead of it trying to merge.

The effect of poor ramp geometrics is evident. If in the previous example the critical gap was 5.0 sec, the maximum ramp service volume q_r for the same freeway volume ($q = 1,500$) would be 50 vph or a total merging service volume for the ramp of 1,550 vph.

The graphs also point out the differences in the macroscopic philosophies of ramp metering (17, 18) and the microscopic approach explained in this paper. In the macroscopic approach, metering would be based on one of the curved lines (one representing the boundary between stable and unstable flow and the other representing possible capacity) regardless of the ramp geometrics or critical gap. This means that, for all conditions except those described as unstable flow on the graph, vehicles would be

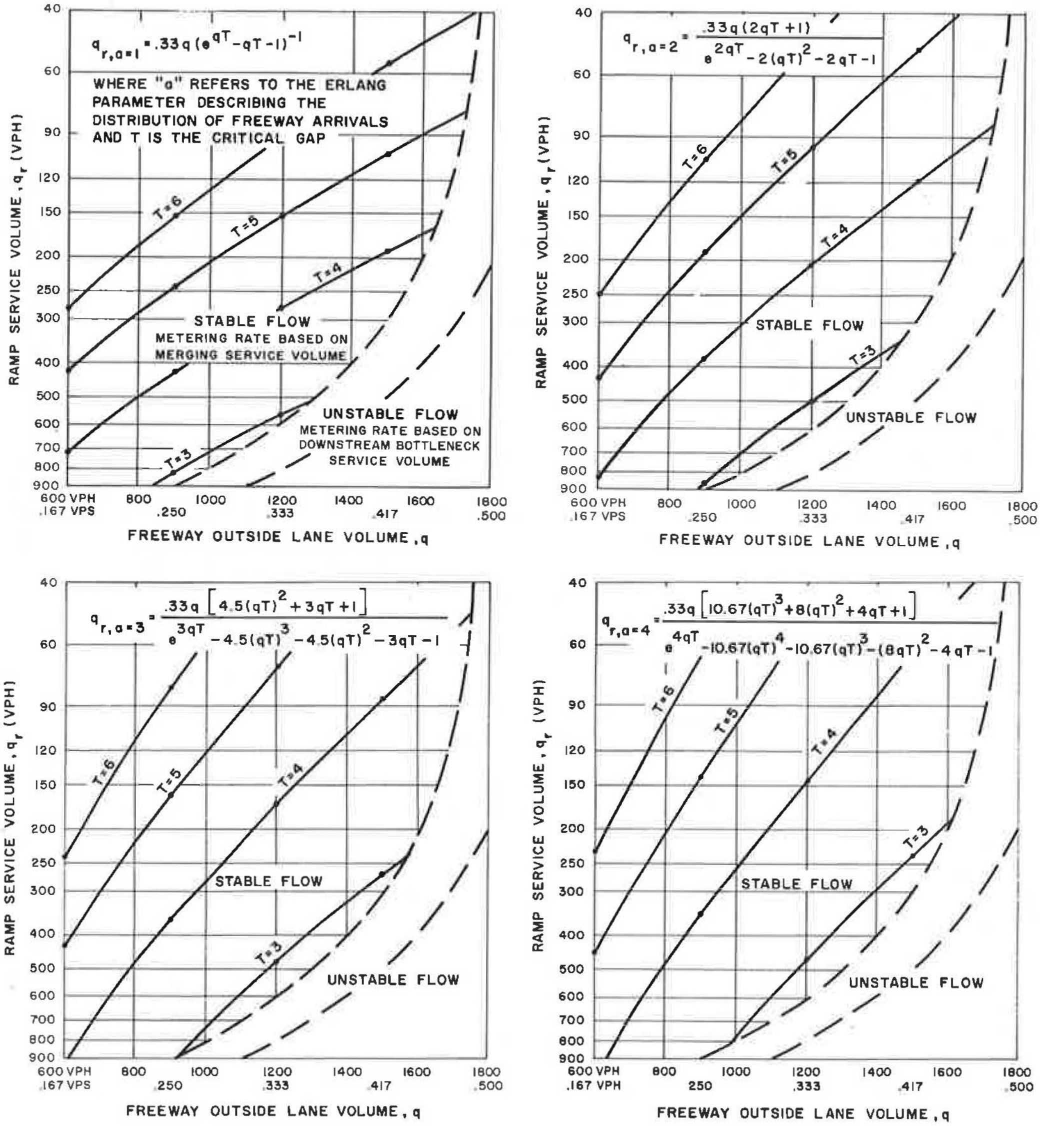


Figure 16. Maximum service volume q_r for a ramp arrival to have a probability of 0.67 of finding no ramp vehicles in merging area.

metered at a faster rate than the service rate at the merging area (available critical gaps) encouraging drivers to either accept smaller gaps than the critical gap or become part of a steadily growing queue at the merging area.

Figure 16 shows the need for a ramp control technique combining both the macroscopic and microscopic approach. For conditions described by "ustable flow," the ramp geometrics do not govern and hence the macroscopic approach based on the downstream bottleneck service volume applies. However, to the left of the 1,800-vph line dividing stable and unstable flow, the critical gap governs since the merging service volume is less than the bottleneck service volume. (It should be explained that the term "service volume" is used here consistent with the level of service concept proposed in the 1965 Highway Capacity Manual, although the word capacity could be substituted in this particular comparison with only the risk in a loss of generality.)

Ramp Design

After the determination of the number of freeway lanes in a rational design procedure, the operating conditions at critical locations of the freeway must be investigated for the effect on capacity and level of service. Unless some designated level of service is met at every point on the freeway, bottlenecks will occur and traffic operation will break down. Critical locations on a freeway are manifest by either sudden increases in traffic demand, the creation of intervehicular conflicts within the traffic stream, or a combination of both.

Entrance ramps represent the third and most serious case since they create two potential conflicts with the maintenance of the adopted level of service of a roadway section. First, the additional ramp traffic may cause operational changes in the outside lane at the merge. This condition, of course, will be aggravated by any adverse geometrics, such as high angle of entry, steep grades, and poor sight distance. Second, the additional ramp volume may change the operating conditions across the entire roadway downstream from the on-ramp. This is particularly true where there is a downstream bottleneck.

The three basic procedures employed in checking capacity for the design of entrance ramps are based on the same philosophies discussed for metering ramps. One method is based on preventing the total freeway volume upstream from the ramp plus the entrance ramp volume from exceeding the capacity of a downstream bottleneck. A second method (19) takes into consideration the distribution of freeway volumes per lane and then limits the ramp volume to the merging capacity less the upstream volume in the outside lane. The third method states that the ramp capacity is limited by the number of gaps in the shoulder lane which are greater than the critical gap for acceptance.

Figure 16 can be useful in the implementation of all three approaches. Thus, if a ramp on a new facility is of a high-type geometric design guaranteeing a low critical gap, methods 1 and 2 are applicable since the merging service volume will exceed any bottleneck service volume. If, however, due to the terrain, spacing of interchanges [illegible]

Freeway design, like most real world phenomena, involves a series of compromises. Because of the spacing of interchanges on many urban freeways, the fulfillment of desirable entrance ramp design, desirable exit ramp design, and an adequate weaving section between them, offers a dilemma. The alternatives are (a) reduction in the standards of one or more of the features (b) elimination of one of the features (such as one of the ramps), or (c) transferring the weaving from the freeway to the frontage road. These alternatives should be evaluated in terms of their cost, their effect on the freeway and ramp operation at that location, and their effect on adjacent facilities such as adjacent interchanges, and cross-street signalization. The procedure exemplified by Figure 16 enables a designer to evaluate alternatives more rationally and if compromise is needed, to select the element or location where it will be the least objectionable.

SUMMARY AND CONCLUSIONS

Many researchers (4, 5, 9, 10) have defined a parameter called the critical gap to describe the operation of two interacting traffic streams. The critical gap, as a time gap just as likely to be accepted as to be rejected, is by implication a median gap. The advantages of this parameter for describing entrance ramp operation are (a) it provides a single typical figure rather than the whole range of observed gap acceptance characteristics, (b) it gives simplicity in such advanced stages of ramp behavior as delay calculations, (c) it is sensitive to variations in geometrics and, therefore, provides a rational means of evaluating ramp design, and (d) like such parameters as capacity, optimum speed and optimum concentration, it is independent of traffic volumes and is, therefore, invariant for a given location under a given set of environmental and operating conditions. These concepts are verified in this report.

Of particular significance is the development of a distribution of critical gaps which enables a researcher to predict the probability of a driver having a certain critical

gap. Delay values calculated using this distribution (Eq. 19) will always be higher than those calculated assuming that all drivers have the same fixed critical gap (Eq. 8). The fact that the delay obtained using the fixed value is the special case of the delay obtained using the distribution for a variance of zero proves that the mean of the critical gap distribution, not the median, should be used in delay computations.

The delay equations (Eqs. 8, 10, 11, 12) developed in this study are based on the assumption that the distribution of freeway headways is of the form of the Erlang frequency distribution. This represents a generalization of past work (5, 11, 13, 14, 16) in which the distribution of freeway headways was assumed to be of that particular form of the Erlang distribution called the negative exponential distribution.

The distribution of time spent by merging ramp vehicles at the head of the queue may be approximated by a gamma distribution, suggesting that an entrance ramp merging operation may be considered within the context of classical queuing theory. Based on this queuing model the ramp metering curves of Figure 16 were conceived. The metering technique suggested combines a macroscopic and a microscopic approach, the former based on a controlling downstream bottleneck, the latter on the merging area.

In conclusion, it is apparent that additional theoretical research is needed to provide a more complete relationship between the many variables associated with the interaction of vehicles traversing a ramp and merging into a freeway so as to determine the effects of traffic characteristics, merging area geometrics, and environmental elements on merging capacity and operation. This information would be of profound significance in the operation and control of existing facilities, the design of future facilities, and the development of usable distributions of traffic variables for simulation programs.

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Appendix

THE PEARSON TYPE III DISTRIBUTION

A random variably y is said to be distributed as the Type III distribution if its density is

$$f(y) = \frac{b^a}{\gamma(a)} y^{a-1} e^{-by} \; ; \; 0 < y < \infty \qquad (29)$$

where γ (a) is called the gamma function and is defined by the formula

$$\gamma(a) = \int_0 z^{-\,-} e^{-} dz$$

To apply the Type III distribution to traffic phenomena such as space headways in which the distribution curve does not go through the origin (the space headway between successive vehicles in the same lane can never be zero because vehicles possess length), it is necessary to translate the distribution c units from the origin.

Recognizing that the area under f(y) is unity, we obtain the desired generalized distribution by substituting x = y + c for y in Eq. 29:

$$\int_0^\infty f(y)dy = \int_c^\infty \frac{b^a}{\gamma(a)} (x-c)^{a-1} e^{-b(x-c)}$$

giving

$$f(x) = \frac{b^a}{\gamma(a)} (x-c)^{a-1} e^{-b(x-c)} \; ; \; c < x < \infty \qquad (30)$$

Taking moments about c,

$$\mu_k^c = \frac{b^a}{\gamma(a)} \int_c^\infty (x-c)^{k+a-1} e^{-b(x-c)} dx$$

$$= \frac{b^a}{\gamma(a)} \frac{\gamma(k+a)}{b^{k+a}} \int_c^\infty \frac{b^{k+a}}{\gamma(k+a)} (x-c)^{k+a-1} e^{-b(x-c)} dx$$

$$= \frac{b^a}{\gamma(a)} \frac{\gamma(k+a)}{b^{k+a}} \tag{31}$$

since the integral represents the area under a Type III curve and must equal unity. The first moment about c_1 is obtained by letting $k = 1$ in Eq. 31

$$\mu_1^c = \frac{a}{b} \tag{32}$$

Adding c to Eq. 32 gives the mean of this distribution

$$\mu = \frac{a}{b} + c \tag{33}$$

The variance is given by

$$\sigma^2 = \mu_2^c - (\mu_1^c)^2 = \frac{a}{b^2} \tag{34}$$

Discussion

PAUL D. CRIBBINS, Associate Professor of Civil Engineering, North Carolina State University, Raleigh—The author is to be commended for his careful investigation of gap acceptance characteristics which comprises a welcome and valuable addition to the literature on capacity-demand relationships on urban freeways. In particular, consideration of the entrance ramp merging operation within the context of classical queuing theory signifies an important step in the evolution of theoretical research related to the ramp-freeway merging maneuver.

This research endeavor, however, would appear to reinforce the implication that a formidable communications barrier exists between the traffic flow theorist and the practitioner faced with problems requiring immediate solutions. An appreciable gap in understanding between these two groups is apparent and efforts must be made by responsible transportation engineers if we are to maximize the utilization of flow theory findings in the real world of traffic operations. In order to narrow the gap between the theorist and the practicing engineer a better appreciation by the flow theorist of the day-to-day problems of the traffic engineer and a greater effort on the part of the latter to obtain and study theoretical findings related to his particular problem are of paramount importance.

Particular consideration has been given in this paper to the design and conduct of an experimental study to validate theoretical models that will eventually be used in an automatic surveillance and control system for the Gulf Freeway in Houston, Texas. Without major design revisions to the freeway, the surveillance and control system ideally will permit an acceptable level of service to be maintained on the freeway during peak demand periods.

Previous operational studies on other freeways have indicated a need for some form of control of the vehicles entering the freeway from the on-ramps during peak demand periods. The first obvious solution to the ramp control problem was complete closure of critical on-ramps, which was often followed by almost complete stagnation of traffic

on the streets surrounding the freeway. Probably the next significant step in the evolution of ramp control techniques was made at the Illinois Expressway Surveillance Project in Chicago where a traffic-adjusted metering scheme was initiated in 1963. If we assume, and I think we must, that the primary objective of automatic ramp metering is to maximize flow on the freeway or expressway at the expense of flow on the surrounding street system, then the initial experience on the Illinois project was very satisfactory. However, recent observation by this writer of peak-period conditions at the demonstration area of the Illinois project indicated that congestion on the arterial streets leading to the on-ramps may become intolerable when the ramp volumes are restricted for prolonged periods.

While it may be argued that such a condition, even though undesirable, is inevitable, there is growing evidence that research in metering and controlling ramp traffic must encompass a comprehensive approach to the total capacity-demand of the freeway and the existing street network treated as a single system. Such an undertaking, even though it involves the expenditure of considerable money and energy, has tremendous potential, and research personnel on the Gulf Freeway Surveillance Project have wisely addressed themselves to the problem in this manner. In doing so they have become aware, and hopefully will make many more of us who are interested in improved traffic flow aware, of the need for a better understanding of the basic freeway characteristics and control parameters.

In attempting to measure delay characteristics at the merging area of the entrance ramps, this investigation fulfills at least two needs. These include the provision of gap acceptance data for ramp simulation inputs and the accumulation of information for calibrating ramp metering models. Perhaps of greater significance, however, is the definition and supporting case that is built for use of a parameter called the "critical gap."

The probability of a driver having a particular critical gap is facilitated by the development of a distribution of critical gaps, and the fact that the mean of the critical gap rather than the median should be used in delay computations is amply supported

ways follows the form of the Erlang distribution rather than a more specific form, the negative exponential distribution. This would corroborate earlier work in this area by Haight who has indicated that, while the utilization of a generalized Poisson distribution is satisfactory if the arrival times of vehicles are randomly distributed, many vehicles that appear to have gap distributions in the intermediate area between random and equally spaced distributions can best be described by a Pearson Type III distribution. The author has gone a step further by fitting the Pearson Type III distribution to the observed distributions of delays at each of the observed ramps of the Gulf Freeway and developing an equation for the expected queue length on the ramp.

The major criticism that could be leveled at this paper is very likely beyond the scope of the investigation but concerns its failure to suggest the potential cost savings that could accrue to the motorist from the eventual refinement of a more sophisticated surveillance and control system. Nevertheless, this study of gap acceptance characteristics is a major step toward the dual goals of moving high volumes of traffic at acceptable levels of service and coincidentally offsetting the cost of the control system by increasing the road-user benefits to all drivers on the system.

R. F. DAWSON, Assistant Professor, University of Connecticut, Highway Research Project, Storrs—Gap acceptance characteristics, gap characteristics and merging delay characteristics are essential in the development of ramp metering systems. There is an absolute need for research to formulate general models of these and other essential descriptors of the microaspects of the ramp-freeway queuing system. The work by Drew is a sound initial approach to the problem. There are, however, some aspects that have perhaps been oversimplified. The following discussion will be con-

fined to several of Drew's assumptions in the development of a ramp queuing model without consideration for the application of the proposed queuing model to ramp metering.

Main Components of Drew's Queuing Model

Gap Acceptance Characteristics.—Only a limited amount of gap acceptance research has been conducted, and as a consequence the phenomenon is not well understood. Drew's method for plotting the density function for critical gaps (it is not applicable to lags) is perhaps the best technique that has been proposed to date. The individual observations for which a density function is constructed consist of time range delimited by the largest gap rejected and the gap finally accepted. It is obvious that a driver's critical gap must fall in this range. Drew was successful in fitting an Erlang (a = 5) function (special case of the gamma function) to the density plot, but he presented data from just one ramp situation as evidence of the adequacy of the Erlang function as a general density model.

It will probably be necessary to give some consideration to the development of a technique for weighting individual critical gap density function observations obtained by the proposed Drew method. By their very nature, observations of narrow range would delimit a driver's critical gap more closely, but in the analysis by Drew, observations with wider ranges have greater influence on the shape of the density function. Two simple weighting procedures that might alleviate this undesirable property seem apparent. In the first such procedure individual observations could be given equal weight by recording each observation in the single 0.5-sec interval containing the midpoint of the range, on the assumption that the midpoint is the best estimate of a driver's critical gap. A second approach to weighting the observations would involve the application of a weighting factor that is inversely proportional to the length of the time range. The latter technique has been applied to Drew's data for the Dumble Street on-ramp, and the results are plotted in Figure 17. The Erlang function which has been fitted to

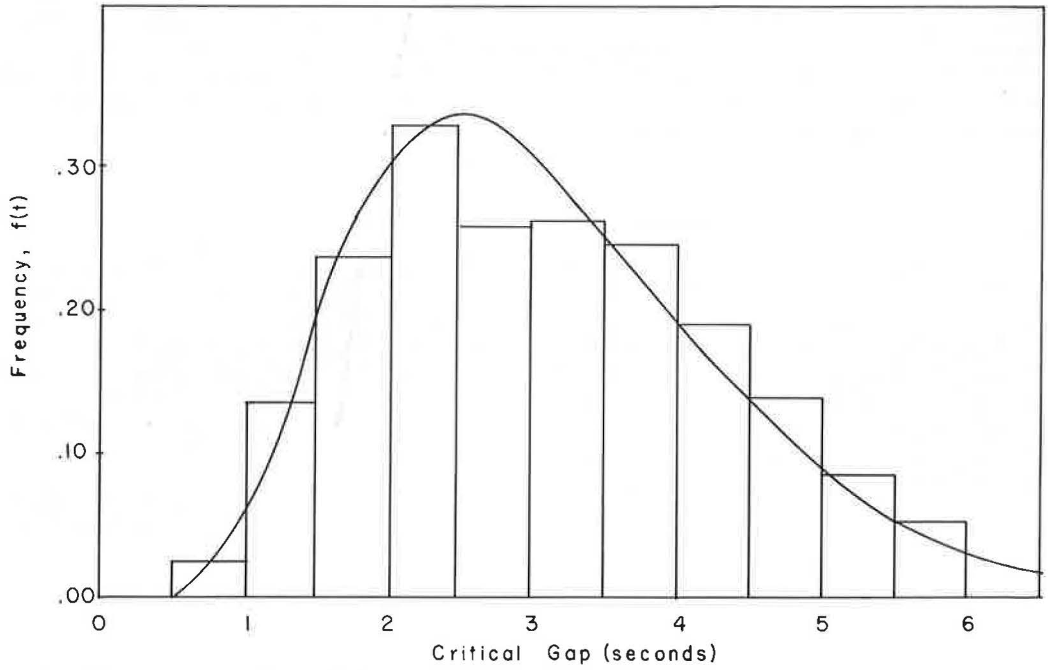

Figure 17. Comparison of weighted critical gap distribution with Erlang density function (a = 4).

these results [an Erlang (a = 4) function with a mean of 2.63 sec and a minimum of 0.50 sec] appears to be a much better fit than that presented by Drew in Figure 11.

