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Foreword

The eleven papers presented in this Research Record are concerned with the traffic characteristics and the operational aspects of freeways. The information given will, therefore, be of special interest to highway department personnel engaged in freeway planning, design, and operations.

The first paper indicates how traffic volume and capacity characteristics affect traffic operations and level of service on freeways. Extensive studies of lane-use distribution are presented. The next paper uses mathematical traffic flow theory to define peak hour traffic congestion and suggests employment of variable speed-limit signs and ramp metering, whereas the third paper provides useful information for freeway control and analyzes use of vehicular density as a freeway traffic control element.

The fourth paper investigates capacity and traffic characteristics for on-ramp and off-ramp operation and traffic distributions on freeway lanes nearby. Fatal accidents on California freeways were analyzed and results presented in the next three papers emphatically point out that only one-half as many people lose their lives on freeways as compared to other roads. One of the studies indicates the possibility further of significant accident reduction on freeways through increasing the number of traffic lanes. Design features and accident frequency are correlated in one of the studies.

The next three papers offer information concerning planning peak period surveillance and control on a Texas freeway; the effects of trucks on operating characteristics of multilane roads and the relationships between number of trucks, speed, and steepness and length of grade; and the operational efficiency, relative safety and general adaptability of left-hand ramps based on conclusions of a Chicago area study.

The last paper in this Record analyzes pedestrian accidents on California freeways over a 5-year period. It was found, among other things, that despite fencing and signing, about 13 percent of all total freeway accidents involve pedestrians.

Taken as a whole, research in this Record greatly increases our knowledge of freeway characteristics and reinforces the widely held belief that properly designed and operated freeways are the safest highways of all.

Contents

Freeway Level of Service as Influenced by Volume and Capacity Characteristics

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•AS APPLIED TO the traffic operation on a particular roadway, level of service refers to the quality of the driving conditions afforded a motorist by a particular facility. Factors which are involved in the level of service are (a) speed and travel time, (b) traffic interruption, (c) freedom to maneuver, (d) safety, (e) driving comfort and convenience, and (f) vehicular operational costs.

Each of the foregoing factors is somewhat related to all the others. The volume of traffic using a facility affects all of the factors and, in general, the greater the volume, the more adverse are the effects. As the ratio of the volume of traffic on a facility to the volume of traffic the facility can accommodate approaches unity, congestion increases. Congestion is a qualitative term, long used by the general public as well as traffic engineers, which refers to what can quantitatively be defined as vehicular density. The end result of an oversupply of vehicles is the formation of a queue of stopped (or "crawling") vehicles at bottleneck locations (a "breakdown" of the operation) such that volumes momentarily drop to zero, leaving only congestion on the facility until a clearout can be effected.

Traffic volumes are known to be continuously variable; even at very low hourly volumes there will be infrequent, short-term occasions when a relatively large number of vehicles will pass a given point. There also are regions on a facility which, due to the geometry, inherently will tend to accommodate fewer vehicles. This implies that bottlenecks do exist and thus the level of service on a given facility may vary even with a "constant" hourly volume. Bottlenecks may be fixed in space due to the aforementioned geometrical considerations of the facility and thus may be studied at the particular location. Such geometrical aspects as entrance and exit ramps have been studied and evaluated as bottlenecks. It is possible also, however, that the random "bunching" of vehicles at any point in space may result in "bottlenecking" due to the statistically variable nature of streams of vehicles, in which case, the designers should be able to predict such peaking characteristics in order to assure acceptable levels of service.

Basically, congestion will be the direct result of the nature of the "supply and demand" on a facility. The supply, in terms of traffic engineering, has been referred to as capacity; the demand placed on the facility is, as it implies, the number of motorists who would seek to use the facility, and can be estimated by origin-and-destination surveys if the times of the desired trips are obtained. It is often futile to measure the flow of traffic on an existing facility with the objective of determining the demand on that facility. About the only relationship between existing volumes and actual demand which can be determined from such measurements is whether the demand is, in fact, as large as the capacity during any significant length of time during a day. In borderline instances, peaking occurring within a peak hour might show where capacity is exceeded by demand for intervals of time less than one hour. Even this feature cannot be exhibited by a traffic system which is so inadequate that it limits (or "meters") the input of vehicles such that the volumes are less than the capacity of the particular facility being studied.

If it is possible, in a given system, for more vehicles to enter than the facility can handle, then congestion will result whenever the demand exceeds capacity and the accompanying inefficiency results in fewer numbers of vehicles being accommodated

Paper sponsored by Committee on Highway Capacity.

by the facility in a given period of time. It is theoretically true that there is some maximum number of vehicles which can use a facility (1). This "possible capacity" is the volume of traffic during the peak rate of flow that cannot be exceeded without changing one or more of the conditions that prevail. From this value more restrictive conditions of roadway and traffic conditions are imposed to describe the measure of "level of service" that a given lane or roadway should provide. If the conditions are associated with highways or streets to be constructed at a future date, it is defined as design level of service. If the conditions express prevailing traffic flow conditions, it is designated as operational level of service.