Drew also describes a procedure, combining some of the features of the Raff method and some of the features of the Bissell method, that is applicable to the analysis of gap acceptance by both stopped and moving vehicles. The "probability of accepting a gap size t" as computed by this combination procedure, however, is a distinctly different statistic than the "probability of accepting a gap size t" as computed from the cumulative distribution of critical gaps obtained in the proposed Drew procedure. These two statistics do not even appear to have density functions, or cumulative density functions, from the same general family. This is evidenced by a comparison of plots of the two statistics on a standardized graph used for analysis of cumulative functions assumedto have Erlang density functions. If the cumulative function (in this case the probability of accepting a given gap) follows a given a plot on the standardized graph, the density function is an Erlang function of the order a. Failure of the observed cumulative function to follow any a plot is an indication that the density function is not of the Erlang family. The cumulative density function for stopped vehicles, obtained by the Drew method and plotted in Figure 18, is apparently a cumulative plot of the area under an (a = 4) or an (a = 5) function. Plots of the probabilities that given gaps are acceptable, as computed by the Raff-Bissell method, are shown in Figure 19 for stopped vehicles. Since this latter plot does not match an a line on the standardized graph, it can be concluded that the density function for critical gaps implied by this plot is not of the Erlang family.

Gap Characteristics.—As stated by Drew, it is necessary to have a description of gaps in the outside freeway lane, as well as a description of the critical gap distribution, in order to develop a mathematical model for estimating delay due to merging. Drew again proposes that the Erlang function might be the appropriate descriptor. The Erlang distribution, however, is a limited model; it would be useful for describing the availability of gaps in the freeway stream if it could be demonstrated that the distribu-

evidence has been presented, however, indicating that distributions of freeway vehicles tend to range from random distributions to what might be called hyper-random (more than random) distributions. Gap distributions presented in the Highway Capacity Manual are adequately described by a negative exponential (or random) distribution with a shifted axis. In their more recent research studies, Schuhl, Greenshields and May have found that the hyper-exponential distribution is a good descriptor of distributions of time gaps between freeway vehicles.

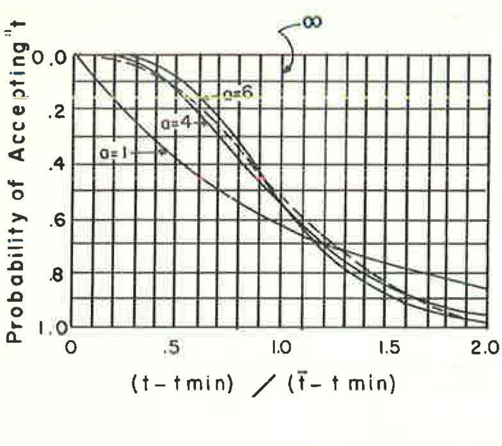

Figure 18. Comparison of drew probability of acceptance with standardized Erlang functions (stopped vehicles).

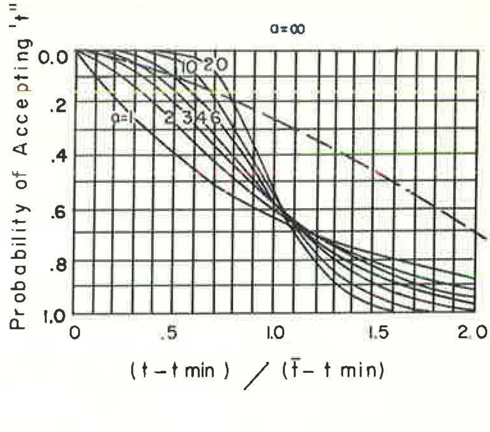

Figure 19. Comparison of "Raff-Bissell" probability of acceptance with standardized Erlang functions (stopped vehicles).

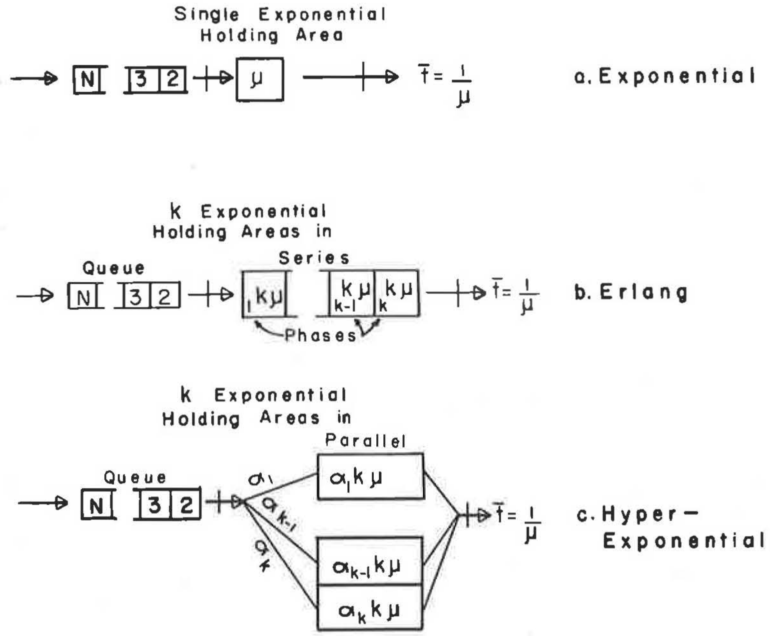

Figure 20. Schematic diagrams of simulators for generating exponential, Erlang and hyper-exponential gap distributions.

Proponents of both the Erlang and hyper-exponential gap distribution models offer rational, but incomplete, bases for their respective models. A more comprehensive description would be afforded by a model combining the properties of the hyper-exponential and Erlang models. Before proposing a specific model it would perhaps be worthwhile to show the physical relationship between the component parts. A schematic diagram of a device, or simulator, to generate completely random time gaps is shown in Figure 20(a). If a continuous queue exists at the entrance to a holding area from which units are allowed to depart at random with a mean rate μ, the times between departures from the holding area will be described by an exponential distribution with a mean of $\bar{t} = 1/\mu$.

Figure 20(b) shows a simulator for generating time gaps that range from the completely random exponential type to the completely uniform type. From a continuous queue at the entrance to the holding area, one unit at a time is allowed to enter the area, which is made up of several phases in series. The entering unit must first go through phase 1 which has an exponential time distribution with a mean rate of completion of $K\mu$. Upon leaving phase 1 the unit goes immediately to phase 2, phase 3, . . . , phase k-1, and to phase k, each with a mean rate of completion of $k\mu$. The distribution of times between departures of the units from phase k, and therefore from the holding area, are described by the Erlang distribution with a mean time $\bar{t} = 1/\mu$. Although the holding area is compounded, it can be considered as a single area because only one unit is allowed in it at a time. That is, the (n + 1) unit cannot enter phase 1 until the n unit has departed from phase k.

Hyper-random or hyper-exponential time gaps are generated by the simulator shown in Figure 20(c). From a continuous queue at the entrance to the holding area, one unit at a time is allowed to enter the area made up of k branches in parallel. As the unit enters it is assigned to one of the k branches at random, but on the average it is assigned to the i th branch a portion α_i of the time. When a unit is held in any one of the branches the entire holding area is considered to be occupied and no other unit can enter any branch until the preceding unit is discharged. The distribution of time gaps

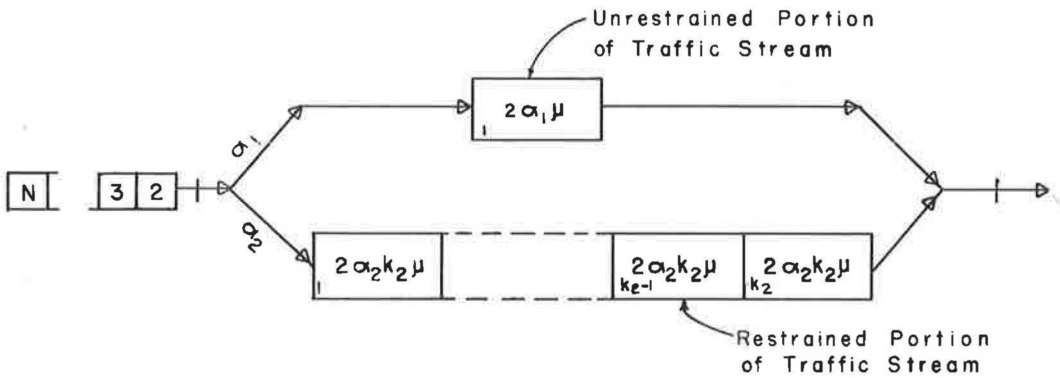

Figure 21. Hyper–Erlang gap distribution simulator for freeways.

between departures from this holding area is described as a hyper-exponential distribution with a mean time between departures of $\bar{t} = 1/\mu$.

At a glance it is apparent that the single exponential holding area is a special case of both the Erlang series arrangement and the hyper-exponential parallel arrangement. It is the case in which there is only one phase in the series arrangement, and the case in which there is only one branch in the parallel arrangement. The exponential distribution is therefore a boundary between Erlang distributions and hyper-exponential distributions.

There is, of course, no real physical device inherent in the outside lane of the freeway that regulates stream flow in such a manner as to generate either hyper-exponential or Erlang type headways. Field observations have shown that the simple exponential

the simple exponential function as a special case.

A more general model for describing gap availability could be constructed by combining the hyper-exponential and Erlang functions into a hyper-Erlang model, shown schematically in Figure 21. The model that might be fitted to gap data would probably consist of an Erlang ($a = k = 1$) function to describe unconstrained vehicles in the stream, in parallel with an Erlang ($a = k \geq 1$) function to describe the constrained vehicles in the stream. In addition, simple shifts of the axes to establish minimum gaps for both the constrained and unconstrained vehicles might be in order.

Delay and Queuing Characteristics

Drew has presented a collection of general, theoretical models for describing delay to the ramp stream as it traverses the merge area. The mere formulation of these models in general form is a significant contribution to the solution of the ramp queuing problem. Using the general equations, and substituting in any desired functions to describe gap availability in the shoulder lane of the freeway and gap acceptance by ramp vehicles, a traffic engineer could obtain estimates of the delay characteristics for given levels of freeway ramp operation. Some care would have to be exercised in applying the models, because they have been structured in such a way that they are not applicable to all ramp situations. In general, Drew's models are applicable to those situations in which all ramp drivers use the same decision model to make gap acceptance decisions. Such is the case on stop-sign controlled ramps where every ramp driver must make his gap-acceptance decision in a stopped condition.

If there is no absolute stop control on the ramp, a driver does not necessarily have to stop and he may make the gap acceptance decision in a moving condition. All drivers refusing a gap while moving would be forced to stop (or at least slow down) and would therefore make subsequent decisions in a stopped condition. Obviously, this complex situation cannot be handled by the proposed delay model unless a gap acceptance function

adequately approximating both decision functions can be obtained. But since Drew himself offers evidence that gap acceptance models for moving and stopped ramp vehicles are different, any such approximation would provide only approximate delay estimates.

There is reason to believe that gap acceptance is directly related to the relative speed between ramp vehicles and shoulder lane vehicles, rather than being just a two-phase phenomenon described separately for stopped and moving ramp vehicles. There is also evidence that gap acceptance characteristics for ramp vehicles merging into a freeway gap in queues are different from characteristics for single vehicles merging into the freeway flow. If these latter aspects of gap acceptance are significant, it will be difficult to adequately describe traffic flow on a ramp with a queuing model. Perhaps the traffic engineer will find that the ramp problem can best be analyzed by a Monte Carlo simulator, in which it is possible to include almost all of the micro aspects that influence overall traffic performance in the ramp-freeway merge area.

JAMES H. KELL, Principal Traffic Engineer, Traffic Research Corp.—Dr. Drew is to be complimented for preparing an excellent paper on a subject which is becoming increasingly critical in freeway operations.

I would like to confine my discussion to gaps and gap acceptance. I realize that my comments may be somewhat out of context with Drew's presentation in that he is primarily concerned with the utilization of gaps in a ramp control scheme. However, it is my belief that more knowledge concerning normal gap acceptance is desirable to develop fully the potential of ramp metering.

Drew defines a gap as the "interval of time between the arrival of two successive shoulder lane vehicles." This is not precisely true for the first gap (or lag, as many authors have chosen to call it) available to a merging vehicle. It is really the time between when the merging vehicle is ready to merge and the arrival of the next shoulder lane vehicle. Measurement of the first gap (or lag) and even succeeding gaps is complicated by the indefinite time when the merging vehicle is ready to accept a gap.

If visibility is good, the merging driver may be able to evaluate an approaching gap quite early and decide to accept it. He then accelerates or decelerates as appropriate to arrive at the merge point at the proper time to accept the gap. On the other hand, if visibility is not good, the driver cannot see the stream in advance and must, therefore, make a more hurried decision.

The technique described in this paper measured gaps in the shoulder lane and ramp arrivals at the nose of the ramp. Ramp vehicles may have already altered their behavior before reaching this point. Therefore, a true measurement of the ramp time is not really obtained.

Turning to the subject of critical gaps, Drew implies that each driver has a specific critical gap. Later on, he divides this into two critical gaps, one for moving vehicles and one for stopped. This is one area where the terminology of lags and gaps might be beneficial.

Assuming for the moment there is such a thing as a critical gap for an individual driver, it is my belief that this value would vary depending on traffic conditions present. The flow rate in the shoulder lane certainly affects the merging driver. With low flows, he is not too concerned if he misses one acceptable gap; another will appear directly. With high flows, he tends to accept shorter gaps since he might have to wait longer for another acceptable gap to arrive.

Speed also affects the entering driver. He will accept a shorter space gap when the freeway is moving slowly. Although I have no data to substantiate my contention, I believe that time gaps also lengthen with higher speeds.

In Bissell's work, which Drew quotes, approxmiately 5 percent of the vehicles observed ultimately accepted a shorter gap than one previously rejected. There is no indication in the paper that Drew observed this in his study, but I have observed it on

freeways, especially under conditions of high flow. Either the driver has not had time to evaluate conditions or he becomes impatient with the delay.

I have not collected data in the field, but have been interested in gaps and merging for quite some time, especially during the last few months since I reviewed Drew's paper. During this period I have attempted to evaluate my own behavior subjectively and have watched others perform the merging operation.

When the freeway is moving freely, I seem to follow a certain rote or procedure. The speed in the shoulder lane seems to be my primary criterion. In many cases, there is a lead vehicle starting the first available gap (orlag) in the shoulder lane. I apparently decide whether I can match his speed, merging behind him in the area available. If this decision is affirmative, I look for the following vehicle. In this case, space seems to be the important criterion. I have frequently accepted space gaps of 50 ft or less. At 60 mph this is a rather short time gap. I do tend to consider the following vehicle's speed and may reject the gap if that vehicle appears to be closing the gap, but not necessarily. In heavy traffic, when the freeway speed is lower, space seems to be the most important item. If there is space available much greater than a vehicle length and I can match the speed of the traffic stream, I will accept the gap.

My behavior is radically altered if there is a vehicle in front of me on the ramp. I tend to hold back to let the preceding vehicle merge and to allow myself maneuvering room to make my own merge.

When traffic is so heavy that I have to stop, especially where I am unable to observe the approach, I may miss acceptable gaps because I have not had time to evaluate the freeway stream. I know there have been numerous such occasions where I finally accepted gaps shorter than ones I had rejected.

Of course, I cannot say that I am an average or typical driver, but I am one charactor in the traffic stream. I have also watched other drivers merge. I have seen some drivers (especially where there was an acceleration lane) drive almost to the end of the land and then turn their heads for the first time to look for a gap. If an acceptable gap is not present right then and there, they stop. This tends to create the

This opens a much discussed topic as to whether driving habits vary in different parts of the country. I believe they do, but this is not the time nor place to pursue that subject.

However, there is one item that is worthy of mention that does occur on California freeways. There is frequently an interaction between the ramp vehicle and what I have called the following vehicle in the shoulder lane. The freeway vehicle will often modify his speed to alter the available gap when a ramp vehicle is present. Occasionally, he will speed up to reduce the gap and prevent the ramp vehicle from entering. More frequently, however, the following vehicle decelerates slightly to increase the gap and then closes up after the merge has been accomplished. There is also a strong tendency at many locations for the shoulder lane vehicles (even heavy trucks) to merge left in advance of a ramp, which is observed even under relatively high volume conditions.

To summarize this portion of the discussion, I believe that freeway speed and volume have a significant effect on gap acceptance. I alsofeel that an individual driver's critical or acceptable gap varies in any given situation as he comes down the ramp. It varies from an initial value for a "flying" merge to some other value (which may change) when he actually stops. To my mind, this becomes critical for a ramp metering scheme where gaps are measured at an upstream location and vehicles are released from the ramp to utilize these gaps.

I dispute Drew's nomenclature, that any vehicle that has rejected one gap is a stopped vehicle. I believe several gaps can be rejected with the ramp vehicle still moving with appreciable speed. This is highly dependent, of course, on the configuration of a particular on-ramp.

There is one other point in Drew's paper on which I would like to comment. In the discussion of stopped vehicles, he points out that a single vehicle can reject a number of gaps but can accept only one. I agree that a bias might be present if all rejected gaps are given the same weight as the one accepted. But, the one accepted gap is also

biased because the same vehicle might have accepted a shorter gap. In Drew's illustration (Fig. 10), the defining example is a vehicle that accepted a gap between 4.0 and 4.4 sec after rejecting a maximum gap of 0.5 to 0.9 sec. Might not this vehicle have accepted a gap of 3 sec, or 2 sec, or more? Conversely, might it not have rejected a gap of 2 sec, or 3 sec, or more? Yet this vehicle contributes eight intervals to the total being used to determine a distribution where other vehicles contribute only one. The resultant distribution is still biased. I believe the whole subject of determining gap acceptance distributions is worthy of more detailed investigation.

In conclusion, Drew is to be commended for adding to the basic knowledge of gap acceptance. Although this research was only an adjunct to their primary objective of freeway surveillance and control, it has provided valuable information in one of the basic areas where knowledge is lacking. I believe it has pointed out the need for more study in this area.

D. R. DREW, Closure—The author is appreciative of the excellent reviews by Dr. Paul D. Cribbins, Dr. Robert F. Dawson and Professor James Kell.

Dr. Cribbins has very aptly defined some of the objectives and problems associated with ramp control. His criticism of the failure in this report to suggest the potential cost savings to motorists attainable through more sophisticated surveillance and control systems is both constructive and timely. There is a definite need for cost-benefit work in this area.

Dr. Dawson has made several important observations that contribute much to the understanding of gap availability and gap acceptance:

1. Either of the weighting procedures he suggests would improve the fit between the observed and theoretical critical gap distributions shown in Figure 11.
2. The critical gap parameters summarized in Table 3 obtained from the procedure utilizing the gap acceptance functions of Figure 7 are, as he states, not the same as those obtainable from the critical gap distribution procedure illustrated in Figures 10 and 11. Actually, the linearity of the plots on the log probability paper in Figure 7 imply cumulative log normal distributions rather than Erlang distributions.
3. The "hyper-Erlang" gap distribution simulator he describes should enable investigators to add considerable realism to future freeway simulation models.

However, the advantage of the Erlang distribution over (a) the log-normal as a gap acceptance model, and over (b) the hyper-Erlang distribution as a gap availability model is dramatized in the derivations in the section on "Determination of Merging Delays" in the report (Eqs. 1-20). The latter distributions defy such analytical treatment.

Professor Kell's comments are directed to the oversimplification of merging as treated in the paper. He states that only gaps, not lags, were considered and that such important aspects of merging as multiple gap acceptance, driver policy, and driver characteristics have generally been ignored. Actually, these were not intended for consideration in this research, not because they are unimportant in explaining the merging maneuver, but because they are largely irrelevant to a ramp-freeway surveillance and control system. For example, lags cannot be detected and measured automatically, whereas gaps can. The purpose of this paper was to gain insight into the calibration of a metering system to be installed on the ramps of the Houston surveillance project.

Again, appreciation is expressed to the discussers for their reviews. The author sincerely feels that the three discussions represent significant contributions in their own right.

Peak Period Comfort and Service Evaluation of an Urban Freeway and an Alternate Surface Street

VASANT H. SURTI and EDWARD F. GERVAIS
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•AN IDEAL HIGHWAY transport facility for moving people and goods could be defined as one giving the most service in the least amount of time and with the least effort spent on it. A well-designed, modern urban freeway system should satisfy these requirements. Reference to safety has been purposely deleted from this definition because no accident-breeding situation could fulfill the foregoing conditions.

Undoubtedly, the freeway system moves the greatest number of goods and people in an efficient manner except during certain periods when there is a concentration of vehicles in time and space. These concentration periods, commonly referred to as rush-hour or congestion periods, are daily occurrences in large metropolitan centers. The motorist is given a free choice to select his own route of travel; he may select a freeway, surface street route, or both. During rush-hour periods, unfortunately, there are few, if any, means of giving him advance information on traffic conditions. His judgment dictates that the comfort and convenience along with time savings would most likely be in his favor on the freeway.

The driver entering a freeway has very little individual choice concerning the headway and speed with which he travels. He attempts to drive "on" the fellow ahead and, [illegible] either a frontage road or a surface street is available, paralleling the freeway route.

If drivers are instructed to use an alternate surface street route to preserve good freeway operation, it would be wise to have preknowledge of how they will be accommodated on the alternate route. The instrumentation selected for this study was based on the desire to determine the tensions and stresses to which drivers are subjected on two parallel routes of travel. This is important for two reasons: (a) few drivers could be expected to accept willingly an alternate route offering additional tensions and complications of the driving task; and (b) the likelihood of accidents should not be increased. If the two conditions are negative, it appears more logical to offset deficiencies in freeway traffic carrying capabilities by building more freeways. This choice is expensive; therefore, factual evidence should be gathered to prove whether surface routes, even when used as an alternate to freeways, can perform an acceptable and safe service to the driver.

PURPOSE

The National Proving Ground for Freeway Surveillance, Control and Electronic Traffic Aids project has a 3.2-mi section of John C. Lodge Freeway traffic under surveillance by 14 television monitors and a system of ultrasonic vehicle detectors. From the central control room of the project it is possible to control the on-ramp traffic by displaying the ramp closure sign at the entrance ramp. A part of past and future research concerns the experimentation with ramp control to optimize freeway operation

Paper sponsored by Committee on Quality of Traffic Service and presented at the 45th Annual Meeting.

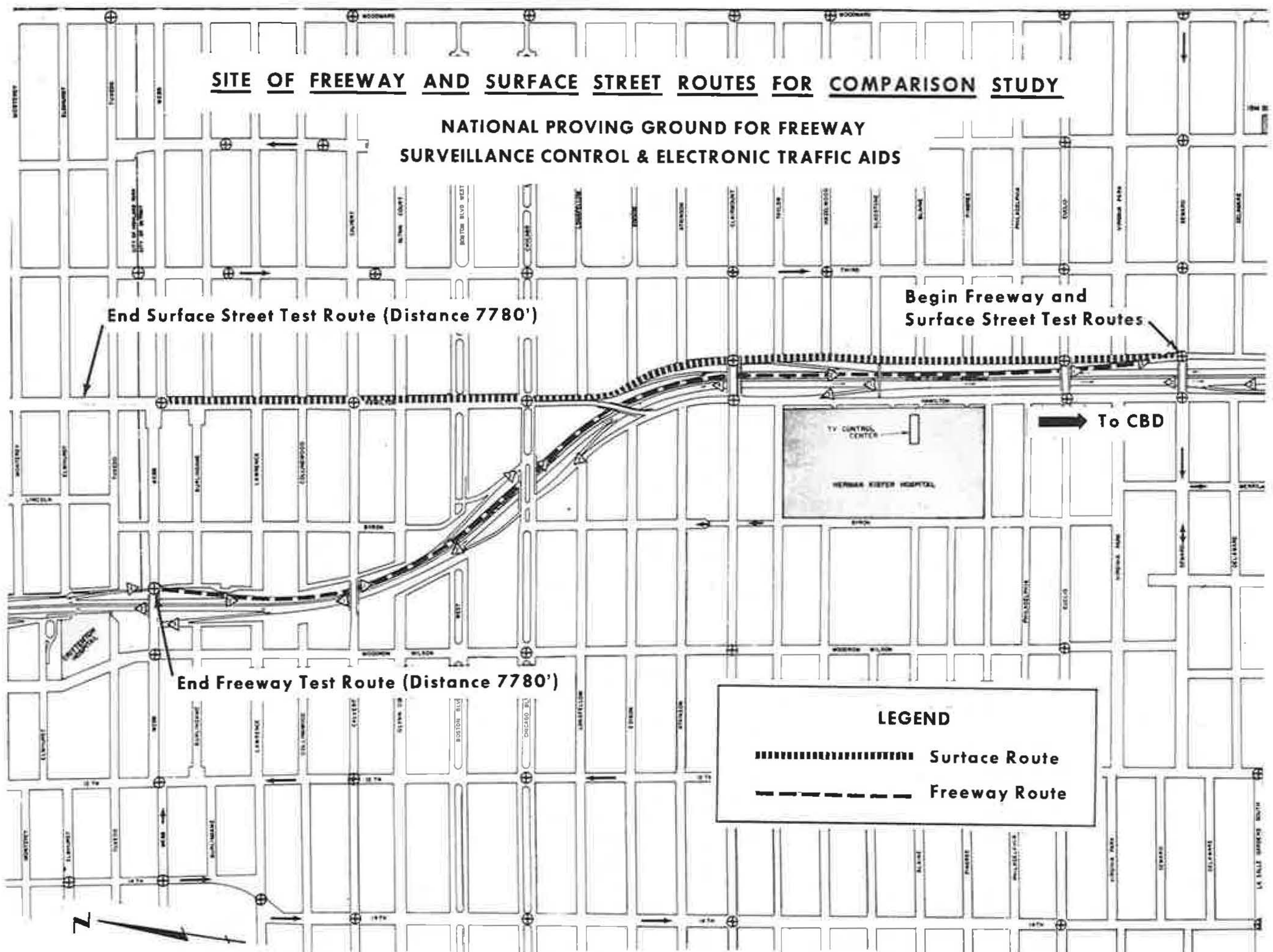


Figure 1. Study site.

and integrate it with surface route travel. The purpose of this study was to evaluate the level of service provided during rush-hour traffic by two facilities radically different in design and to determine to what degree a driver would either accept or be accommodated on each facility.

STUDY SITE

The study site selected was a freeway and an alternate surface street which is a part of the project study corridor (Fig. 1). The alternate surface street route considered was the route a motorist would take if the freeway ramp was closed. The freeway route selected for the study began from Seward Ave. entrance ramp in the northbound (outbound from Central Business District) direction and ended just in front of the traffic signal at the Webb Ave. off-ramp. The freeway is primarily a six-lane depressed facility with an auxiliary lane provided near some high volume on-ramps. For the test route, the auxiliary lane continues from the paint nose of Seward Ave. entrance ramp to the paint nose of Chicago Blvd. exit-ramp, about a third of the total distance of the test route. Two vertical curves with grades ranging from 0.4 percent to 1.0 percent are superimposed on a 3-deg reverse curve between Hazelwood Ave. and Webb Ave. There is a continuous 8-ft bituminous-paved refuge shoulder with mountable curb in each direction.

The surface street route began on the northbound service drive just beyond the traffic signal at Seward Ave. The service drive is a two-lane one-way facility and jogs into Hamilton St. at Longfellow Ave. The test route ended on Hamilton St. about 150 ft north of Tuxedo Ave. Hamilton St. has two traffic lanes and a parking lane in each direction. No parking or standing is allowed on the service drive, whereas one-hour parking is allowed on Hamilton St. from 7:00 a.m. to 4:00 p.m. and no standing is allowed from 4:00 p.m. to 6:00 p.m. There are 10 bus stops on the surface street route and the speed limit is posted at 30 mph. The posted speed limit on the freeway during off-peak hours is a minimum of 45 mph to a maximum of 55 mph. During peak hours the speed limit is variable. The traffic signals along the service drive and

street grid system will permit. The freeway television control center displays the speed limits of 25, 40, or 55 mph, depending on the prevailing traffic conditions on the freeway. The spatial distance between the two termini on each test route was 7.780 ft.

TEST EQUIPMENT

Galvanic Skin Response Instrument

The Galvanic Skin Response (GSR) instrument was first developed by Charles Féré in 1888. He attached two electrodes on the forearm, connected in series with a weak source of current and galvanometer. He found that the galvanometer gave quick deflections when the subject was stimulated by a tuning fork, an odor, or a colored glass before his eyes. It measures the electrical conductance (or its reciprocal, resistance) of the skin. The skin conductance ranges from a low level in sleep to a high level in strongly stimulated states like rage, and it is sensitive enough to detect the mild interest aroused by a new sound in a quiet room. It seems to be a good measure of tension, since the conductance is high when the subject is excited and low when he is relaxed (1).

The GSR instrument has been primarily used in the field of psychology. Its use in the traffic engineering field has been very recent and very limited. Cleveland (2) found that GSR discriminates between the conditions of illumination at rural intersections, producing only 80 percent as many tension responses under illuminated conditions as when there is no light. Michaels' (3, 4) findings indicate that driver responses could be used as a means of discriminating between different types of city streets, and also that this technique could be applied to discriminate among the expressways of different design.

Drivometer

The Drivometer is a joint development of B. D. Greenshields of the University of Michigan and F. N. Platt of the Ford Motor Company. It records the following information in digital form:

1. Steering wheel reversals of $^3/_8$ in. or more on a 17-in. diameter wheel per count.
2. Accelerator reversals, all up or down motions of $^1/_8$ in. per one-half count.
3. Speed changes of plus or minus 2 mph per one-half count.
4. Brake applications of about 90 psi per count.
5. Trip time in tenths of a minute.
6. Running time in tenths of a minute.

Greenshields found that the information gathered from the Drivometer can be used to distinguish among the different classes of drivers since they exhibit different driving patterns. The classes of drivers considered were: (a) high school drivers who had completed a driver training course; (b) driver education teachers; (c) professional drivers such as truck and taxi drivers; (d) high-violation drivers; (e) high-accident drivers; and (f) average drivers (5). Platt used it to evaluate the effects of driver stress and fatigue as reflected in tracking and speed control factors (6).

DATA COLLECTION

Four male drivers, each having driving experience of more than 5 years, with no accidents during the past 3 years and no more than 6 points on their driving record according to the point system of Michigan, were selected as test drivers. The reason for the selection of four drivers who were considered to be average was to minimize the individual differences between them and eliminate any eccentricities which would be detected. The selection of a wide assortment of drivers was avoided to simplify the prime task of analyzing the differences between the two routes in the study area. Before the beginning of test runs for the day, each driver was calibrated for sensitivity on the GSR instrument. The first and third fingers of the left hand were connected to electrodes and the subject was asked to close his eyes and relax comfortably. A reaction of the subject was generated by a stimulus created by the poke of a finger. The sensitivity was adjusted so that the graphic recorder throughout the test runs would show maximum deflection while remaining on the scale. The sensitivity level for each test driver was left untouched until calibration for another day. The Drivometer was mounted in the glove compartment of a passenger car equipped with power steering, power brakes, and automatic transmission. The instrumentation is shown in Figures 2 and 3. The study team consisted of the driver, the observer, and the GSR operator. The observers' task was to differentiate between the various traffic and geometric events and convey this information in coded form to the GSR operator in the rear seat. The GSR operator identified these events on the graphic record produced by the GSR. The observer also recorded the Drivometer readings at the end of each run. The tension-generating events on the freeway and surface street routes considered are given in Table 1.

No subject driver had any previous experience with this kind of experiment. None of them were aware that two routes were being compared or the nature of responses or actions being recorded. Each driver was assured that his driving ability was not being tested. No specific driving instructions were given except that he was not to exceed the posted speed limit at any time. No smoking, talking, chewing of gum, or other diversions were allowed during the test runs.

The test runs were made between 3:00 p.m. and 6:30 p.m. for a period of three weeks under dry pavement conditions. Again this was done to keep variables to a minimum. The four drivers made the runs in sequential order; this routine was maintained for the entire data collection period. The first driver began the test run on the freeway route from the common control point. The same driver returned to the starting control point and completed the run on the surface street route. At any one time, only one driver was in a car. Observers or operators were not drivers who made test runs. In this manner the individual behavioristic characteristics of the drivers were

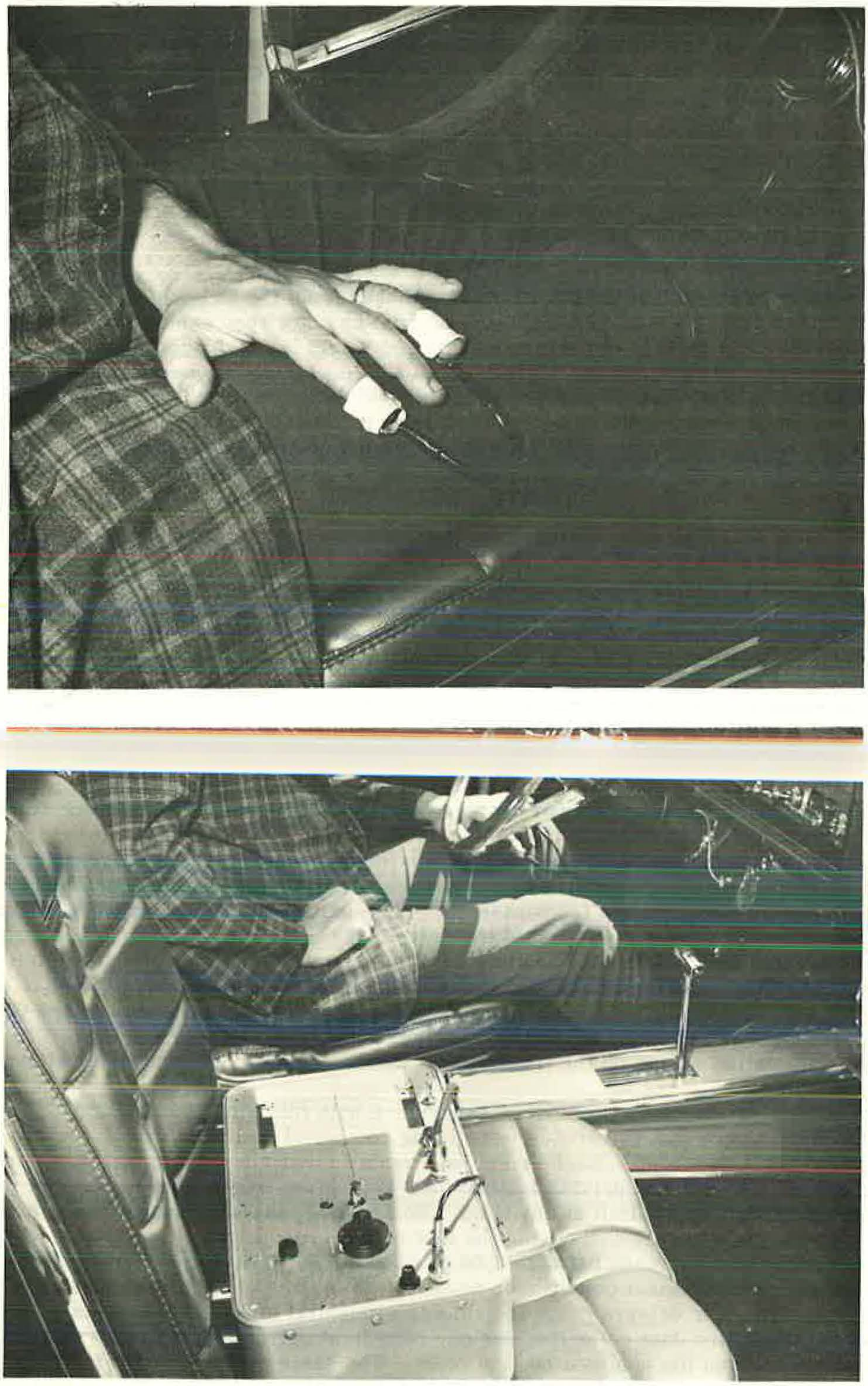


Figure 2. Galvanic skin response instrument setup.

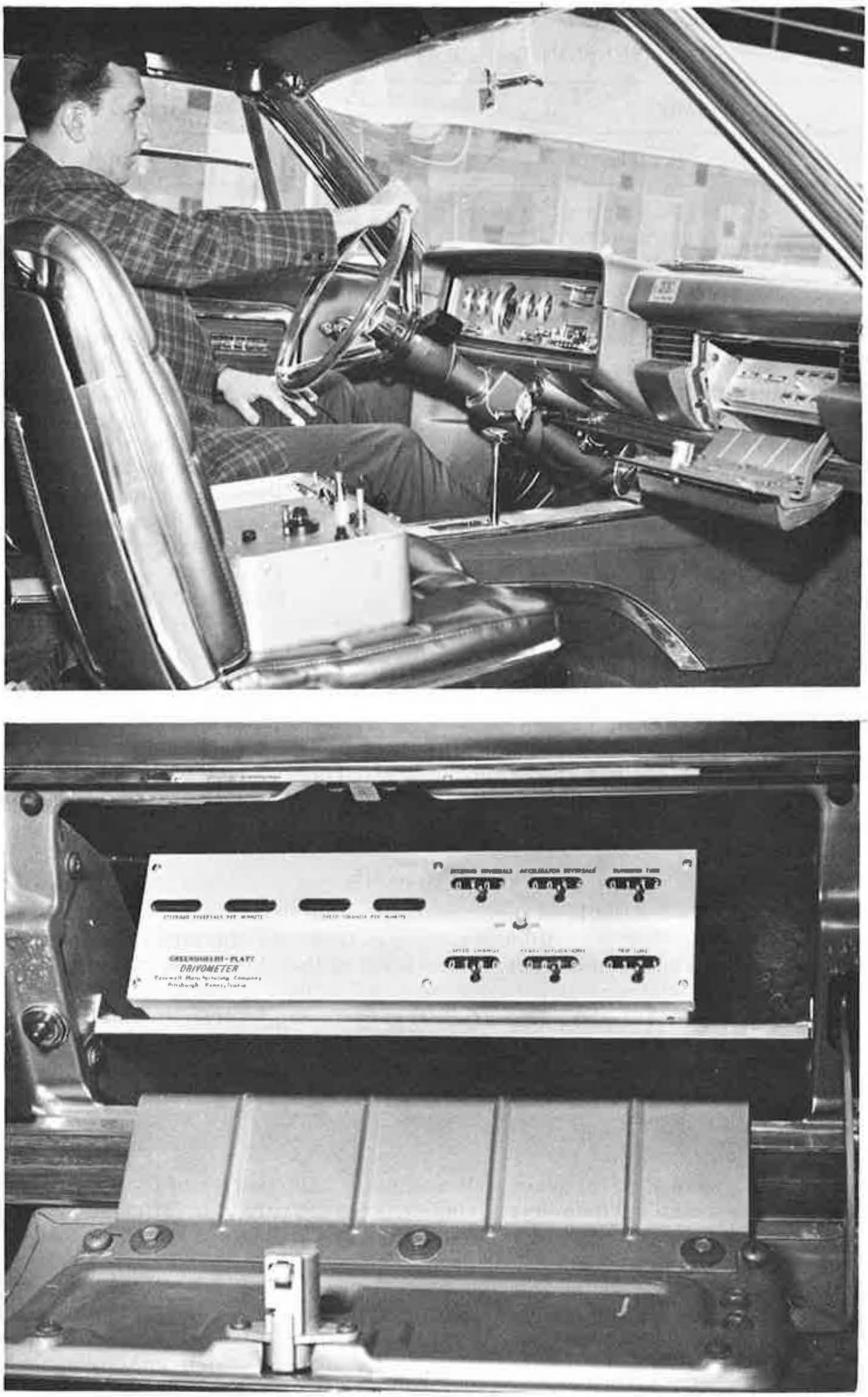


Figure 3. Drivometer and galvanic skin response instrument setup.

TABLE 1

TENSION GENERATING EVENTS

Freeway		Surface Street	
Event No.	Description	Event No.	Description
1	Longitudinal friction: traffic events encountered along the direction of motion. Includes lane change and overtaking of or by the test vehicle.	1	Longitudinal friction: opposing traffic, lane change and overtaking of or by the test vehicle.
2	Change in horizontal or vertical alignment.	2	Change in horizontal or vertical alignment.
3	Merging maneuver at the entrance ramp.	3	Right-turning vehicle and the right turn by the test vehicle.
4	Diverging maneuver at the exit ramp.	4	Left-turning vehicle and the left turn by the test vehicle.
5	Shoulder incident.	5	Parking and unparking; also the loading and unloading of passengers by bus.
		6	Pedestrians.
		7	Signals and signs.

uninfluenced. During the data collection period the observers in the central control room monitored the freeway route and kept a log of lane incidents (accidents or breakdowns). The test runs involving unusual lane incidents were excluded from the study. [illegible] 186 test runs.