Various volume levels can cause various levels of operating conditions, or levels of service. For any volume of vehicles using a particular facility, there is an associated level of service afforded these vehicles. It is possible that the input of vehicles into a particular facility will be regulated such that traffic volumes will not exceed a predetermined, suitable level of service volume. Such operational control procedures are being investigated and seem to offer considerable promise.

Although the use of a design level of service volume has considerable appeal in that it conforms to traditional engineering practice, the determination of such a volume, relative to various levels of service, is complex. There are regions on a freeway which are subject to more restrictive vehicular operation, such as in the vicinity of an entrance ramp or exit ramp. Such regions should be considered when determining the design volume of a facility and a knowledge of the operational characteristics and traffic requirements at such locations is necessary for proper planning in order to avoid future bottlenecks.

SCOPE

This report deals with two main topics which affect traffic operation, and thus the level of service, on freeways. They are volume characteristics and capacity characteristics. The capacity of a highway facility is a measure of its ability to accommodate vehicular traffic. This ability depends, not only on the physical features of the road itself, but also on the traffic demand and the interaction of vehicles in the traffic [illegible] should be apparent then, that an appreciation of freeway volume characteristics is important in the planning, design and operation of freeway facilities.

Freeway volume characteristics utilize some of the same parameters defined in the more general subject of volume characteristics. First, the maximum observed volumes on different types of highway facilities, not only provide an indication of the magnitude of traffic demand, but also establish a lower limit of possible highway capacity. The variations in volume for different time periods are explored for their effect in the selection of representative design hourly volumes. These cyclic patterns include monthly, daily, hourly and peak period variations. The distribution of vehicles by direction and lane and the composition of traffic are important design considerations, whereas the longitudinal distribution of vehicles has considerable practical significance as our emphasis gradually shifts from freeway design to freeway operations and control.

Two main topics under volume characteristics are considered and developed. The first is an analysis of such peak flow characteristics as peak rates of flow, correlation with various parameters, and a comparison of peak 2-hr volumes with peak 1-hr volumes. The second main topic under volume characteristics deals with studies of lane distribution of vehicles on freeways with particular attention given to the effect of entering and exiting vehicles on the lane use distributions.

The second section deals with freeway capacity characteristics, including a theoretical approach to providing a rational relationship between capacity and level of service. It is significant that the first section deals with freeway demand and the second with capacity. In the third section, applications are made of these demand and capacity characteristics to freeway design and operations.

VOLUME CHARACTERISTICS

Peaking Characteristics

The need to consider the peak rates of flow within the peak hour has been recognized for several years (2, 3, 4, 5) and has been given considerable attention by the HRB Committee on Highway Capacity.

No matter what criteria are used for the design and operation of a freeway, it is necessary to know what the traffic demand will be. Although no definite limits have been established as yet for the factors affecting the level of service, there will have to be some correlation between peak volume and level of service. An origin-destination survey which denoted, to the nearest five minutes, when motorists would desire to begin their trips, could yield valuable information about the true nature of the existing demand on an urban transportation system. The peak demand periods, however, would likely exceed the economical limits of any system which could be provided. In some instances, depending on the data available, reasonable estimates can be made of future peak-hour volumes. A peak-hour volume does not, however, necessarily imply that a high rate of flow will exist for less than a full hour, more than an hour, or approximately one hour; it is simply an estimate of the maximum number of vehicles expected on a facility during a full 60-min period. Due to the nature of the peak-hour demand and the statistically variable nature of traffic, it is known that short-term rates of flow within the peak hour are often quite variable.

The statistical variability of volumes of traffic is affected by the time period involved. As the time period is reduced, the average number of vehicles for that time period will reduce accordingly. For example, if the average hourly volume were 1,800 vph, the average minute volume would be 30 vpm and the average second volume would be $\frac{1}{2}$ vps based on the hourly volume. The variability of smaller mean values is greater than that of larger mean values when expressed as a percentage of the mean. The narrowing of the confidence interval band for increasing mean values is characteristic of not only the Poisson distribution, which closely approximates the distribution of light volumes of traffic, but also the many other distributions which have been used to approximate various actual traffic distributions. Even the normal distribution exhibits these same characteristics although it is rarely used as an example of existing traffic distributions. Thus, even without the occurrence of a change of volume within a given peak hour, a short-term period within this hour has increased probability of exceeding its mean by a given percent than does the whole hour.

For planning purposes, future volumes are presently estimated for the peak hour or 2-hr period. To relate such volumes into a design peak rate of flow, the factors which affect this relationship must be established and evaluated.

Data from more than 200 freeway traffic studies were obtained from the Texas Highway Department, Bureau of Public Roads (Ramp Capacity Studies), previous studies conducted by the Texas Transportation Institute and specific studies conducted on this project. The relationship between short-period traffic flow (5-min flow) to total hourly flow was determined from these data.

Because the "loading" of freeway in some instances is controlled by the capacity and operation of the supporting street system and inadequate capacity also sometimes limits "unloading" which results in impaired freeway operation, there was not sufficient knowledge of each of the freeways, except those in Texas and a few other specific sites, to permit consideration of these characteristics. It is possible that much better correlations of the results would have been possible had all conditions been known. Those freeways known to have good loading and unloading characteristics showed very good correlation of the data.