DATA ANALYSIS

Several methods are available to measure GSR. The "resistance change" method was used in this study. The magnitude measure of GSR was obtained by determining the relative difference between the resistance level of the skin at the time the stimulus is presented and the resistance at the maximum deflection. This is the most frequently used measure since the data are recorded in this form and no transformation is required. The galvanic response measured in this study was generally a deflection beginning about $1\frac{1}{2}$ sec after the event was noticed, reaching a maximum in about 4 sec, and then returning gradually to the resting level.

Response Distribution

Figures 4 and 5 show the frequency and magnitude distribution of tension responses generated on freeway and surface street routes, respectively, for different types of events. The percentage distribution was computed on the basis of the total test runs made on each facility.

Longitudinal friction accounted for the greatest percentage of the total responses in frequency and magnitude for freeway as well as surface street route. This effect was more pronounced on the freeway than the surface street. Considering the magnitude of GSR as a measure of the degree of difficulty in the decision-making process of the driver, the on-ramp merging event generally produced a response of higher magnitude than all other traffic and geometric events. Figure 4 shows that the magnitude distribution for the on-ramp merging event was 20.38 percent, whereas the frequency distribution was only 17.45 percent. Thus, fewer responses accounted for the higher

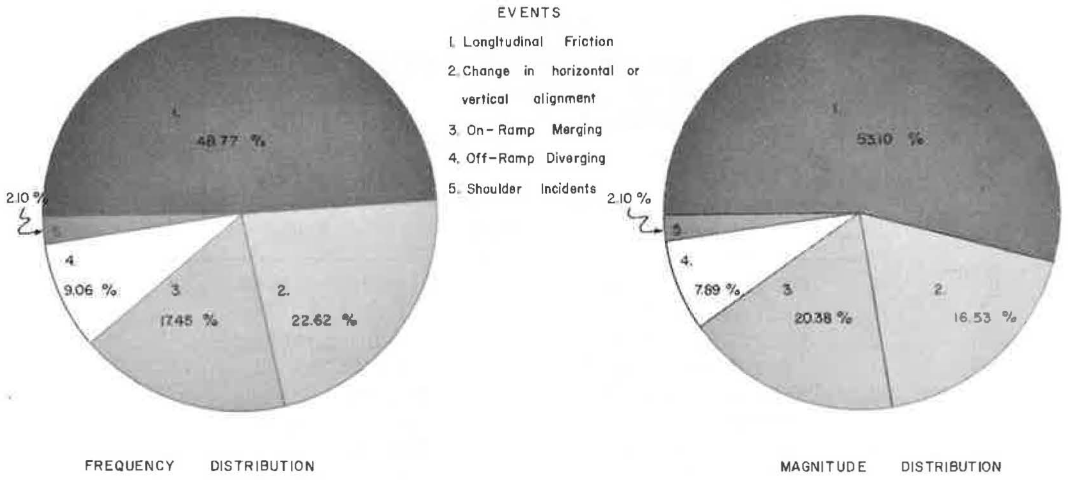


Figure 4. Tension responses generated on freeway by type of event.

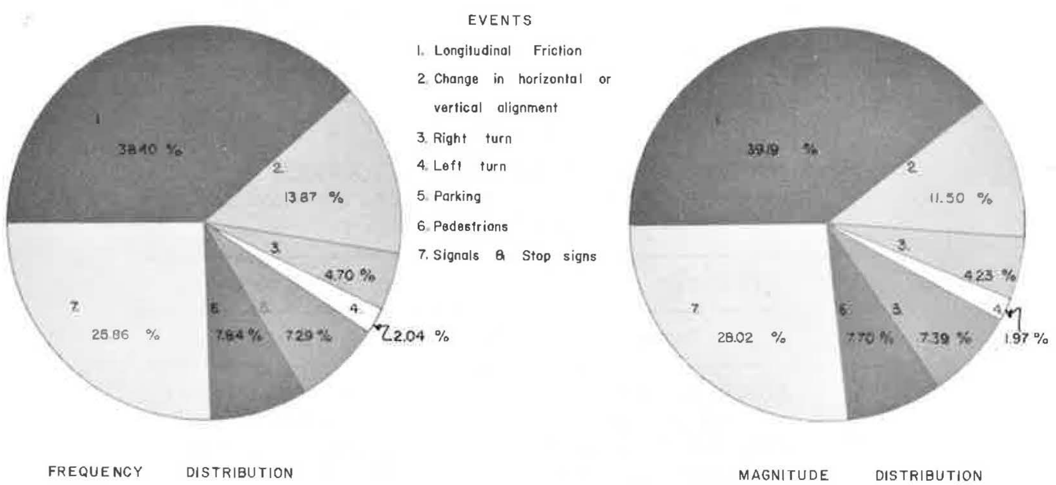


Figure 5. Tension responses generated on surface street by type of event.

percentage of the total magnitude distribution. The longitudinal friction on the freeway was the second most difficult event in the driver decision-making process. Table 2 gives the rank of various traffic and geometric events for freeway and surface street according to the degree of difficulty.

Table 3 ranks the traffic and geometric events by the frequency of occurrence of tension responses. The change in horizontal vertical alignment is ranked second according to frequency but third according to magnitude. This is logical considering the number of horizontal and vertical changes in alignment as compared to on-ramps. All other rankings based on magnitude and frequency are identical.

TABLE 2

TRAFFIC AND GEOMETRIC EVENTS RANKED ACCORDING TO DEGREE OF DIFFICULTY OF DRIVER DECISION PROCESS (MAGNITUDE)

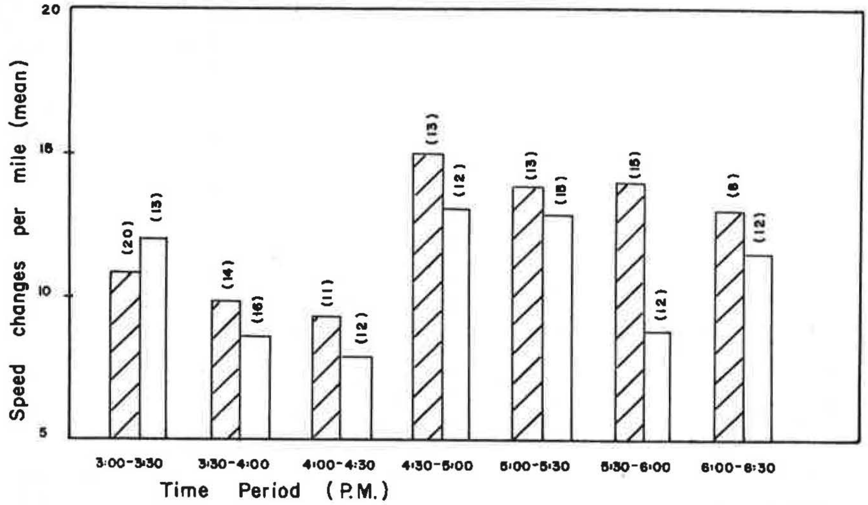
Freeway		Surface Street	
Rank	Event	Rank	Event
1	On-ramp merging (Event No. 3)	1	Signals and stop signs (Event No. 7)
2	Longitudinal friction (Event No. 1)	2	Longitudinal friction (Event No. 1)
3	Shoulder incidents (Event No. 5)	3	Parking (Event No. 5)
4	Off-ramp diverging (Event No. 4)	4	Pedestrians (Event No. 6)
5	Change in horizontal or vertical alignment (Event No. 2)	5	Left turn (Event No. 4)
		6	Right turn (Event No. 3)
		7	Change in horizontal or vertical alignment (Event No. 2)

TABLE 3

ING TO FREQUENCY DISTRIBUTION OF TENSION RESPONSES

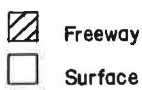
Freeway		Surface Street	
Rank	Event	Rank	Event
1	Longitudinal friction (Event No. 1)	1	Longitudinal friction (Event No. 1)
2[a]	Change in horizontal or vertical alignment (Event No. 2)	2	Signals and stop signs (Event No. 7)
3[a]	On-ramp merging (Event No. 3)	3	Change in horizontal or vertical alignment (Event No. 2)
4	Off-ramp diverging (Event No. 4)	4	Pedestrians (Event No. 6)
5	Shoulder incidents (Event No. 5)	5	Parking (Event No. 5)
		6	Right turn (Event No. 3)
		7	Left turn (Event No. 4)

[a]Based on a magnitude distribution, Event No. 3 ranked 2 and Event No. 2 ranked 3 for the freeway; remaining events had ranks identical to frequency distribution.



( ) Number of test runs made on the route during the time period.

Figure 6. Comparison of mean value of speed changes per mile on freeway with mean value of speed changes per mile on alternate surface street route.



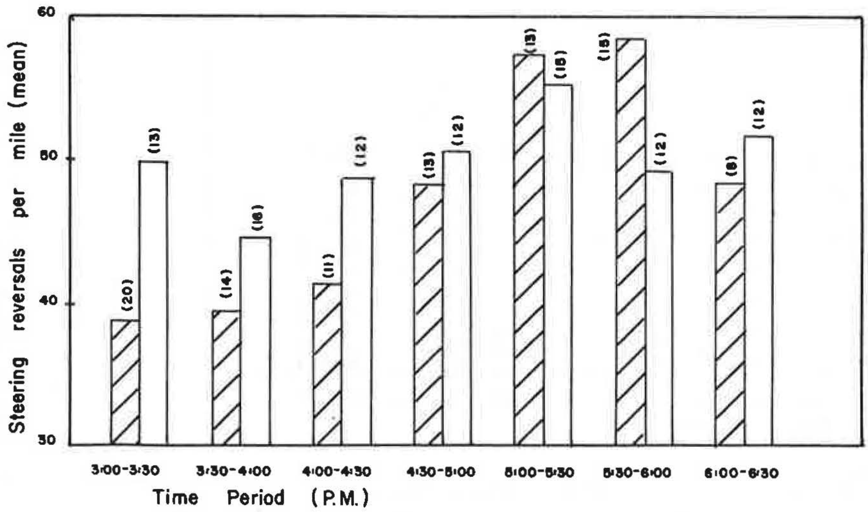
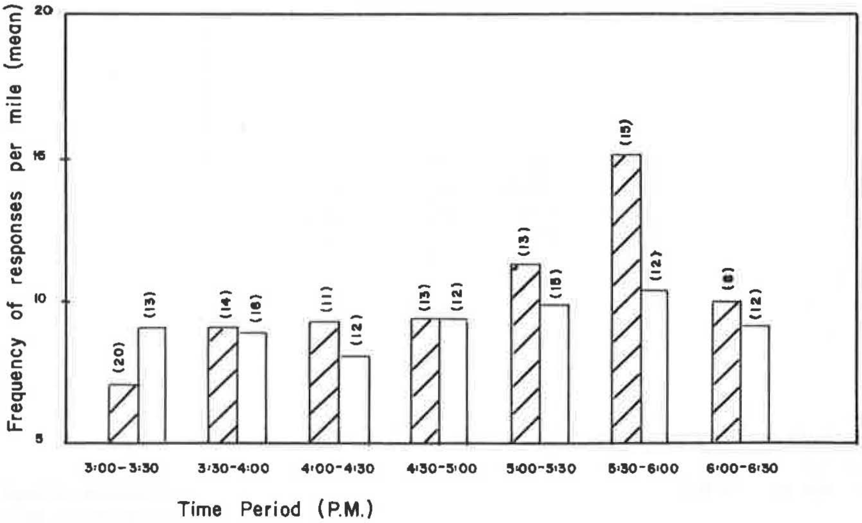


Figure 7. Comparison of mean value of steering reversals per mile on freeway with mean value of steering reversals per mile on alternate surface street route.



( ) Number of test runs made on the route during the time period.

Figure 8. Comparison of mean value of frequency of responses per mile on freeway with mean value of number of responses per mile on alternate surface street route.

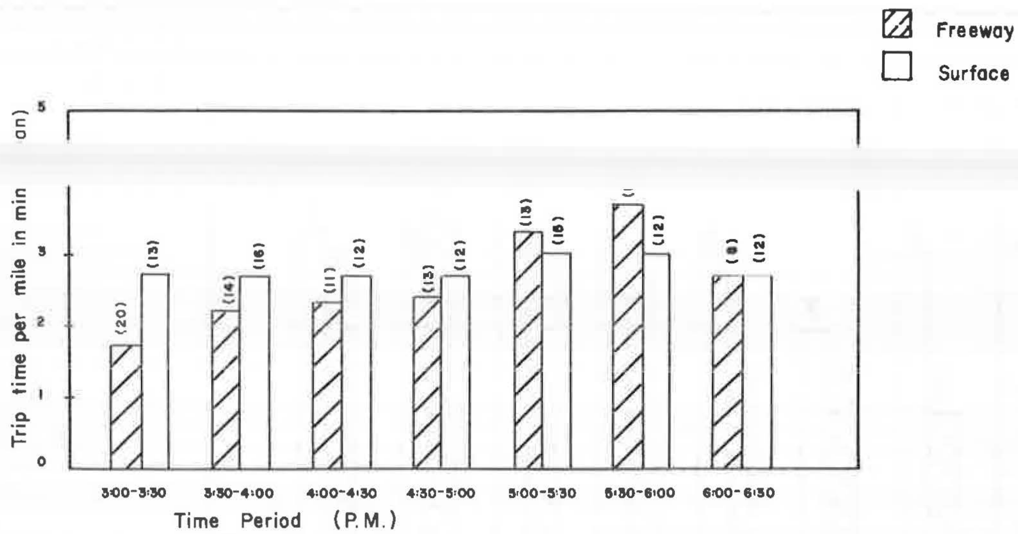
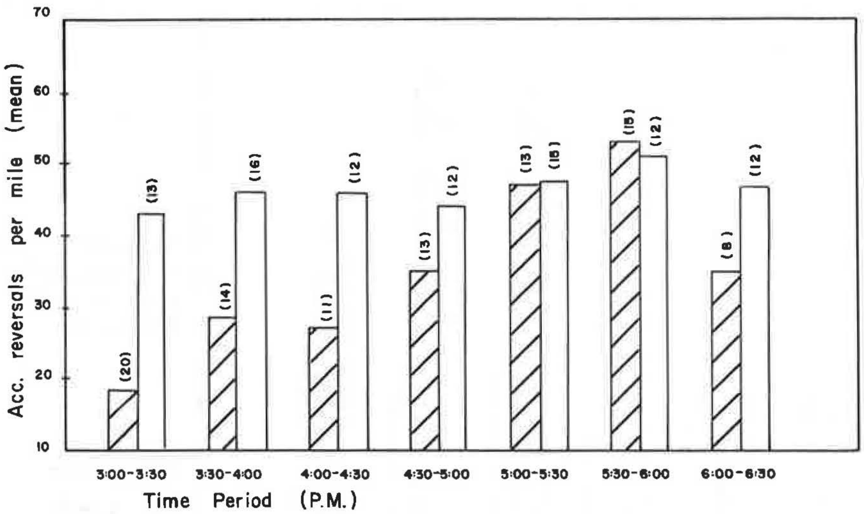


Figure 9. Comparison of mean value of trip time per mile on freeway with mean value of trip time per mile on alternate surface street route.



( ) Number of test runs made on the route during the time period.

Figure 10. Comparison of mean value of accelerator reversal per mile on freeway with mean value of accelerator reversals per mile on alternate surface street route.

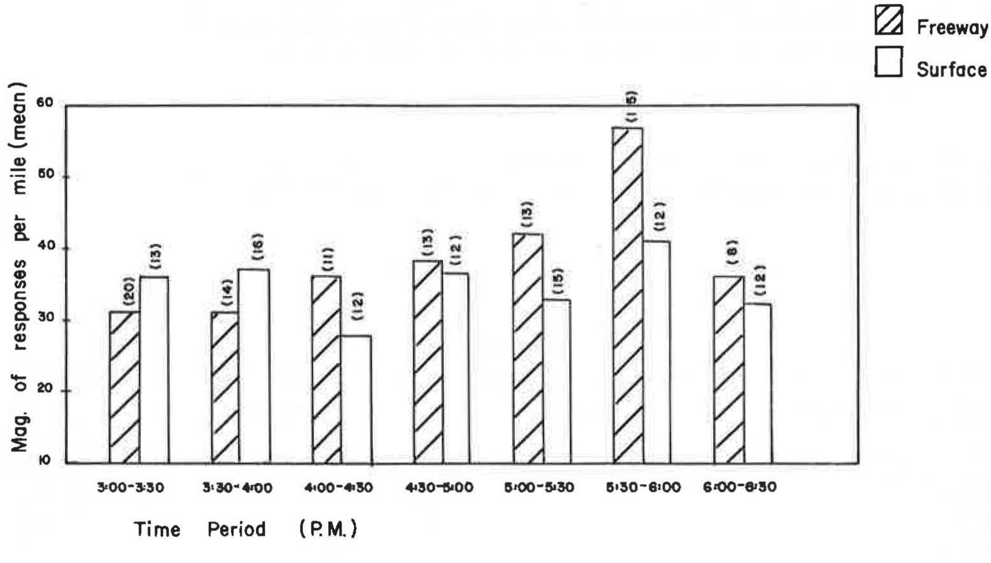


Figure 11. Comparison of mean value of magnitude of responses per mile on freeway with mean value of magnitude of responses on alternate surface street route.

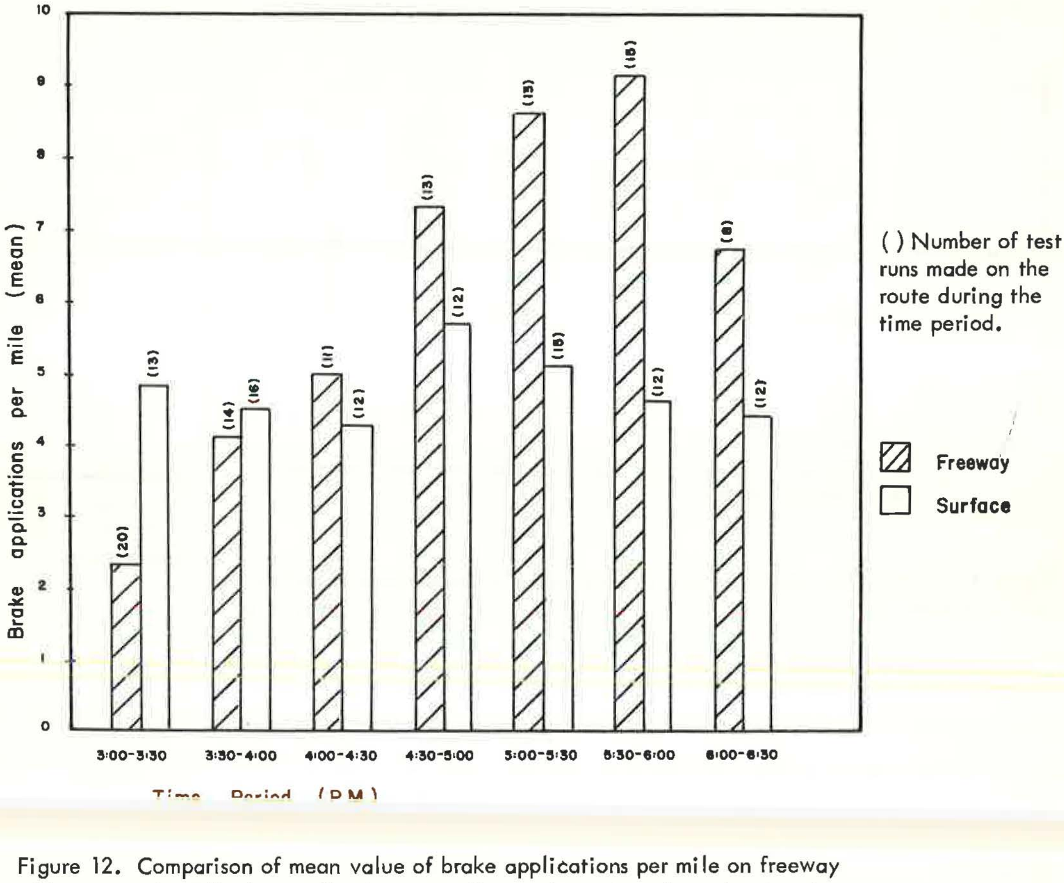


Figure 12. Comparison of mean value of brake applications per mile on freeway with mean value of brake applications per mile on alternate surface street route.