Although many characteristics related to trip generation such as geographical and time concentrations of trips, character of the freeway (radial, circumferential, etc.), character of supporting street system, population, area served, and others perhaps have marked effects on the peaking characteristics, it was possible from the data available to study only the relationship of peaking to the population of the city or urban area. The results are shown in Figure 1. These curves are based on the data for 132 peak periods from studies in 31 cities in 18 states. Congestion was not apparent in the

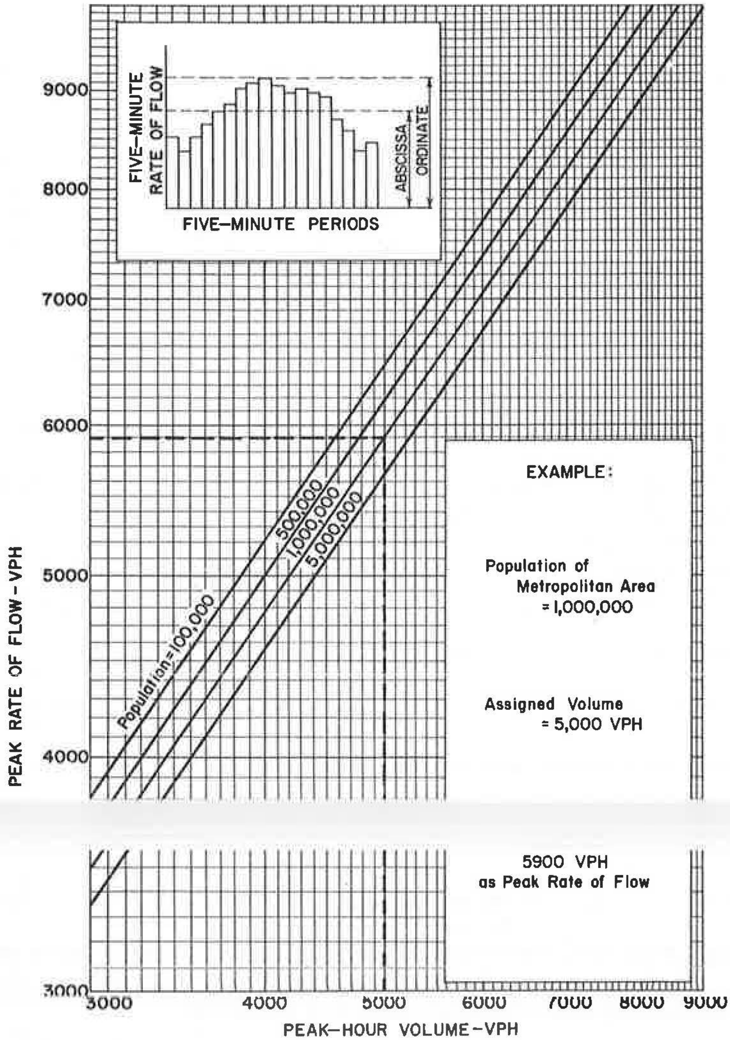

Figure 1. Determination of rate of flow for highest 5-min interval from rate of flow for whole peak hour.

immediate vicinity of any of the study sites. The variables are statistically significant and the curves fit the available data with a standard deviation of 5 percent.

Figure 2 shows the relationship between estimated rates of flow and observed rates of flow and includes a 10 percent error band within which most points were included. Figure 3 shows the frequency distribution of the percent error involved in using Figure 1 to estimate the peak rates of flow. As can be noted, the errors are somewhat normally distributed.

A series of multiple regressions were run on available data in an effort to determine the factors affecting the maximum rate of flow occuring in a peak hour. Consideration was given to the following items: the physical size of the metropolitan area, the concentration of the central business district, the distance of the study site from the main destination or generator (in general the central business district), the population of the metropolitan area as measure of the complexity of the traffic system, and the actual size of the peak period itself, and whether the peak occurred in the morning or afternoon. A few of the cities with multiple study sites indicated a definite relationship with

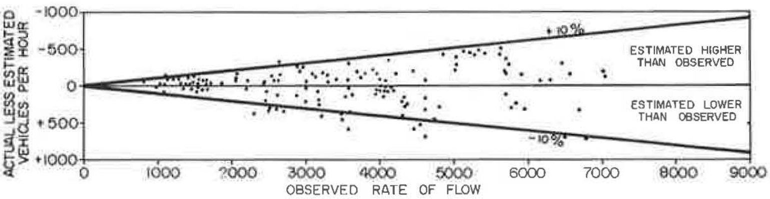

Figure 2. Relationship between error, in vph, and observed rate of flow.

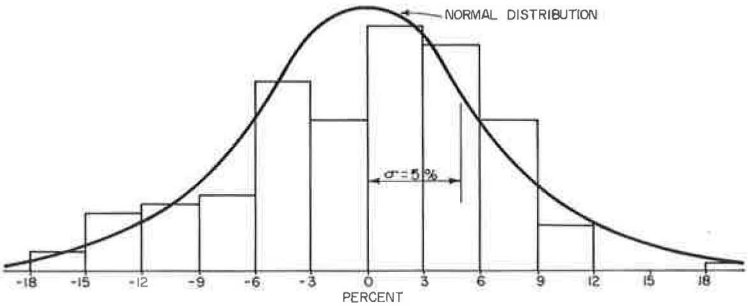

Figure 3. Frequency distribution of percent difference between estimated and observed rates of flow.

the distance from the major generator, while others did not. It was disconcerting to note that this relationship was often contradictory between studies. In the light of such varying results, no relationship could be positively identified. The same was true of most of the other factors.

Improved traffic assignment methods, involving comprehensive programs utilizing large digital computers, are being used to develop predictions of urban traffic volumes for a peak 2-hr period. To establish a relationship between 2-hr peak periods and 1-hr peak periods, the data from the studies mentioned earlier were analyzed. Figure 4 shows the results obtained from 95 studies for which 2-hr peak volumes were available. It can be noted that the peak-hour volume can be expected to lie between 55 and 60 percent of the peak 2-hr volume.

By using the relationships shown in Figures 4 and 1, the design volume can be obtained from the peak 2-hr volume. These design volumes will take into account the peaking effect.

Lane Distribution Characteristics

Critical sections on the freeway often exist adjacent to ramps and, if a certain level of service is to be assured the motorists, it is necessary to give close consideration to such areas in the design of freeways. Because the merging problem directly involves traffic in the outside lane and the entering ramp traffic, a study was made of the percent of total freeway traffic using the outside lane. Only six-lane freeways were considered in this particular project.

A three-part research project was made involving data obtained from 49 study sites located on 14 different six-lane freeways in 10 different states. First, empirical relationships were developed which best fit the data of eight representative study sites. Parameters which were found to be significant were freeway volume, entrance ramp volume, upstream ramp volume, distance to upstream ramp, downstream exit ramp volume, and distance to downstream exit ramp. Second, the relationships which were developed were then used to test against all study sites for validity. The third step was the development of a design procedure which was used for practical applications.

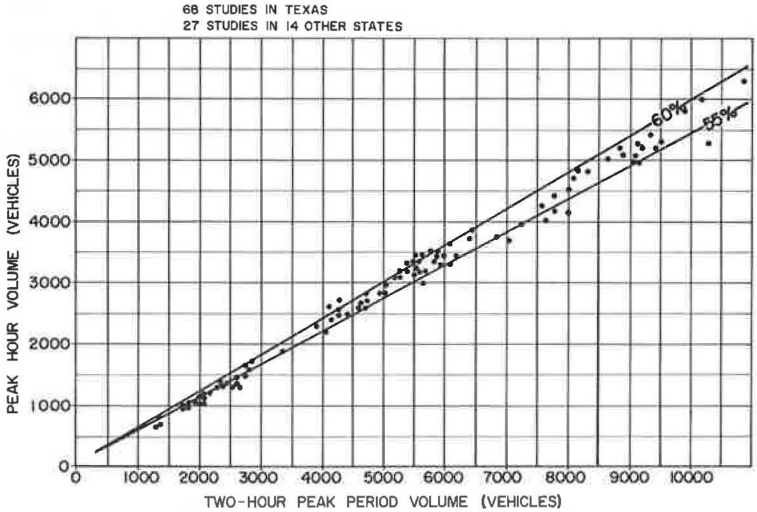

Figure 4. Determination of rate of flow for peak hour from rate of flow for a 2-hr period.

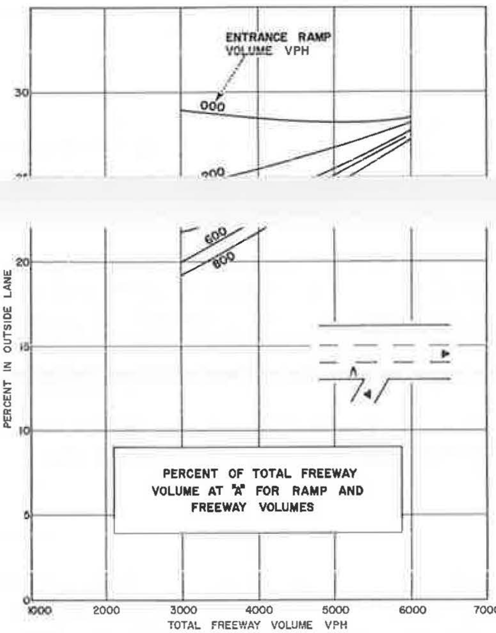

Figure 5. Relationship between percent of total freeway volume in outside lane and entrance ramp volume.

Figure 5 shows the relationship between the percent of the total freeway volume in the outside lane and two of the parameters—total freeway volume and entrance ramp volume. The monographs shown in Figures 6, 7 and 8 represent the relationships between the percent of the total traf- [illegible] and distance to, an upstream exit ramp, a downstream exit ramp, and an upstream entrance ramp, respectively. Distances are referenced to the ramp nose in each case. A downstream entrance ramp was considered to have no effect on the percent of traffic in the outside lane.