TABLE 4

SUMMARY OF SIGNIFICANCE TESTS FOR FREEWAY AND ALTERNATE SURFACE STREET ROUTE BY TIME PERIODS ON PER MILE BASIS[a]

Variable / Time Period (p.m.)	SR[b]/Mi		SC[c]/Mi		AR[d]/Mi		BA[e]/Mi		TT[f]/Mi		FR[g]/Mi		MR[h]/Mi	
	F'way	Surf	F'way	Surf	F'way	Surf	F'way	Surf	F'way	Surf	F'way	Surf	F'way	Surf
3:00-3:30		j	i	i		j		j		j		j	i	i
3:30-4:00		j	i	i		j	i	i		j	i	i	i	i
4:00-4:30		j	i	i		j	i	i		j	i	i	i	i
4:30-5:00	i	i	i	i		j	j		i	i	i	i	i	i
5:00-5:30	i	i	i	i	i	i	j		i	i	i	i	i	i
5:30-6:00	j		j		i	i	j		j		j		j	
6:00-6:30	i	i		i		j	j		i	i	i	i	i	i

[a] At 90 percent or better confidence level.
[b] Steering reversals.
[c] Speed changes.
[d] Accelerator reversals.
[e] Brake applications.
[f] Trip time.
[g] Frequency of response.
[h] Magnitude of response.
[i] No significant difference in rate per mile.
[j] Significantly higher rate per mile.

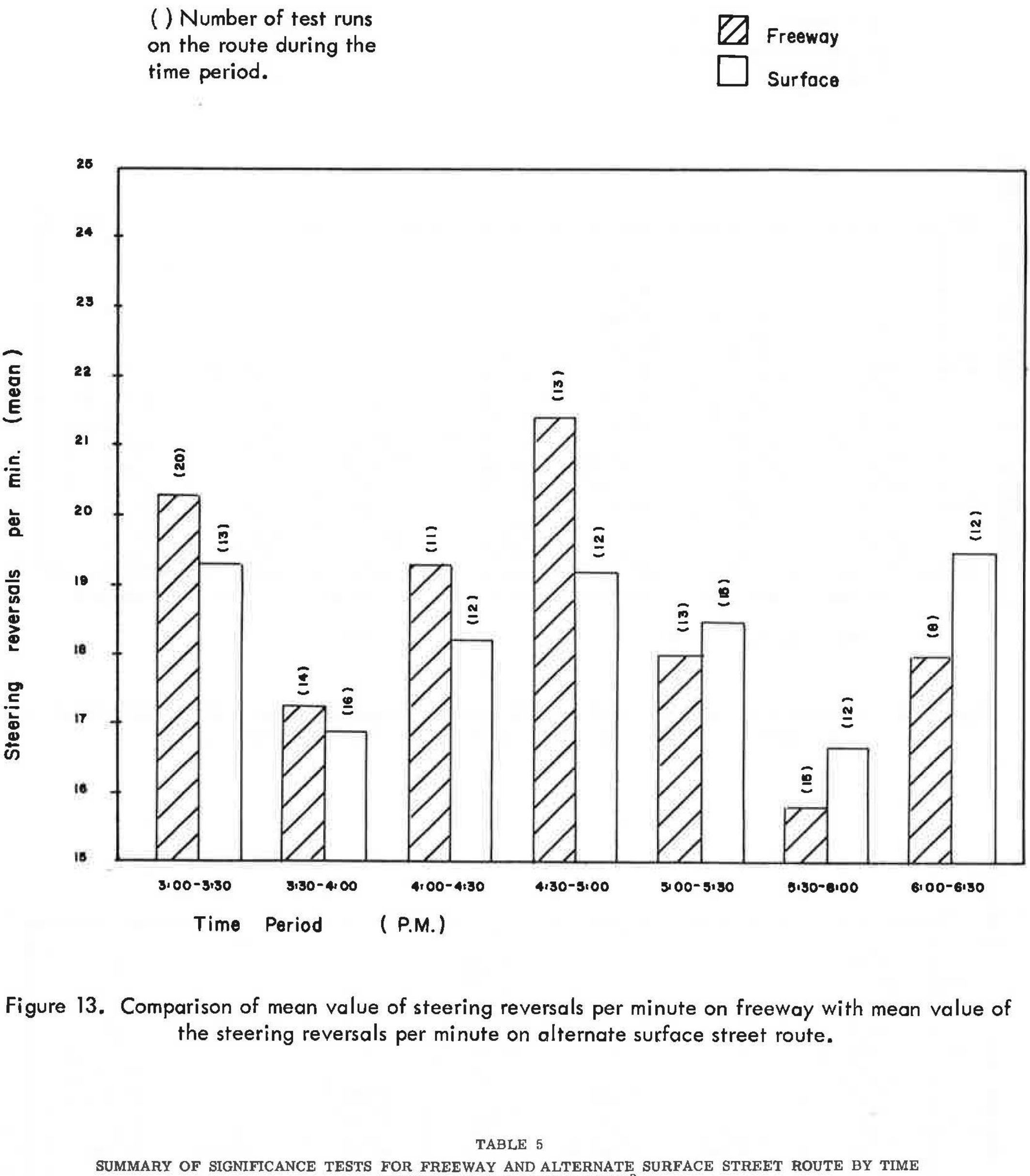


Figure 13. Comparison of mean value of steering reversals per minute on freeway with mean value of the steering reversals per minute on alternate surface street route.

TABLE 5

SUMMARY OF SIGNIFICANCE TESTS FOR FREEWAY AND ALTERNATE SURFACE STREET ROUTE BY TIME PERIODS ON PER MINUTE BASIS[a]

Variable / Time Period (p.m.)	SR[b]/Min		SC[c]/Min		AR[d]/Min		BA[e]/Min		FR[f]/Min		MR[g]/Min	
	F'way	Surf	F'way	Surf	F'way	Surf	F'way	Surf	F'way	Surf	F'way	Surf
3:00-3:30	i		i			i		i	i		h	h
3:30-4:00	h	h	i			i	h	h	i		h	h
4:00-4:30	h	h	i			i	h	h	i		i	
4:30-5:00	i		i		h	h	h	h	h	h	h	h
5:00-5:30	h	h	h	h	h	h	i		h	h	h	h
5:30-6:00	h	h	i			i	i		h	h	h	h
6:00-6:30	h	h	h	h		i	i		h	h	h	h

[a] At 90 percent or better confidence level.
[b] Steering reversals.
[c] Speed changes.
[d] Accelerator reversals.
[e] Brake applications.
[f] Frequency of response.
[g] Magnitude of response.
[h] No significant difference in rate per minute.
[i] Significantly higher rate per minute.

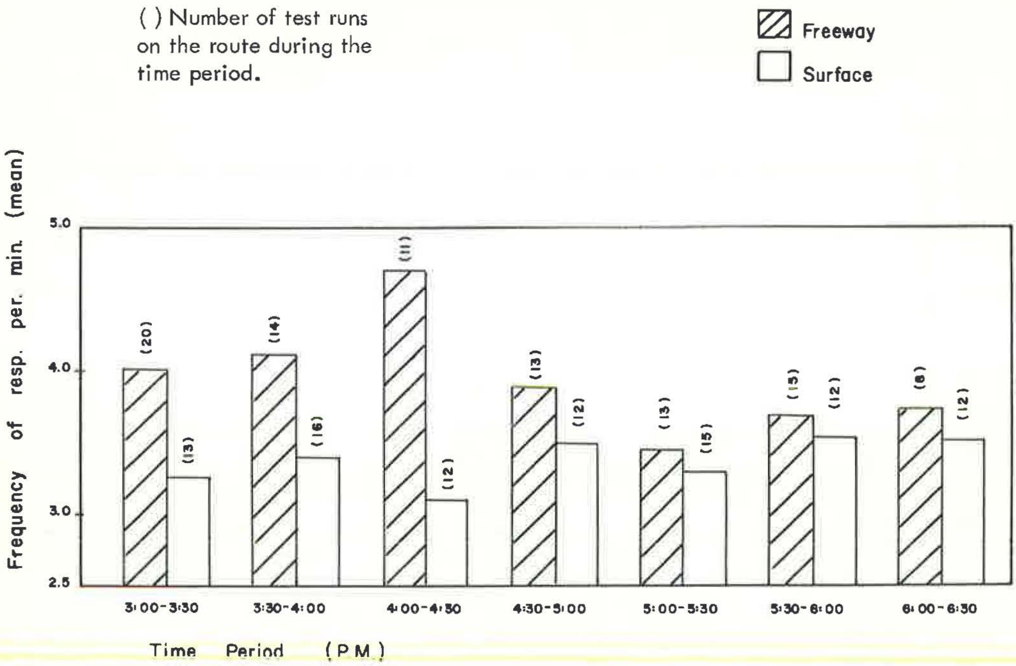


Figure 14. Comparison of mean value of number of responses per minute on freeway with mean value of number of responses per minute on alternate surface street route.

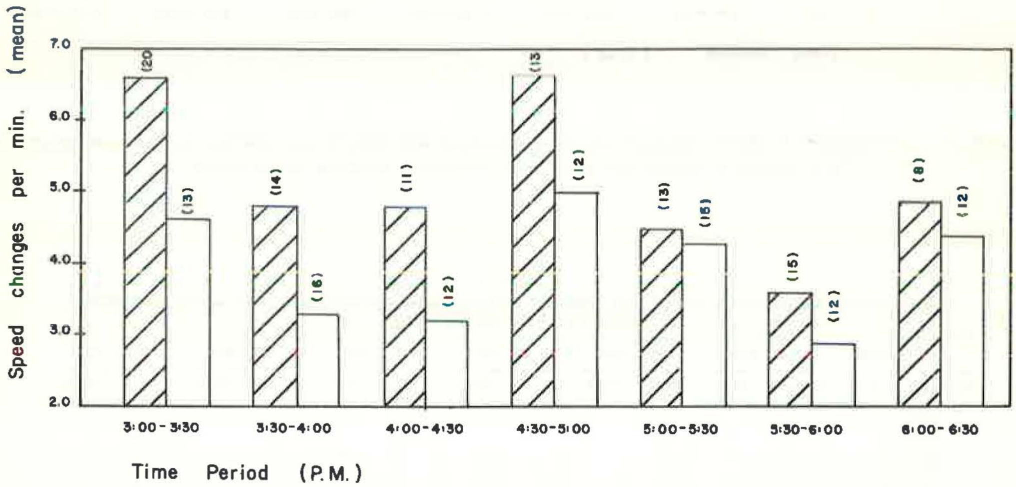


Figure 15. Comparison of mean value of speed changes per minute on freeway with mean value of speed changes per minute on alternate surface street route.

( ) Number of test runs
on the route during the
time period.

Freeway

Surface

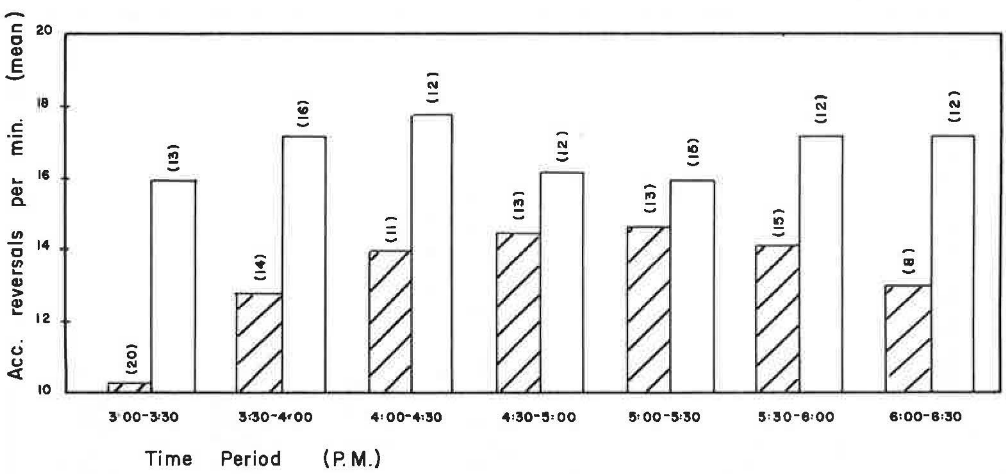


Figure 16. Comparison of mean value of accelerator reversals per minute on freeway with mean value of accelerator reversals per minute on alternate surface street route.

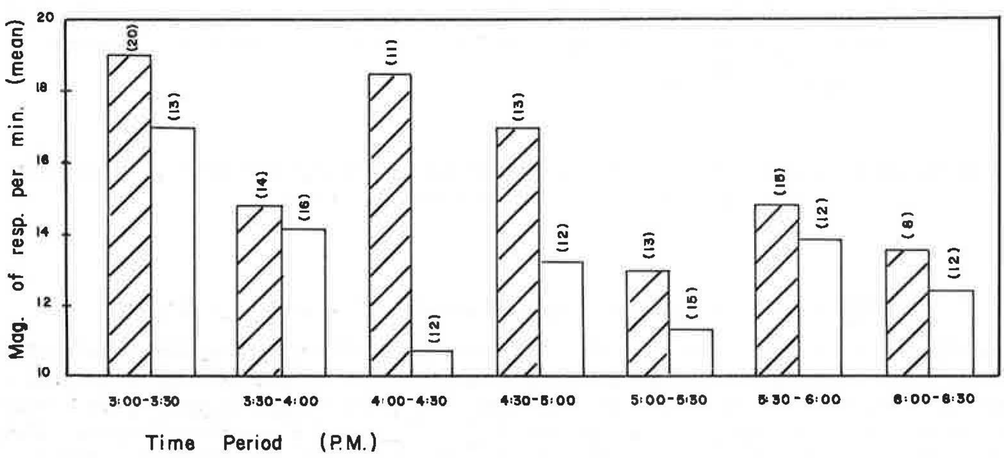


Figure 17. Comparison of mean value of magnitude of responses per minute on freeway with mean value of magnitude of responses on alternate surface street route.

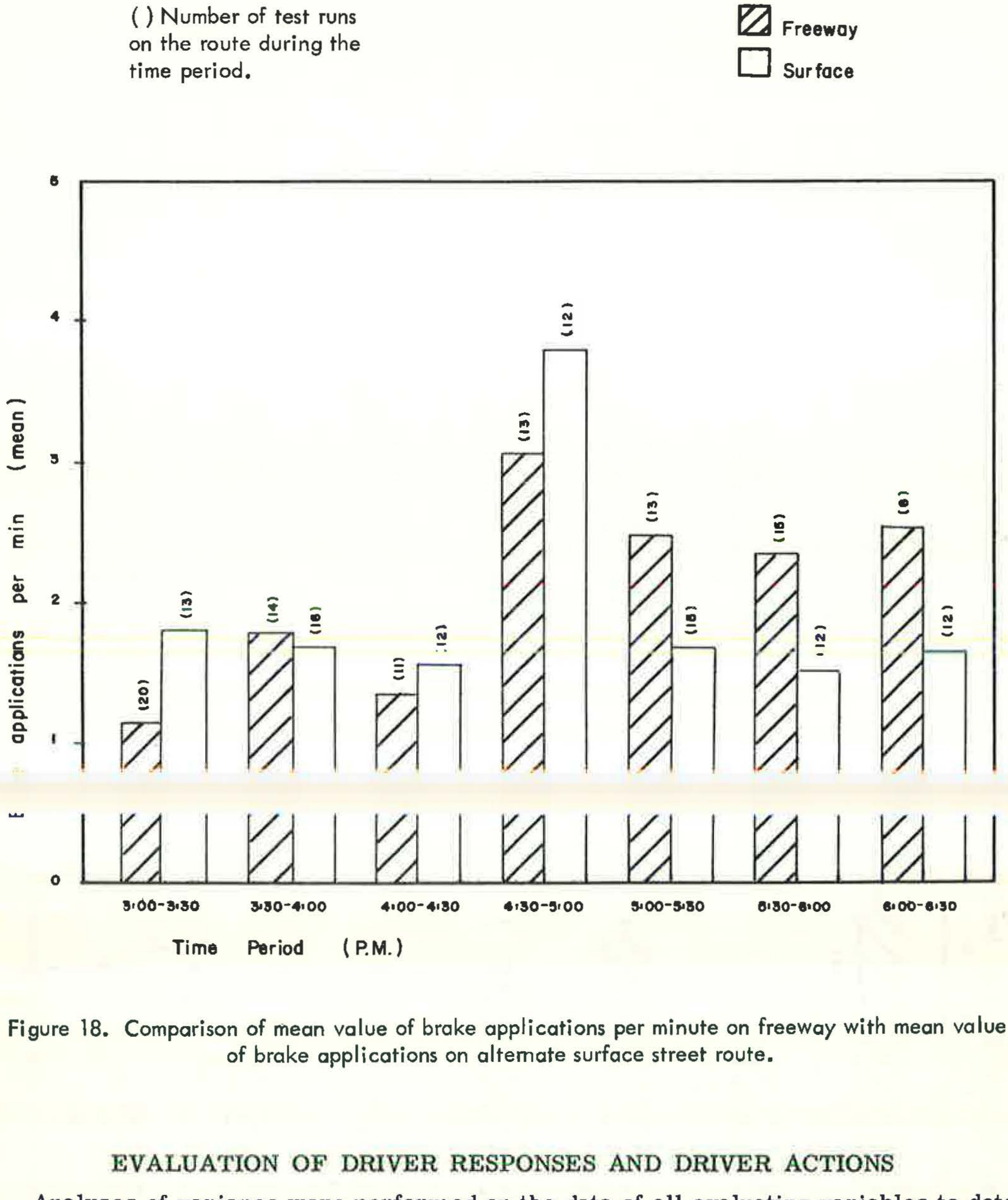


Figure 18. Comparison of mean value of brake applications per minute on freeway with mean value of brake applications on altemate surface street route.

EVALUATION OF DRIVER RESPONSES AND DRIVER ACTIONS

Analyses of variance were performed on the data of all evaluating variables to determine if there were significant differences in mean values for different time periods. This was done for the values on both a per mile and a per minute basis. This hypothesis was found to be true for all variables at the 95 percent level of confidence. Also, t-tests were performed to determine if there were significant differences in mean values of the evaluating variables for the freeway and surface street for the individual time periods.

Figures 6 through 12 show the variation of factors related to driver responses and driver actions by time period (abscissa) on a frequency-per-mile basis (ordinate) on both freeway and surface street route. Results of the significance tests are given in Table 4.

Figures 13 through 18 show the variation of factors related to driver responses and driver actions by time period (abscissa) on a frequency-per-minute basis (ordinate) on

both freeways and surface street route. Results of the significance tests are given in Table 5.

Speed Changes per Mile

With the exception of the 5:30 to 6:00 p.m. period, the speed change rates have no significant differences for the two facilities. From 5:30 to 6:00 the freeway has a significantly higher speed change rate. Figure 6 shows little relationship of speed changes to traffic volume. This can be explained by the fact that the same volume (at upper levels) can occur under fluid and congested flow. Figure 6 shows closer relationship of speed changes to density.

Steering Reversals per Mile

Steering reversals per mile were significantly higher for the surface route than the freeway from 3:00 to 4:30 p.m. (Fig. 7). From 4:30 to 5:30 there was no significant difference in the steering reversal rate. From 5:30 to 6:00 the freeway had a significantly higher steering reversal rate than the surface street. This evidence supports the conclusion that more steering tasks are required on a freeway during rush hours than on a surface street although the reverse is true under normal conditions; the relationship of this action to density seams very close and supports the value of future studies in this area.

Frequency of Responses per Mile

A GSR-derived measurement shows that the frequency of driver tension responses was significantly low on the freeway from 3:00 to 3:30 p.m. (Fig. 8). From 3:30 to 5:30 there was no significant difference. From 5:30 to 6:00 there was a significantly higher number of responses on the freeway than the surface street route. From 6:00 to 6:30 there was no significant difference. The close relationship of the frequency of response graph to the steering reversal graph bears careful consideration. A fine analysis of some recordings of these two measurements indicates that the steering reversal action follows a short time after the GSR response.

Trip Time per Mile

The mean travel time per mile on the freeway varied from a low of 1.71 min (3:00 to 3:30 p.m.) to a high of 3.74 min (5:30 to 6:00 p.m.) The variation of mean trip time to travel one mile on the surface street was relatively very small (Fig. 9). This can be explained by control applied through traffic signalization, keeping the travel times uniform from one level of traffic volume to another. The shortest mean travel time per mile was 2.66 min (3:30 to 4:00 p.m.), whereas the longest was 2.99 min (5:00 to 5:30 p.m.)

For the time period 3:00 to 4:30 p.m., trip times on the freeway were significantly lower than the surface street. From 4:30 to 5:30 there was no significant difference. From 5:30 to 6:00 the mean trip time on the freeway was significantly higher than the surface street. From 6:00 to 6:30 the mean trip times on both facilities were about equal.