The predicted percent in the outside lane can have no less variability than the ordinary or natural variability of the data in general. The 90 percent confidence limits for each of the freeway and ramp volume groups were calculated as follows:

Freeway Volume (vph)	Natural Variability in Outside Lane (%)
2,000-3,000	±8
3,000-4,000	±5
4,000-5,000	±3
5,000-6,000	±2

The "natural" variability of the data was arbitrarily defined as the range within the 90 percent confidence interval. The large natural variability at freeway volumes less

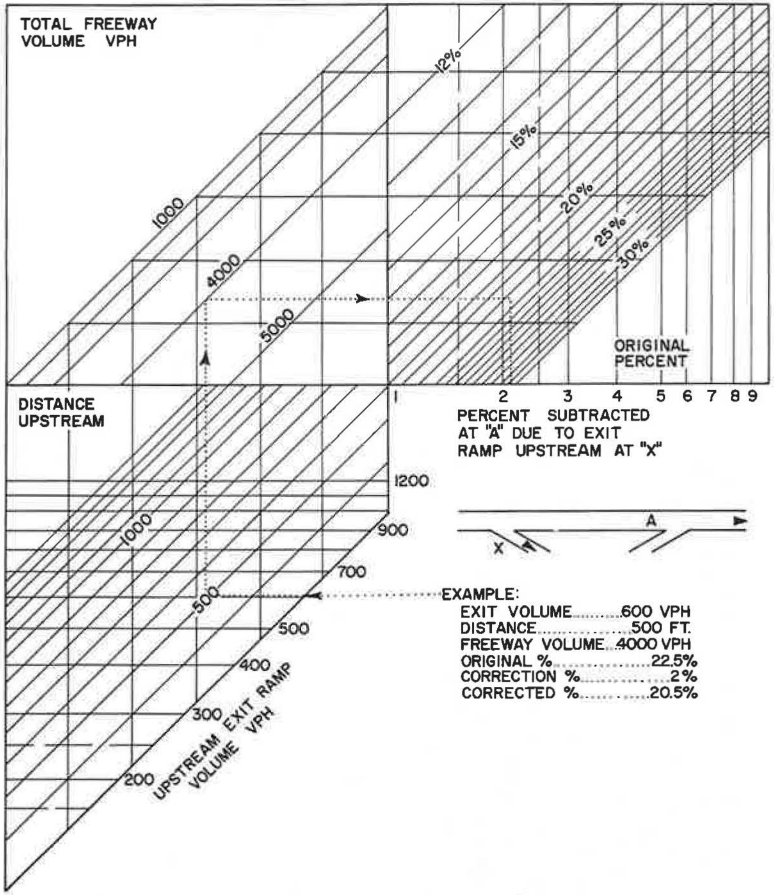

Figure 6. Correction to percent in lane 1 due to exit ramp upstream.

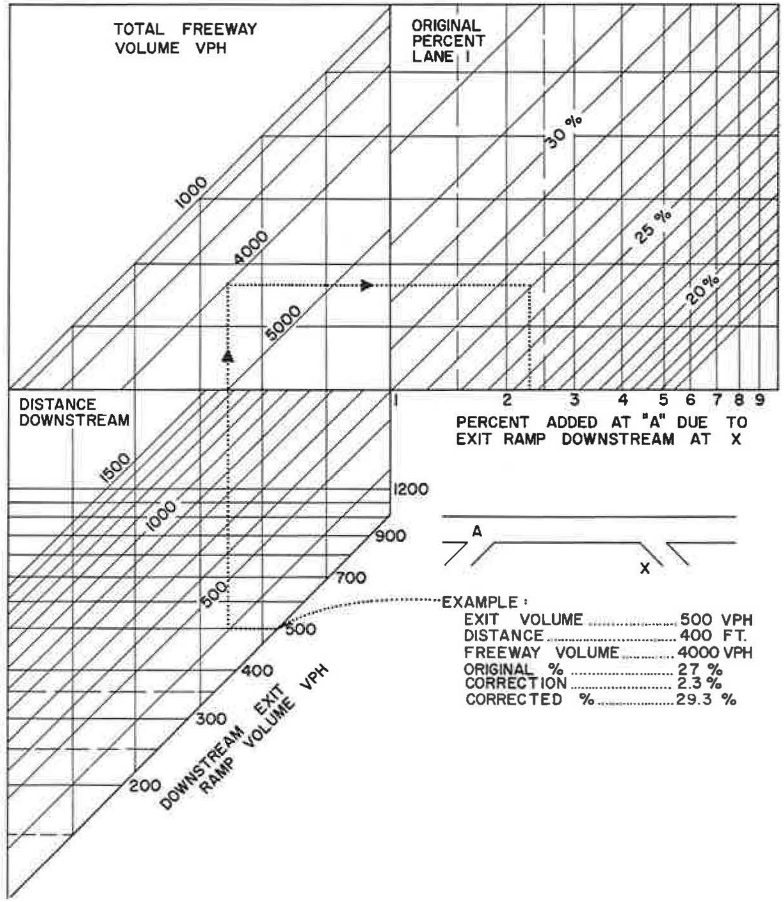

Figure 7. Correction to percent in lane 1 due to exit ramp downstream.

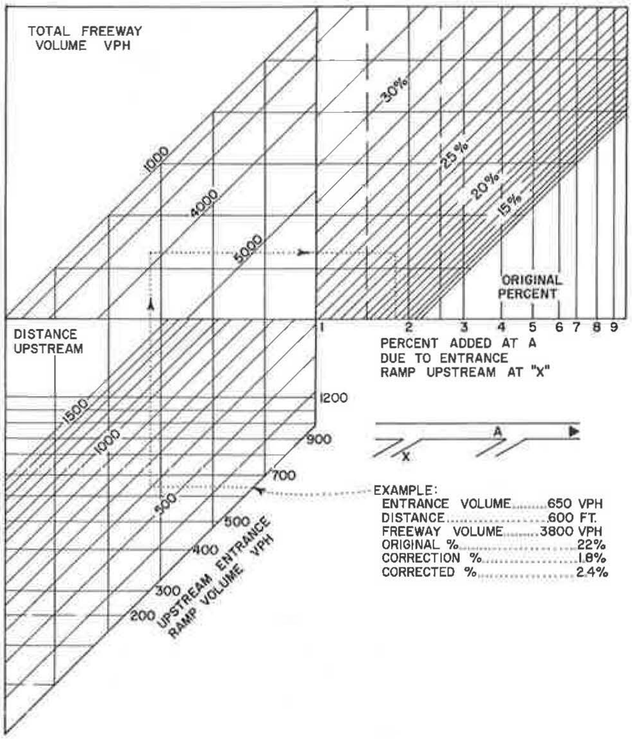

Figure 8. Correction to percent in lane 1 due to entrance ramp upstream.

volume used on six-lane freeways. A comparison of the observed percentages minus the predicted percentages in the outside lane was made for twelve 5-min volumes taken from the 49 study sites (Table 1).

It is intended that the method presented here for six-lane freeways be extended to include four- and eight-lane freeways. In an effort to determine general relationships for four-, six- and eight-lane freeways similar to those shown for only six lanes in Figure 8, data from 132 peak periods were analyzed. These data were the same as those used in the peaking characteristics study and included more than 2,000 five-minute volumes from 31 cities located in 18 states. Multiple regression techniques were used to develop equations from these data and Figure 9 shows the families of curves which were developed from the equations. A multiple correlation coefficient squared, R^2, is generally understood to represent that portion of the variability, or variation, which is accounted for by the derived equations. The obtained R^2s are as follows: four-lane freeways, 0.51; six-lane freeways, 0.70; and eight-lane freeways, 0.48. It is interesting to note that, for the entire range of values shown in Figure 9, the standard deviation was approximately 200 vph. Expressed as a percentage, however, at lower outside lane volumes of 1,000 vph, there would be a standard deviation of 20 percent.

It is known that factors other than freeway volume and ramp volume affect the percent of total freeway traffic in the outside lane. Further studies of those factors are being made so that nomographs similar to those shown in Figures 6 to 8 can be prepared.

As previously mentioned, there is an important relationship between trip length and outside lane utilization. Drivers entering the freeway for only relatively short trips could reasonably be expected to remain in the outside lane. In an attempt to evaluate such relationships, a single study was conducted on the North Central Expressway in

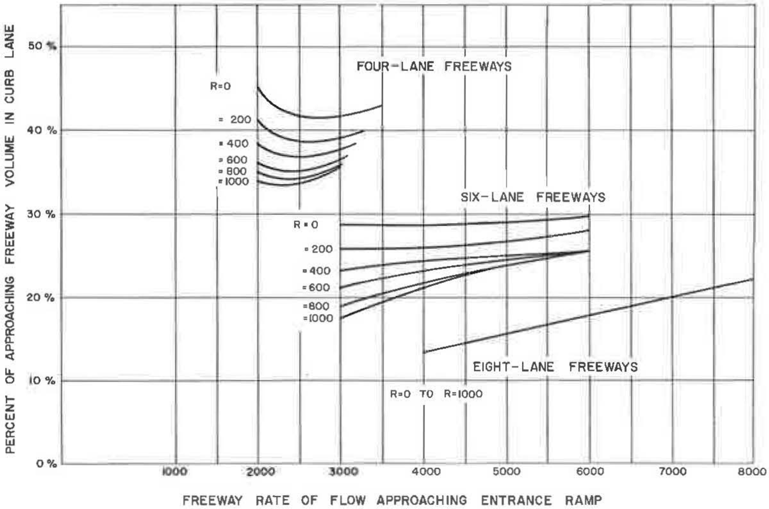

Figure 9. Relationship between percent of total freeway volume in outside lane and freeway and ramp volumes (R = ramp volume).

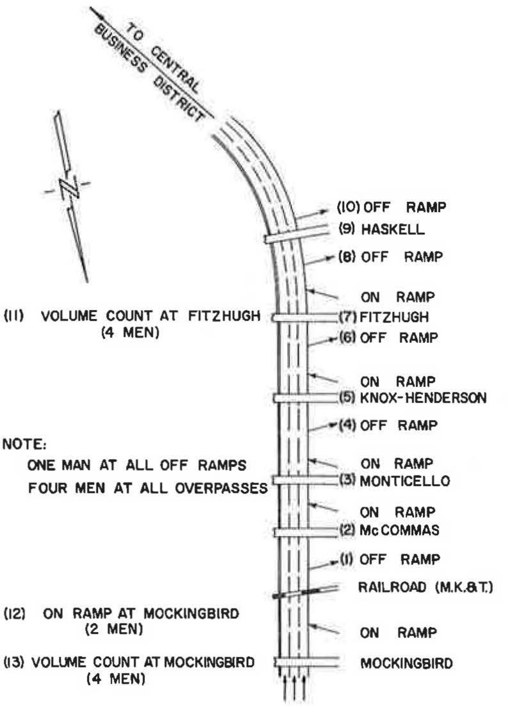

Figure 10. Strip map for observation locations.