Accelerator Reversals per Mile

These reversals increased progressively from 3:00 to 6:00 p.m. for the freeway (Fig. 10). The variation of the accelerator reversal rate on the surface route was quite small for the entire period from 3:00 to 6:30 p.m. From 3:00 to 5:00 the accelerator reversals were significantly less on the freeway than the surface route. From 5:00 to 6:00 there was no significant difference; 6:00 to 6:30 showed the surface street again to have significantly higher accelerator reversals.

Magnitude of Responses per Mile

This GSR measurement indicated that from 5:30 to 6:00 p.m. the average magnitude of responses on the freeway was significantly greater than on the surface street (Fig. 11).

For other time periods, there was no significant difference in the average magnitude of responses. For the same time period the freeway produced the greatest driver tensions.

Brake Applications per Mile

The brake application rate per mile on the freeway increased progressively from 3:00 to 6:00 p.m. and decreased from 6:00 to 6:30 (Fig. 12). The variation was relatively small for the surface route for the entire time period. The brake applications were significantly less on the freeway from 3:00 to 3:30. From 3:30 to 4:30 there was no significant difference in the brake applications for the freeway and the surface route. From 5:00 to 6:30 the freeway had significantly higher brake applications than the surface route. Brake applications show a pronounced deterioration of driving conditions on the freeway during rush hours as compared to the surface route.

Steering Reversals per Minute

These reversals were significantly higher on the freeway from 3:00 to 3:30 and from 4:30 to 5:00 p.m. (Fig. 13). For all the other periods, there was no significant difference. There is an indication that the total number of steering reversals is more influenced by time than space as shown by these statistics. In comparing steering reversals on a rate per mile as against a rate per minute, the maximum number of steering reversals per mile occurs between 5:30 and 6:00 p.m., whereas on a per-minute basis this period shows the minimum number. The increase in travel time during the period over which the total number of steering reversals are recorded is not offset by a similar increase in number of steering reversals.

Frequency of Responses per Minute

The freeway was significantly higher than the surface route from 3:00 to 4:30 p.m. [illegible]

mile and a per-minute basis.

Speed Changes per Minute

The freeway had a significantly higher speed change rate from 3:00 to 5:00 and from 5:30 to 6:00 p.m. (Fig. 15). The periods from 5:00 to 5:30 and 6:00 to 6:30 showed no significant difference. The freeway would appear very unfavorable compared with the surface route on this basis, but even at 3:00 p.m. traffic is quite heavy on the freeway although it is moving fluidly. There is also the possibility that the speed change measurement is set too fine and that oscillations in speed are more prevalent in the upper speed ranges found on the freeway. This logically reveals itself more noticeably on a per-minute basis.

Accelerator Reversals per Minute

The surface route has significantly higher accelerator reversals in the 3:00 to 4:30 p.m. period and the 5:30 to 6:30 period (Fig. 16). There is no significant difference between accelerator reversals on the freeway and the surface street in the period from 4:30 to 5:30. There is much greater consistency between accelerator reversals on a per-mile and per-minute basis. This measurement apparently favors the freeway over the surface street. Comparing this to speed changes, there is some indication that accelerator reversals at higher speeds produce more speed changes than at lower speeds. Also, there is a tendency on the part of the driver to work the accelerator more on the surface street.

Magnitude of Responses per Minute

Except for the period from 4:00 to 4:30 p.m. when the freeway was significantly higher than the surface route in magnitude of responses, there was no significant

TABLE 6

SUMMARY OF MEANS AND STANDARD DEVIATIONS FOR DRIVER TENSION AND DRIVER ACTION VARIABLES

Time Period (p.m.)	Route[a]	No. of Runs	SR[b]		SC[c]		TT[d]		FR[e]		MR[f]		AR[g]		BA[h]	
			Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
(a) Per Mile																
3:00-	F	20	38.75	8.69	10.75	4.90	1.71	0.66	7.05	3.14	30.90	18.45	18.24	10.39	2.25	2.28
3:30	S	13	49.90	7.55	12.08	5.79	2.68	0.33	9.09	4.11	36.20	25.04	43.04	10.30	4.82	2.24
3:30-	F	14	39.29	10.65	9.91	4.99	2.20	0.54	8.89	2.62	30.98	13.40	28.60	8.43	4.08	2.49
4:00	S	16	44.46	6.14	8.59	4.03	2.66	0.35	8.90	2.14	37.12	18.98	45.94	7.63	4.47	1.41
4:00-	F	11	41.23	10.15	9.35	3.61	2.30	0.51	9.22	4.28	36.36	19.58	27.26	11.68	5.02	2.39
4:30	S	12	48.67	7.44	7.93	2.85	2.72	0.35	8.04	2.40	27.76	11.67	45.77	13.34	4.26	1.46
4:30-	F	13	48.23	7.53	15.07	4.39	2.44	0.49	9.30	3.37	38.35	20.96	34.77	12.08	7.29	2.39
5:00	S	12	50.59	7.49	13.16	6.61	2.69	0.39	9.36	2.85	36.37	15.57	44.07	11.13	5.66	2.06
5:00-	F	13	57.12	10.13	13.87	5.22	3.28	0.79	11.25	4.17	42.16	23.62	47.27	11.05	8.64	3.65
5:30	S	15	55.21	10.33	12.87	6.52	2.99	0.31	9.83	4.57	33.41	18.29	47.55	11.53	5.11	1.85
5:30-	F	15	58.30	16.46	13.91	6.43	3.74	0.76	15.17	6.74	57.07	27.73	53.23	15.84	9.15	5.19
6:00	S	12	49.25	10.54	8.67	2.65	2.97	0.34	10.42	3.10	40.62	18.24	51.17	8.12	4.58	1.36
6:00-	F	8	48.34	9.36	12.94	6.53	2.71	0.38	10.12	3.62	36.05	19.63	34.60	7.36	6.71	2.69
6:30	S	12	51.75	10.65	11.56	7.38	2.69	0.43	9.12	2.92	32.23	16.87	46.62	11.14	4.42	1.79
(b) Per Minute																
3:00-	F	20	23.34	4.16	6.61	3.28			4.03	1.40	18.86	11.71	10.25	2.63	1.15	0.69
3:30	S	13	19.35	3.05	4.62	2.42			3.26	1.28	16.83	11.21	15.93	2.36	1.80	0.77
3:30-	F	14	17.75	4.19	4.81	3.21			4.14	1.35	14.79	6.95	12.84	1.93	1.79	0.90
4:00	S	16	16.87	2.24	3.26	1.51			3.38	0.89	13.94	7.24	17.35	2.49	1.69	0.53
4:00-	F	11	19.31	4.05	4.78	2.33			4.67	1.56	18.63	8.51	14.03	4.86	1.36	0.77
4:30	S	12	18.23	3.47	3.16	1.38			3.07	1.17	10.79	5.55	17.78	2.92	1.56	0.50
4:30-	F	13	21.38	3.43	6.65	2.61			3.92	1.16	16.93	8.13	14.57	3.42	3.06	0.83
5:00	S	12	19.17	3.42	4.98	2.49			3.47	0.91	13.34	4.69	16.26	2.85	3.80	0.89
5:00-	F	13	18.03	4.14	4.48	2.26			3.45	1.08	12.92	7.34	14.66	2.77	2.49	0.90
5:30-	S	15	18.49	2.23	4.30	2.09			3.30	1.48	11.33	6.31	15.99	3.32	1.68	0.55
5:30-	F	15	15.77	4.02	3.57	1.10			3.66	0.78	14.93	5.73	14.13	3.23	2.35	1.02
6:00	S	12	16.71	3.48	2.87	0.88			3.55	1.14	13.87	6.99	17.25	2.03	1.61	0.35
6:00-	F	8	18.01	3.37	4.87	2.47			3.75	1.29	13.61	7.40	12.85	2.07	2.55	1.04
6:30	S	12	19.46	3.03	4.37	2.88			3.52	1.30	12.42	6.80	17.37	2.78	1.67	0.67

[a] F = freeway route; and S = alternate surface street route.
[b] Steering reversals.
[c] Speed changes.
[d] Trip time.
[e] Frequency of response.
[f] Magnitude of response.
[g] Accelerator reversals.
[h] Brake applications.

difference between the two routes (Fig. 17). The effect of travel time on the magnitude of responses is shown by comparing the values taken on a per-mile and a per-minute basis. The faster the motorist travels, the more responses and hence the more magnitude of responses he can accumulate. Since his trip times are shorter, however, it would be interesting to prove whether his accumulated tension, or accident proneness, is greater or less with a short travel time over a fixed distance compared to a slower trip. These statistics support the belief that tension is a function of speed only because the driver increases his frequency of events. By decreasing speed on the same route, he increases travel time, but from findings in this study he increases his number and magnitude of responses when spread over the longer travel time involved.

Brake Applications per Minute

The surface route was significantly higher in brake applications than the freeway from 3:00 to 3:30 p.m. (Fig. 18). There was no significant difference from 3:30 to 5:00. The freeway was significantly higher than the surface street from 5:00 to 6:30. Here also, there was a similarity on a comparison basis when based either on a per-mile or a per-minute rate. There is a noticeable increase in the amount of braking applications on both the freeway and surface route from 4:30 to 5:00. This may be explained by there being higher traffic volumes at higher speeds on the freeway and thus more opportunities to brake in a short time period. As the freeway traffic slows down, the brake applications per minute lessen. On the surface street where the speeds are more uniform from one time period to another, there is more roadside interference caused by departures from work, etc., to account for this.

A summary of means and standard deviations for driver tension and driver action variables on per-mile and per-minute bases is given in Table 6.

SERVICE INDEX AND COMFORT INDEX

Previous efforts by Greenshields to determine the quality of traffic flow index num-

of the change of speed and change of direction (7). In this study, all the variables related to the driver response process have been taken into consideration and placed into two separate categories, service index and comfort index.

The reasoning behind two means of comparison is that a road facility may be more comfortable to drive at certain times even though the travel time per unit distance may be longer. To take this into account, the comfort index does not include the variable identified as trip time in its determination.

The comfort index is derived by summing the normalized scores of the following variables: (a) steering reversals, (b) speed changes, (c) accelerator reversals, (d) brake applications, (e) number of responses, and (f) magnitude of responses.

The service index adds the travel time variable to its determination; however, it is given a weight equal to the other six variables. This, of course, is arbitrary as is the composition of the two indexes. Since travel time is an important economic factor, it is felt that this weighting is justified. In the future it may prove to be erroneous in magnitude, but should show trends which are all-important at this stage of analysis.

Both the service and comfort index variables are given on a per mile basis. This is consistent since the time variable is isolated by using two indexes and, further, the length of a trip is a constant whereas the time of the trip is a variable. This variable is also influenced by the other variables under consideration.

Each of the variables used in the determination of the two indexes is transformed into a normalized score (N.S.) represented by

$$\text{N.S.} = \frac{\overline{X} - X}{S}$$

where

$\overline{X}$ = Mean value of all runs made on freeway and surface street route from 3:00 to 6:30 p.m.;

X = Mean value for all runs made during a specific time period on one of the route facilities; and

S = Standard deviation.

This equation measures the deviation from the mean in units of the standard deviation.

The measure units comprising the variables of both the service and comfort index, although different, are consistent in one respect. In all cases, lower values represent better service, i.e., fewer speed changes, shorter travel time, and fewer brake applications.

Table 7 gives normalized scores of all the variables considered in the determination of two indexes for different time periods.

A value of zero for the service or comfort index would indicate that the conditions on the facility for that time period are exactly equal to the average condition. Table 7 indicates this to be around the beginning of the peak period (about 3:00 p.m.) for comfort index on the freeway and the end of the peak period (about 6:30 p.m.) for both indexes on both routes. A positive value indicates that the facility is operating better than average while a negative value indicates that it is operating below average. Since conditions found both on the freeway and surface routes at the zero level are acceptable to the motorist, this is used as a satisfactory level of driver acceptance of quality.

An examination of the comfort index values from Table 7 indicates that in the period from 3:00 to 4:00 p.m. both the freeway and surface street have comfort index above satisfactory level. The freeway is relatively more comfortable. From 4:00 to 4:30 the

TABLE 7

NORMALIZED SCORES, COMFORT INDEX AND SERVICE INDEX

Route	SR[a]/Mi	SC[b]/Mi	TT[c]/Mi[d]	NR[e]/Mi	MR[f] /Mi	AR[g]/Mi	BA[h]/Mi	CI[i]	SI[j]
(a) 3:00-3:30 p.m.									
Freeway	+1.091	+0.141	+8.634	+0.828	+0.322	+2.108	+1.307	+5.797	+14.431
Surface	-0.243	-0.111	-0.366	+0.136	+0.026	-0.281	+1.183	+0.710	+ 0.344
(b) 3:30-4:00 p.m.									
Freeway	+0.839	+0.307	+5.112	+0.290	+0.437	+1.369	+0.462	+3.704	+ 8.816
Surface	+0.614	+0.707	0	+0.350	-0.015	-0.760	+0.539	+1.435	+ 1.435
(c) 4:00-4:30 p.m.									
Freeway	+0.690	+0.579	+4.236	+0.100	+0.025	+1.111	+0.094	+2.599	+ 6.835
Surface	-0.059	+1.232	-1.026	+0.670	+0.778	-0.422	+0.664	+2.863	+ 1.837
(d) 4:30-5:00 p.m.									
Freeway	0	-0.827	+2.694	+0.010	-0.072	+0.445	-0.861	-1.305	+ 1.389
Surface	-0.315	-0.260	-0.462	+0.102	+0.030	-0.353	-0.209	-1.005	- 1.467
(e) 5:00-5:30 p.m.									
Freeway	-0.878	-0.466	-4.710	-0.384	-0.225	-0.645	-0.934	-3.532	- 8.242
Surface	-0.676	-0.219	-6.390	-0.039	+0.188	-0.643	+0.065	-1.324	- 7.714
(f) 5:30-6:00 p.m.									
Freeway	-0.612	-0.384	-8.526	-0.819	-0.730	-0.826	-0.755	-4.126	-12.652
Surface	-0.097	+1.045	-5.472	-0.248	-0.207	-1.358	-0.477	-0.388	- 5.860
(g) 6:00-6:30 p.m.									
Freeway	-0.12	-0.230	-0.792	-0.130	+0.04	+0.753	-0.550	-0.129	- 0.921
Surface	-0.331	-0.016	-0.420	+0.182	+0.273	-0.583	+0.453	-0 475	- 0.895

[a]Steering reversals.
[b]Speed changes.
[c]Trip time.
[d]Values shown are 6 times normalized scores.
[e]Number of responses.
[f]Magnitude of responses.
[g]Accelerator reversals.
[h]Brake applications.
[i]Comfort index.
[j]Service index.

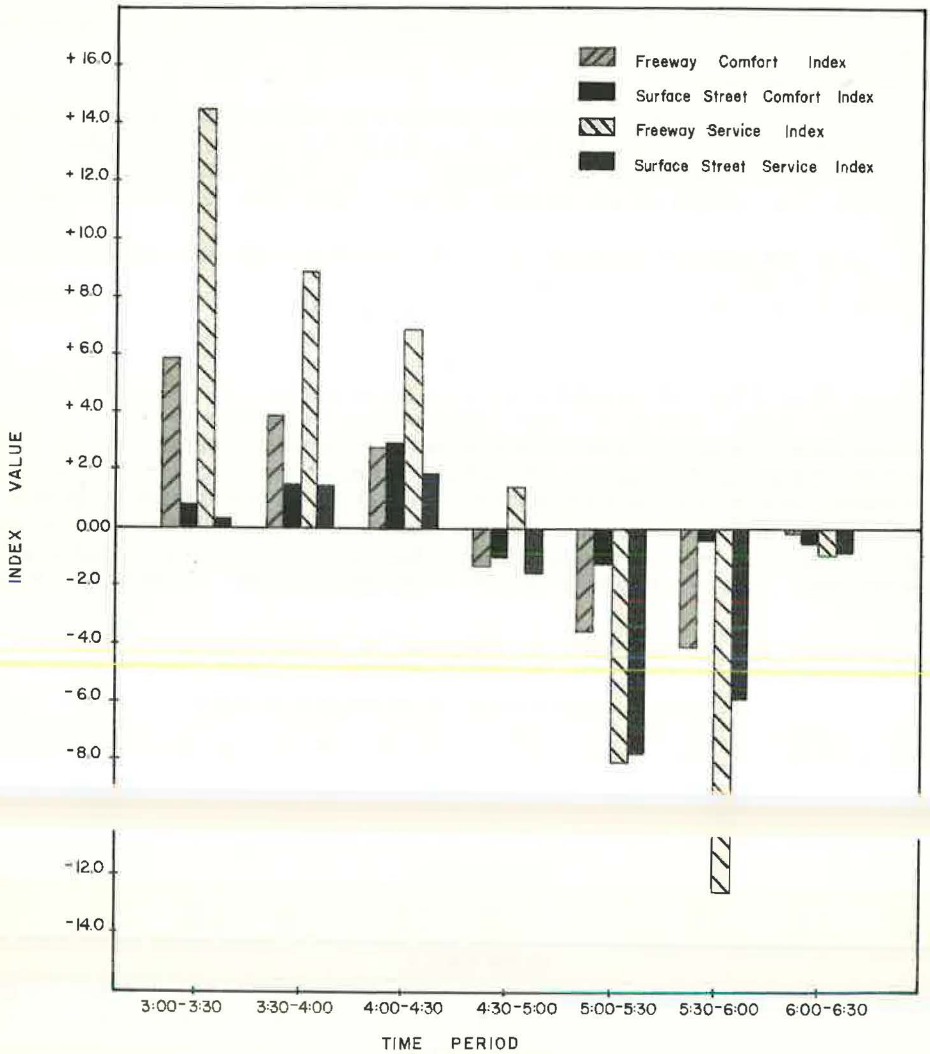


Figure 19. Comparison of service index and comfort index of freeway and alternate surface street route.

freeway and surface street are operating above the satisfactory level and are equally comfortable.

After 4:30 the effect of peak traffic sets in and both the freeway and surface street drop below a satisfactory comfort level, which remains until 6:30. From 4:30 to 5:00 the freeway and surface route operate at approximately the same comfort level, which is slightly less than satisfactory. From 5:00 to 6:00 the surface street is relatively much more comfortable to drive than the freeway. From 6:00 to 6:30 the freeway and surface street operate at about the same comfort level, which is just below satisfactory.

An analysis of the service index, which gives weight to shorter travel time, shows that from 3:00 to 5:00 p.m. the freeway has a superior level of service to the surface route. The freeway deteriorates at a faster rate than the surface route as traffic volumes increase. From 5:00 to 5:30 both facilities operate below satisfactory level with their service indexes about equal.

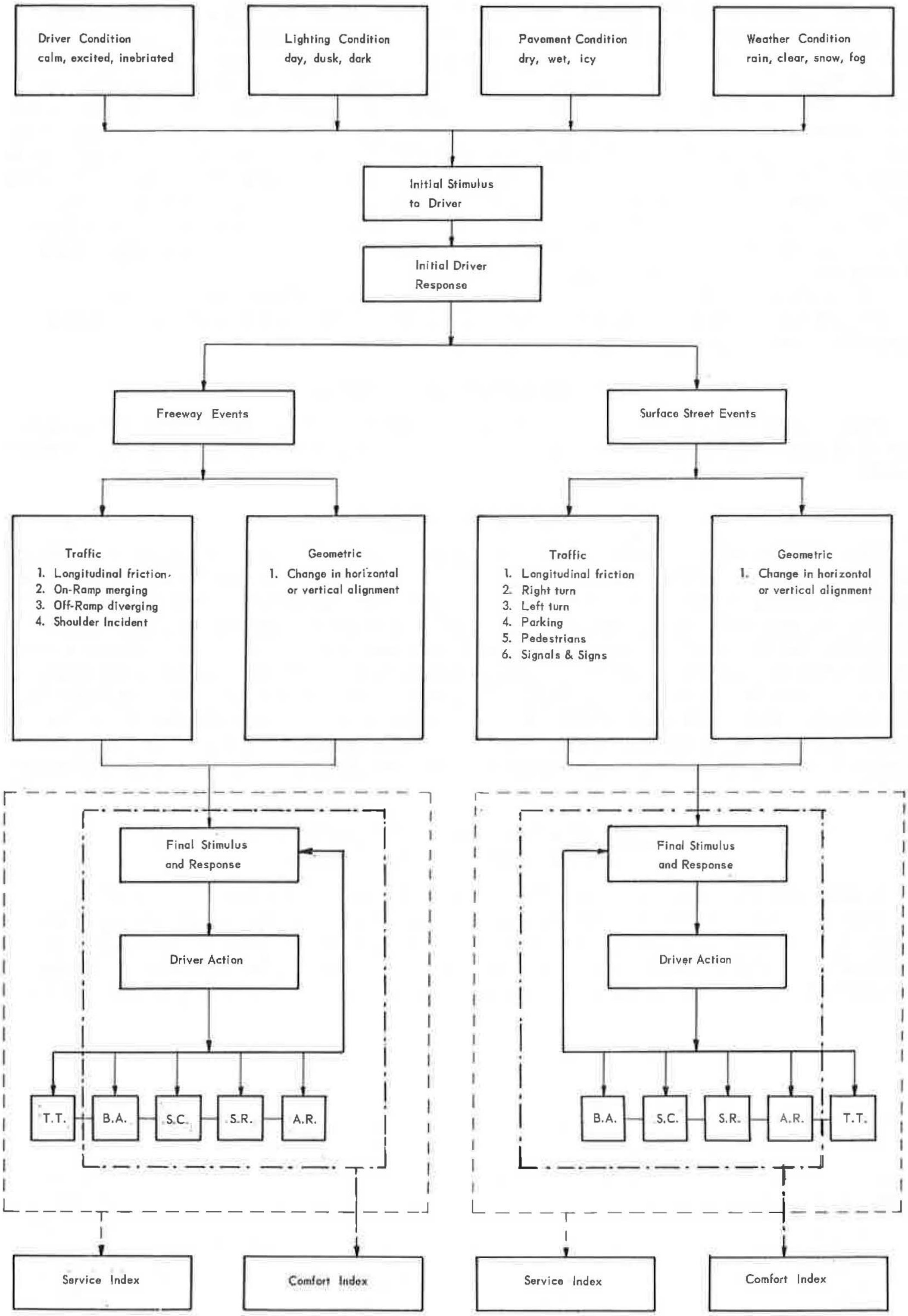


Figure 20. Driver response process.