Dallas using the "lights-on" study technique previously used by the New York State Department of Public Works and the Port of New York Authority (6).

Figure 10 shows a scheme of entrances, exits, and observation points along the $2\frac{1}{2}$ miles of the six-lane freeway study section. Vehicles entering at the Mockingbird entrance ramp were advised by signs to drive with their lights on for 20 minutes. Policemen were standing near the signs and would call motorists' attention to the message by simply pointing to the signs. Observers were located at strategic points along the freeway, generally on the cross-street overpass structures and at each off-ramp.

As the vehicle entered the freeway, the last four digits of the license number and the vehicle type were recorded. Failure to turn on headlights was noted to determine compliance ratios.

As the vehicle with headlights on moved through the study area, the observer at each point would record the license number, vehicle type and the lane in which the vehicle was traveling. Observers located at each exit ramp recorded the license number of each vehicle with headlights on. Each observer noted the end of each 5-min period on the data sheet.

Freeway volumes were counted, by lanes, at two locations during the entire study period of four hours. One volume count station was located at the beginning of the study section and one at the end of the section. Volumes were recorded in 5-min intervals.

TABLE 1

TABULATION OF THE NUMBER AN GNITUDE OF THE TIMES THE DIFFERENCE BETWEEN
THE OBSERVED PERCENTA ESS THE PREDICTED PERCENTAGE EXCEEDED
THE RAL DATA VARIABILITY

NUMBER	STATE	FREEWAY	NUMBE 5 Min. PERIOD	UMBER OF TIMES THE NATURAL DATA VARIABILITY IS EXCEEDED BY: 1%	1-2%	2-3%	3-4%	4-5%	5-6%	6-7%	7-8%	8-10%	10%
05	Texas	Gulf	24	[illegible]	0	0	0	0	0	0	0	0	0
07	Texas	Dallas	32	[illegible]	2	2	1	0	0	0	0	0	0
12	Texas	Gulf	32	[illegible]	0	0	0	0	0	0	0	0	0
13	Texas	Gulf	32	[illegible]	1	0	0	1	0	0	0	0	0
31	Texas	Gulf	30	[illegible]	0	0	0	0	0	0	0	0	0
33	Texas	Gulf	30	[illegible]	1	2	0	0	0	0	0	0	0
41	Texas	Gulf	36	[illegible]	2	0	0	0	0	0	0	0	0
44	Texas	Gulf	36	[illegible]	0	0	1	0	0	0	0	0	0
0501	Calif.	San Bernardino	35	[illegible]	3	2	1	1	1	2	1	0	0
0503*	Calif.	Harbor	24	[illegible]	2	1	1	3	2	0	0	0	0
0505*	Calif.	Hollywood	30	[illegible]	3	5	5	3	3	1	2	1	1
0609(1)	Colo.	Valley Highway	24	[illegible]	1	0	0	0	0	0	0	0	0
0610*	Colo.	Valley Highway	24	[illegible]	0	2	4	3	2	2	3	1	3
1101	Ga.	Atlanta Expressway	24	[illegible]	0	1	0	1	1	0	0	0	0
1102	Ga.	Atlanta Expressway	24	[illegible]	0	0	0	1	0	0	0	0	0
1103	Ga.	Atlanta Expressway	18	[illegible]	1	0	0	0	0	0	0	0	0
1104	Ga.	Atlanta Expressway	24	[illegible]	4	0	0	1	0	0	0	0	0
1105	Ga.	Atlanta Expressway	24	[illegible]	2	1	1	0	2	0	0	1	0
1106	Ga.	Atlanta Expressway	18	[illegible]	3	2	0	0	1	0	0	1	0
1107	Ga.	Atlanta Expressway	24	[illegible]	3	2	2	1	0	0	0	0	0
1108	Ga.	Atlanta Expressway	26	[illegible]	1	1	0	0	1	1	2	0	0
1109	Ga.	Atlanta Expressway	18	[illegible]	2	1	2	1	0	0	0	0	0
1430*	Ill.	Congress Street	24	[illegible]	5	4	4	2	3	0	0	1	0
1503**	Ind.	Tri-State	30	[illegible]	0	0	0	0	0	0	0	0	0
2313	Mich.	Edsel Ford	28	[illegible]	3	1	0	1	0	0	0	0	0