From 5:30 to 6:00 the freeway service index deteriorates badly while the surface street service index actually is improving. This phenomenom is apparently caused by the metered flow of traffic on the surface street as compared to the unrestricted flow on the freeway. The traffic signal system on the surface street prevents the traffic volume from exceeding the capacity of the roadway if a particular section is contained somewhere along the route beyond the source of traffic generation. The freeway does not have any means of regulating the flow of traffic without control or metering through bottlenecks. This factor coupled with the lesser traffic volumes which can pass a point on the freeway under conditions of congestion bring about the condition described.

With reference to control on the freeway, it is important to mention that no effort to regulate traffic flow on the John Lodge Freeway was made during this experiment although this capability does exist.

From 6:00 to 6:30 p.m. both facilities operate near a satisfactory service.

Figure 19 shows the variation of the comfort index and service index for the two route facilities during different time periods.

DRIVER RESPONSE PROCESS

Figure 20 summarizes all the conditions, factors, and processes by which the material of this study was brought to conclusion, in the form of service index and comfort index.

ACCIDENT RATES

The accident rates on both freeway and surface routes were examined with the hope that variations in accident rates could be associated with either some or a combination of the variables studied in this report. Unfortunately, the length of the routes chosen and the classification of accidents on an hourly basis did not permit detailed, valid analysis. An examination was made on a broad basis whereby several surface routes which could be considered the equivalent of the surface route used in the study along

The trends were in the same direction as the service and comfort indexes indicated. This evidence, along with the assumption that driver tension may be an accurate predictor of potential high accident locations, should be probed more thoroughly in future studies.

TRAFFIC VOLUMES AS RELATED TO SERVICE AND COMFORT INDEX

Traffic volumes were recorded during the study period at midsection locations of freeway and surface street. The average volumes for the study period are given in Table 8. The traffic on the freeway showed gradual increase from 3:00 to 4:00 p.m. From 4:00 to 5:00 it carried approximately 5,600 vehicles (1,866 vph/lane). During this period the demand and practical capacity of the freeway are nearly equal. From

TABLE 8

AVERAGE TRAFFIC VOLUMES RECORDED ON FREEWAY AND SURFACE STREET DURING STUDY PERIOD

Time Period (p.m.)	Freeway	Surface Street
3:00-3:30	2,510	320
3:30-4:00	2,620	450
4:00-4:30	2,800	495
4:30-5:00	2,795	600
5:00-5:30	2,350	710
5:30-6:00	2,290	720
6:00-6:30	2,490	455

TABLE 9

SIMPLE CORRELATION COEFFICIENTS OF FREQUENCY OF RESPONSES PER MILE WITH DRIVOMETER READINGS

Variable/Mi	Simple Correl. Coeff.
Trip Time	0.4913
Accelerator Reversals	0.4336
Brake Applications	0.3569
Steering Reversals	0.2513
Speed Changes	0.1140

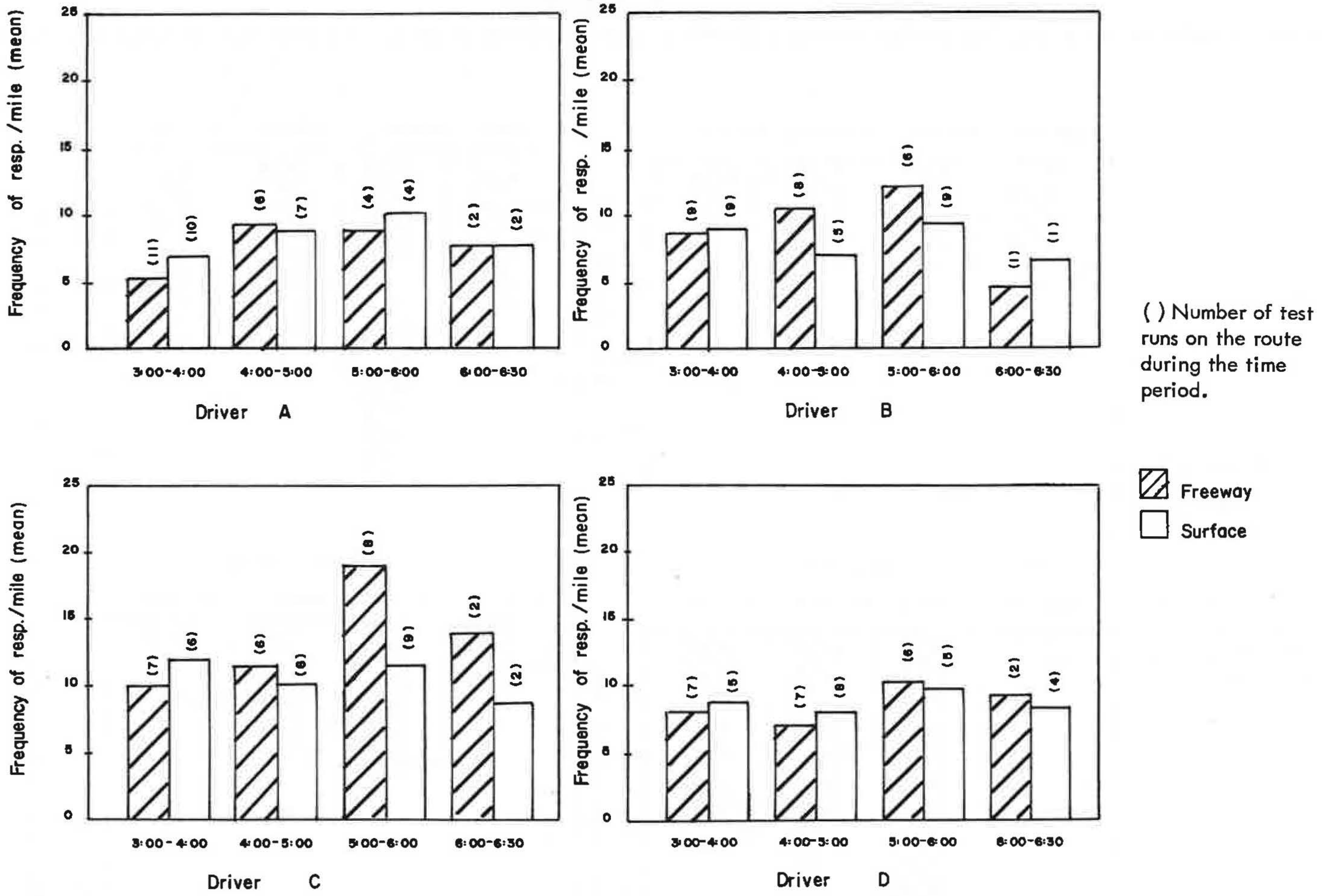


Figure 21. Comparison of mean value of frequency of responses per mile on freeway with mean value of frequency of responses on alternate service street route for different drivers.

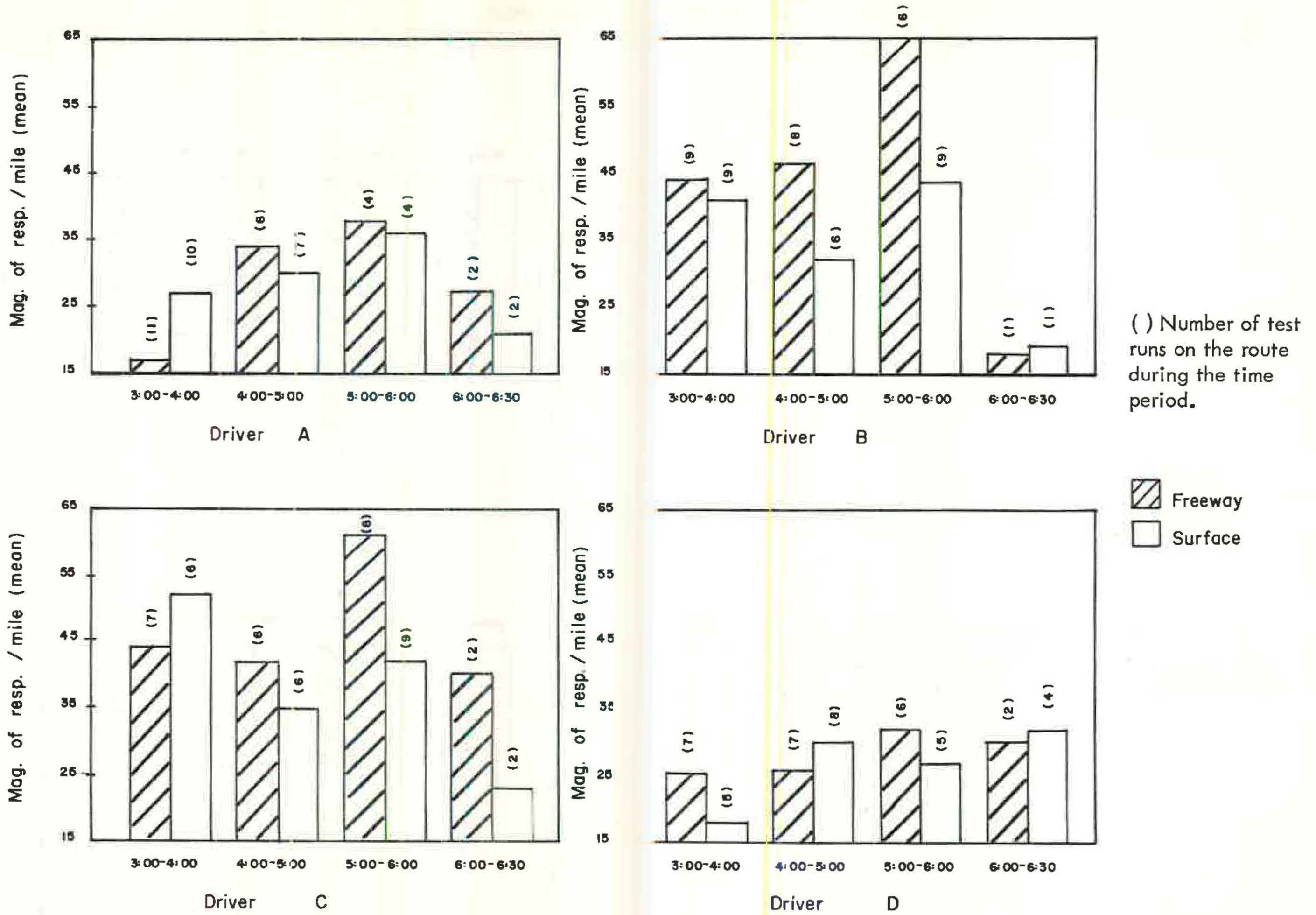


Figure 22. Comparison of mean value of magnitude of responses per mil freeway with mean value of magnitude of responses per mile on alternate surface street e for different drivers.

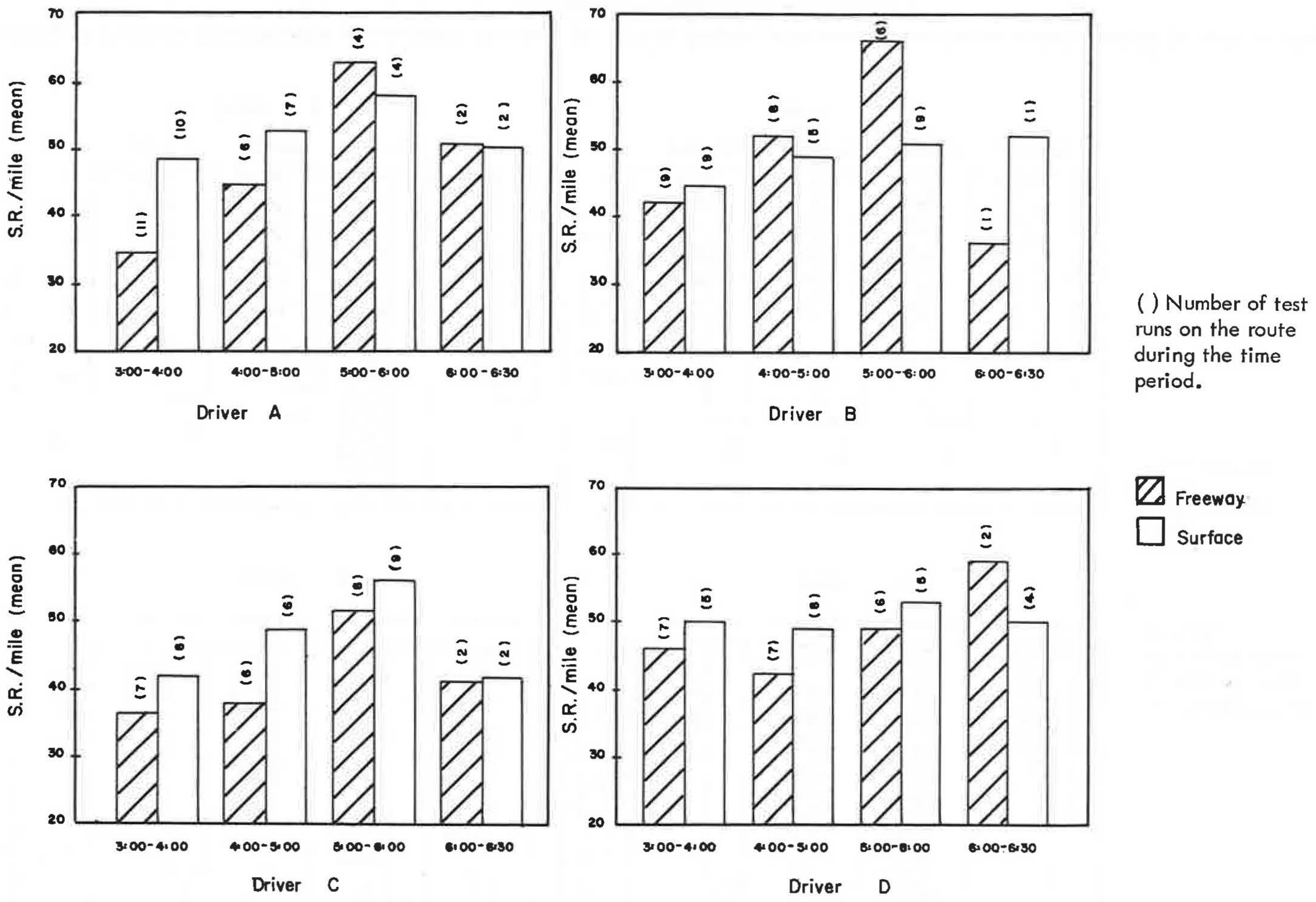


Figure 23. Comparison of mean value of steering reversals per mile on freeway with mean value of steering reversals per mile on alternate surface street route for different drivers.

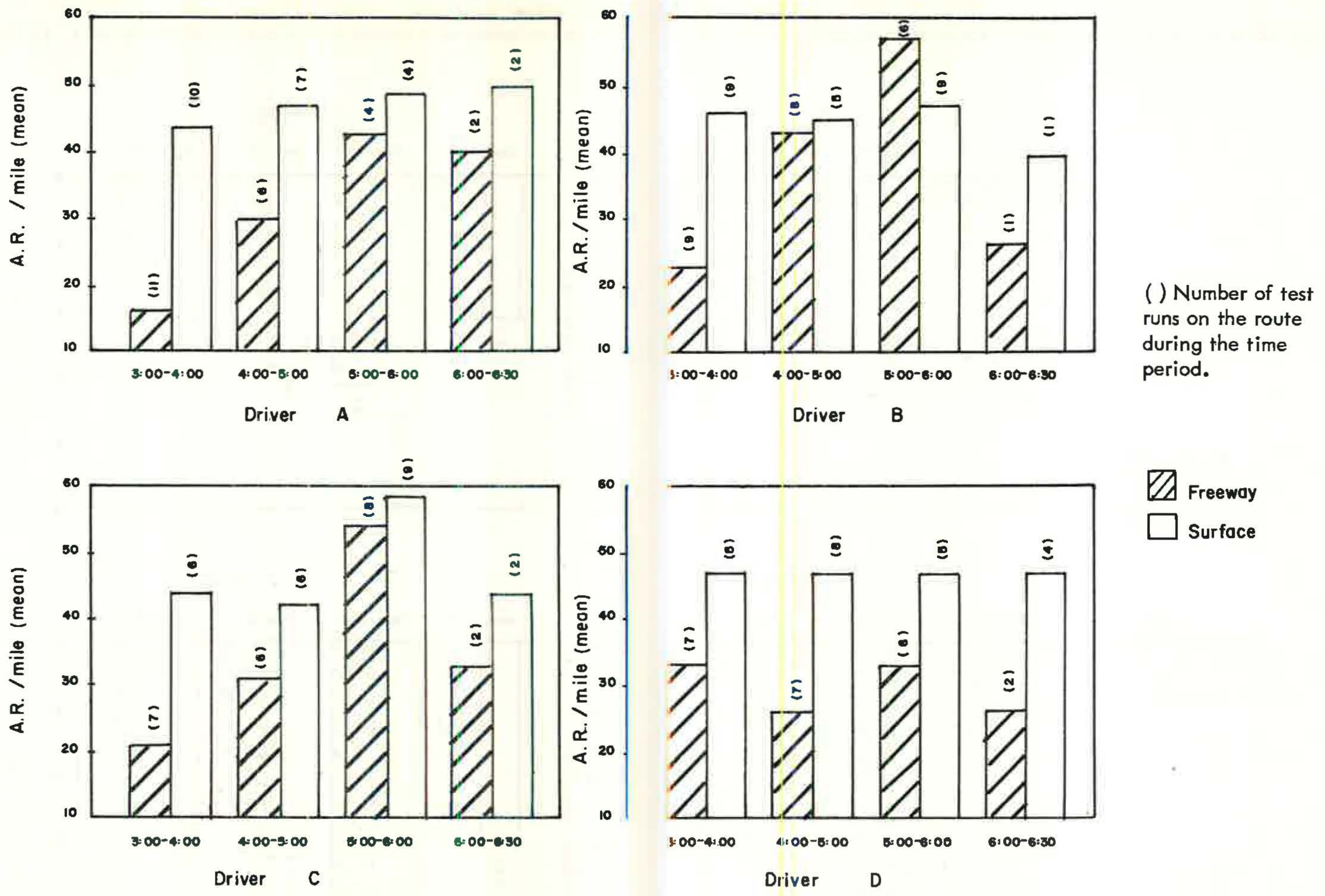


Figure 24. Comparison of mean value of accelerator reversals per'mi freeway with mean value of accelerator reversals per mile on alternate surface stre ute for different drivers.

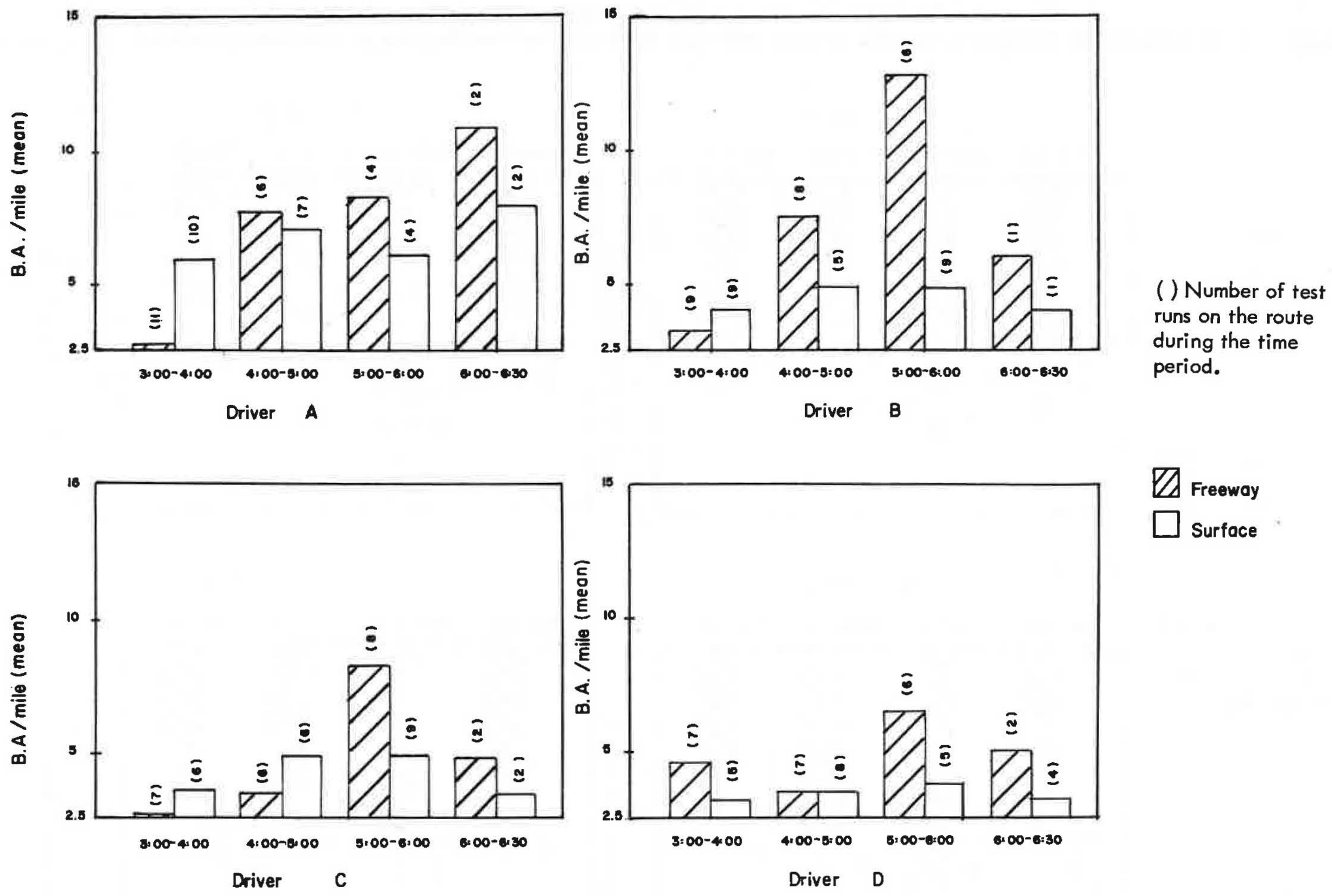


Figure 25. Comparison of mean value of brake applications per mile on freeway with mean value of brake applications per mile on alternate surface street route for different drivers.

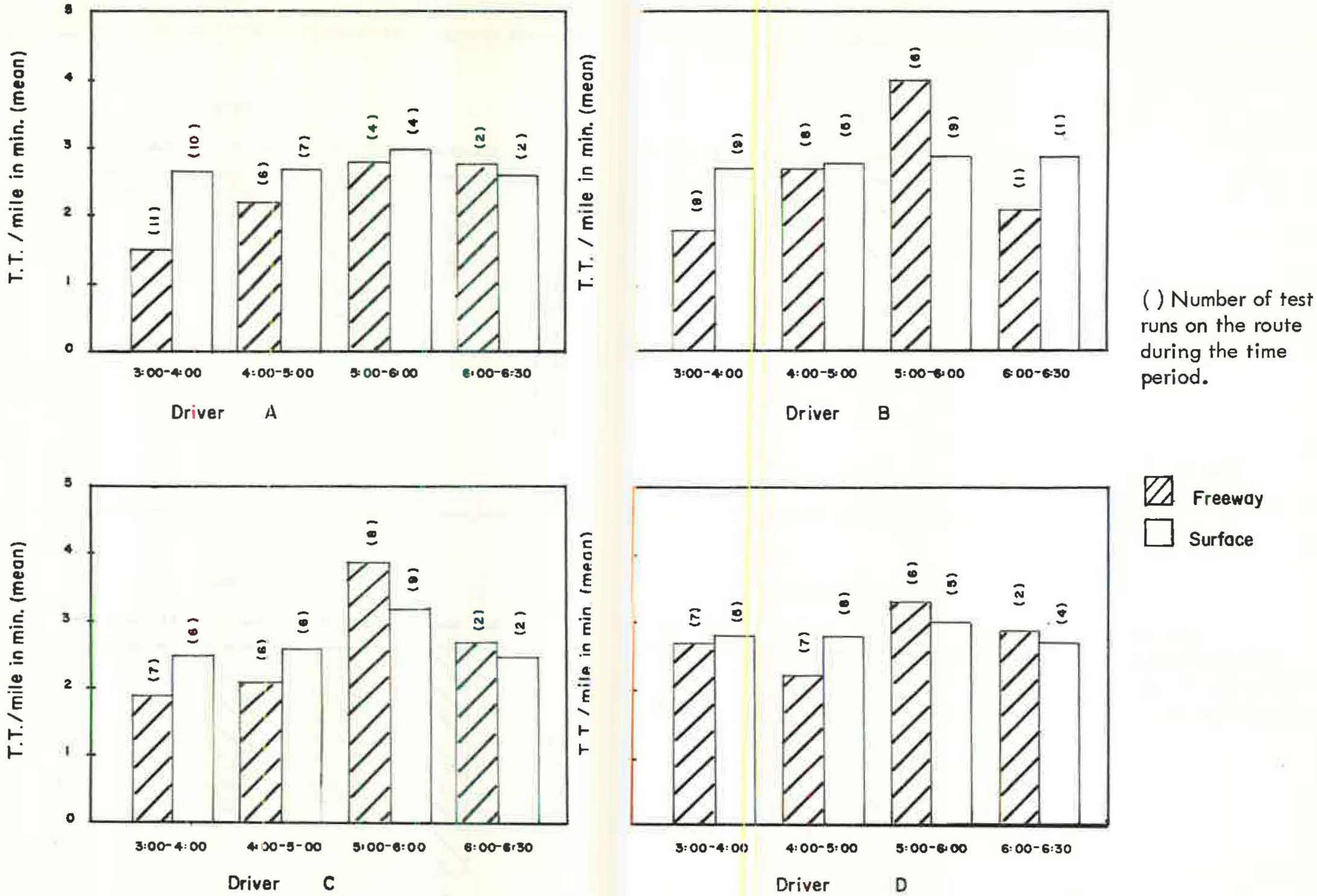


Figure 26. Comparison of mean value of trip time per mile on fre[illegible] with mean value of trip time on alternate surface street route for different driv[illegible]

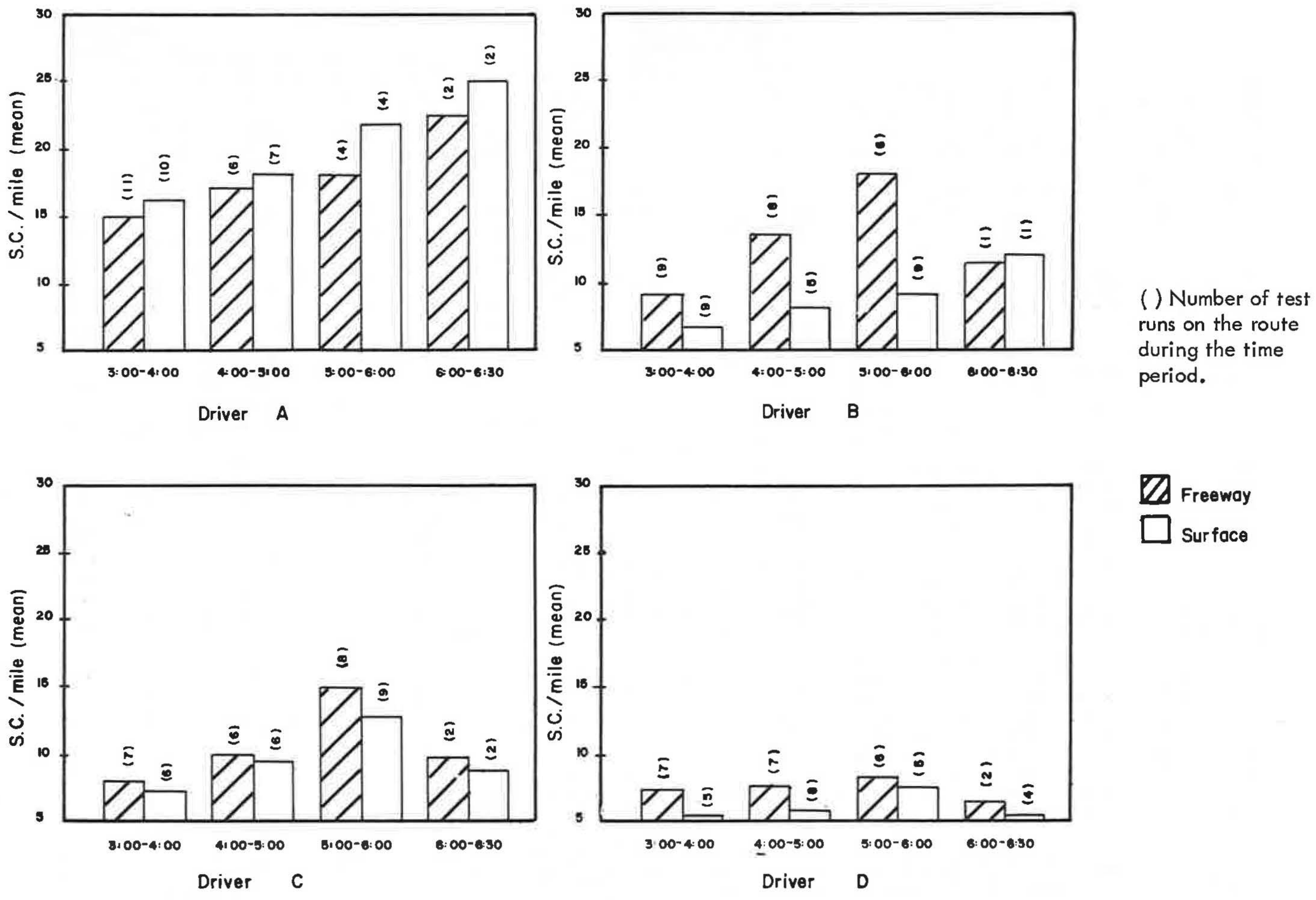


Figure 27. Comparison of mean value of speed changes per mile on freeway with mean value of speed changes per mile on alternate surface street route for different drivers.

BLE 10

SUMMARY OF MEANS AND STANDARD DEVIATIONS FOR DRIVI NSION AND DRIVER ACTION VARIABLES (PER MILE) FOR FOUR DRIVERS

Time Period (p.m.)	Route[a]	No. of Runs	SR[b] Mean	SR[b] Std. Dev.	SC[c] Mean	SC[c] Std. Dev.	T (r Mean	Dev.	FR[e] Mean	FR[e] Std. Dev.	MR[f] Mean	MR[f] Std. Dev.	AR[g] Mean	AR[g] Std. Dev.	BA[h] Mean	BA[h] Std. Dev.
Driver A																
3:00-	F	11	34.75	6.25	15.03	5.02	1.53	1	5.27	1.62	17.27	7.45	16.27	5.73	2.42	2.14
4:00	S	10	48.78	6.98	16.45	3.13	2.65	6	7.06	2.67	27.26	14.44	43.65	11.07	5.92	1.86
4:00-	F	6	44.67	7.36	17.23	4.61	2.18	5	9.40	4.60	34.80	27.21	30.12	12.03	7.72	2.65
5:00	S	7	53.21	4.49	18.16	3.72	2.70	9	8.94	2.59	30.80	13.63	46.51	12.53	7.00	1.66
5:00-	F	4	62.72	3.22	18.02	7.41	2.80	4	9.00	1.15	37.92	25.60	43.00	7.07	8.32	3.89
6:00	S	4	58.15	5.90	21.92	5.58	3.02	3	10.35	4.21	36.40	22.35	48.77	5.54	6.10	1.51
6:00-	F	2	50.95	1.91	22.45	1.91	2.75	07	7.80	1.41	27.50	3.39	40.45	3.32	10.90	0.99
6:30	S	2	50.35	9.55	25.15	3.89	2.60	8	7.80	1.41	21.05	1.91	50.30	3.82	7.85	0.49
Driver B																
3:00-	F	9	42.07	9.96	9.29	3.88	1.84	56	8.78	3.83	43.97	18.70	23.29	12.26	3.34	3.29
4:00	S	9	44.59	6.17	6.80	1.32	2.69	25	9.08	2.43	41.36	17.72	46.09	8.75	4.01	0.94
4:00-	F	8	51.85	5.91	13.76	2.59	2.74	54	10.54	2.48	46.67	16.10	42.82	12.46	7.59	2.39
5:00	S	5	49.10	8.98	8.34	1.30	2.80	34	7.08	2.76	32.34	14.18	45.16	5.45	4.88	1.10
5:00-	F	6	65.57	15.50	18.13	6.53	4.05	87	12.35	4.48	65.73	29.35	57.33	20.19	13.82	4.80
6:00	S	9	51.00	7.87	9.29	1.57	2.88	44	9.51	3.38	43.59	19.60	47.00	10.87	4.90	1.66
6:00-	F	1	36.00	0.00	11.60	0.00	2.10	00	4.80	0.00	18.40	0.00	26.50	0.00	6.10	0.00
6:30	S	1	52.30	0.00	12.20	0.00	2.90	00	6.80	0.00	19.00	0.00	39.40	0.00	4.10	0.00
Driver C																
3:00-	F	7	36.31	5.16	8.07	1.88	1.90	26	10.10	1.65	43.80	13.46	21.27	8.35	2.43	1.36
4:00	F	6	42.03	7.40	7.27	1.05	2.45	37	12.00	2.45	52.33	25.86	44.07	8.29	3.63	0.36
4:00-	F	6	37.83	10.31	10.08	2.13	2.13	37	11.45	4.60	42.38	24.15	31.40	9.91	3.40	2.11
5:00	S	6	49.07	8.74	9.53	1.73	2.57	46	10.18	3.10	35.23	18.35	41.78	17.48	4.88	1.45
5:00	F	8	51.35	12.42	14.97	1.87	3.86	67	19.11	7.40	61.44	24.43	54.32	5.73	8.25	3.97
6:00	S	9	56.13	16.05	12.77	3.03	3.17	30	12.61	2.35	42.17	8.35	58.56	6.89	4.83	1.42
6:00-	F	2	41.10	7.21	9.85	1.48	2.65	35	13.95	0.49	40.15	22.13	32.65	1.91	4.75	0.92
6:30	S	2	41.50	9.62	8.85	1.91	2.50	71	8.80	4.81	23.45	15.91	43.55	11.53	3.40	0.00
Driver D																
3:00-	F	7	45.74	12.72	7.29	2.24	2.69	85	8.06	1.75	25.53	10.39	32.93	11.45	4.56	3.11
4:00	S	5	50.08	9.31	5.30	0.86	2.82	15	8.74	3.64	18.50	11.47	46.64	8.57	3.26	1.74
4:00-	F	7	42.46	11.06	7.61	3.81	2.20	68	7.10	2.69	26.36	9.19	25.74	11.19	3.50	2.86
5:00	S	8	49.29	7.62	5.76	1.74	2.82	27	8.16	4.14	30.51	19.15	46.57	11.10	3.57	0.82
5:00-	F	6	49.18	5.21	8.27	2.27	3.28	84	10.32	2.47	32.53	19.24	42.95	16.85	6.47	2.77
6:00	S	5	52.90	9.56	7.48	1.83	2.98	24	9.66	3.41	26.78	13.17	42.98	11.13	3.80	1.57
6:00-	F	2	58.80	8.20	6.50	2.40	2.90	57	9.20	1.41	29.25	0.92	37.75	12.94	5.10	.42
6:30	S	4	50.32	10.28	5.25	0.30	2.67	39	8.35	2.79	31.62	20.12	44.85	12.35	3.22	..67

[a]F = freeway route; and S = alternate surface street route.
[b]Speed reversals.
[c]Speed changes.
[d]Trip time.

Frequency of response.
Magnitude of response.
Accelerator reversals.
Brake applications.

5:00 to 6:00 the demand on the freeway greatly exceeded the practical capacity, and consequently the traffic-carrying ability literally broke down. The traffic volume recorded during this period was lowest during the entire afternoon rush period. Comparison of traffic volumes with the service index and comfort index indicates that under free-flow conditions increase in traffic volume lowers the value of each index. During congested traffic, even though the volume of traffic carried is low, the service index and comfort index values drop to intolerable levels. Comparison of surface street traffic volumes with the two indexes revealed that the surface street carried the highest traffic volume when the freeway was operating at its lowest efficiency during the period from 5:00 to 6:00 p.m. The values of the two indexes varied directly with the traffic volumes.

CORRELATION ANALYSIS OF GSR AND DRIVOMETER

To examine the relationships of GSR and Drivometer readings, simple correlation coefficients were determined. The correlation coefficients of frequency of responses per mile with the Drivometer readings are given in Table 9.

The analysis of GSR data is very time-consuming because the traffic events have to be separated from other irrelevant events. Also, the events recorded on the chart have to be counted and measured manually. Since the Drivometer variables are recorded in the digital form, the analysis procedure is considerably simpler. The multiple regression technique can be employed to predict the frequency of responses from the significant Drivometer variables.

DRIVER RESPONSES AND DRIVER ACTIONS SEPARATED BY DRIVERS

To evaluate the differences among drivers, the test runs were separated by drivers and grouped into four time periods: 3:00-4:00; 4:00-5:00; 5:00-6:00; 6:00-6:30 p.m. Since separation of test runs by drivers resulted in fewer runs, no attempt was made to determine the statistical significance. These are shown in Figures 21 through 27, on a per-mile basis for different variables. Table 10 gives the summary of means and standard deviations for driver tension and driver action variables for different drivers on a per-mile basis.

CONCLUSIONS

This report discusses the study of seven variables resulting from driver reactions measured through the GSR instrument and his output to his vehicle measured through the Drivometer. These variables are combined into a service index which incorporates all the variables used in the comfort index plus a weighted variable in the form of travel time per mile.

The results indicate that the freeway, although a better route to travel than the surface route when operating below capacity in the low density regions, becomes equal to the surface street when both approach their practical capacity limits. The freeway also becomes a poorer place to drive than the surface route when it operates in the high density region at volumes below practical capacity. These findings are more pronounced when based on a service index reading compared with a comfort index reading.

The variation in service and comfort levels on the surface street is much less than on the freeway. The influence of traffic control primarily created by signalization of the surface route would seem to account for this more uniform condition. This would support the conclusion that the freeway should be allowed to operate under the same conditions which it does today under fluid flow, but control should be exerted during periods of operation near practical capacity to preserve fluid flow and thus lessen driver tension with resulting improvement in operation and safety.

One aspect of freeway control which has already proven to be beneficial to freeway operation, both in Detroit and in experiments elsewhere, is ramp control. The measurements taken on this study which reflects a driver's tension and severity of his driving

task tend to support the conclusion that since the freeway operates at a lower service and comfort level than the surface route during peak travel periods, the driver could be educated to accept an alternate route paralleling the freeway, if properly guided.

This study points out the need for further research involving measurement of human outputs, both emotionally and in task performance. Accident studies should be made and related to these measurements for determining a valid means of predicting accident-producing situations. More driver testing should be performed to show the effects of various driver categories, driving under variations of environmental conditions, effects of design differences for various routes, effects of fatigue, and drinking. This information, when more completely documented, will permit much more scientific designing and planning of operation for a safe and efficient road system.

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