2317(1)	Mich.	Edsel Ford	35	2	3	2	4	1	2	0	0	0	0
2318(1)	Mich.	Edsel Ford	19	2	3	0	0	1	0	1	0	0	0
2321	Mich.	Edsel Ford	14	1	0	1	1	0	0	0	0	0	0
2322	Mich.	Edsel Ford	15	0	2	3	1	0	0	0	0	0	0
2323(1)	Mich.	Edsel Ford	26	1	0	0	0	0	0	0	0	0	0
2503(1)	Mo.	Intercity	24	0	2	0	0	0	0	0	0	0	0
3101	N.J.	Garden State	30	2	6	1	1	1	0	0	0	0	0
3315	N. Y.	N. Y. State	24	0	0	0	0	0	0	0	0	0	1
222	Texas	Eastex	26	2	1	0	0	0	0	0	0	0	0
131	Texas	Eastex	18	3	2	2	1	0	0	0	0	0	0
221	Texas	Eastex	26	0	4	1	1	0	0	0	0	0	0
130	Texas	Eastex	24	1	1	2	0	0	0	0	1	0	0
3010	Texas	Gulf Freeway	16	0	2	1	0	0	0	0	0	0	0
4020	Texas	Gulf Freeway	15	2	0	2	0	0	0	0	0	0	0
5030	Texas	Gulf Freeway	13	2	2	4	0	0	0	0	0	0	0
5930	Texas	Gulf Freeway	10	0	2	1	1	0	0	0	0	0	0
6050	Texas	Gulf Freeway	16	0	1	1	1	0	0	0	0	0	0
7060	Texas	Central Expressway	13	2	1	1	0	0	0	0	0	0	0
4413*	Texas	Central Expressway	36	2	4	4	0	2	0	0	0	0	0
4414*	Texas	Central Expr.	36	3	1	3	4	1	1	1	2	8	1
R12(1)	Texas	Gulf Freeway	36	3	2	2	1	0	1	0	0	0	0
R13(1)	Texas	Gulf Freeway	36	2	2	1	0	0	0	0	0	0	0
GRAND SUM			1212	83	86	61	38	26	20	8	11	13	6

Percent of the time that the difference exceeds the Natural Data Variability by a percent more than:

0%	1%	2%	3%	4%	5%	6%	7%	8%	10%
29.1%	22.2%	15.1%	10.1%	6.9%	4.8%	3.1%	2.5%	1.6%	0.5%

* Unusual Geometrics (lane dropped or added)
** Freeway Volume is less than 2000 vph
(1) Study with complete adjacent ramp data

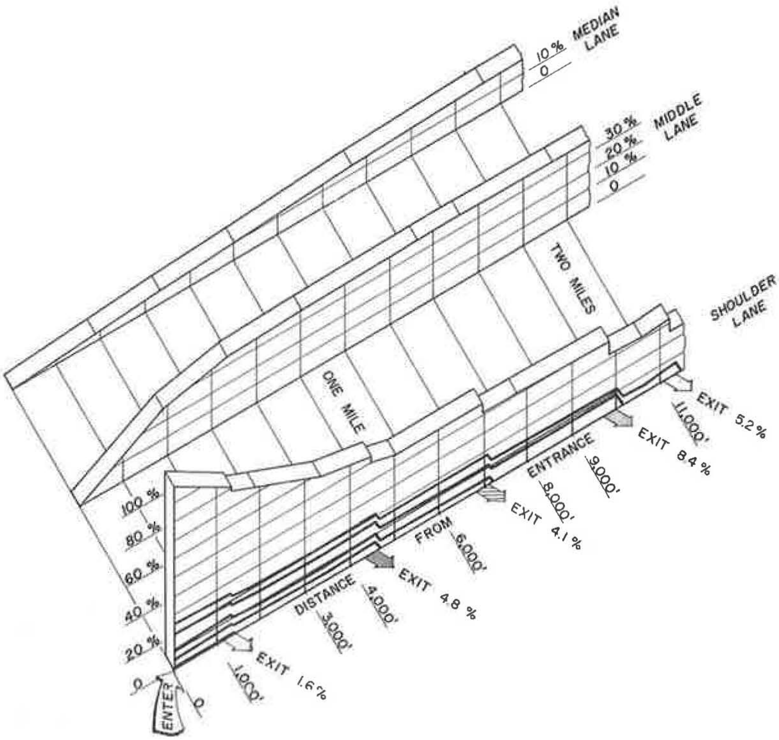

Figure 11. Lane use distribution by vehicles entering at Mockingbird ramp, distribution by percent in each lane, 1,444 vehicles in sample.

During the 4-hr study period more than 1,800 vehicles entered on the Mockingbird ramp and more than 1,500 complied by turning on their headlights. About 1,100 of the complying vehicles did not exit within the study area and about 400 vehicles were noted to exit at one of the five exit points within the study section.

Regarding the study method, the following can be summarized:

1. The "lights-on" study method was an effective means of studying the lane use related to trip length on this particular freeway which had numerous major street overpasses.

2. The presence of the signs, policemen, or observers had no noticeable effect on the traffic flow.

3. Compliance ratios in the 75-85 percent range were obtained by using this method. The sample size was much higher and less expensive to obtain than by other methods considered.

4. Proper design and location of signs is a big factor in obtaining high compliance ratios.

5. A loudspeaker was not essential.

6. A policeman can be used to advantage to encourage motorists to observe the signs.

7. Newspaper, radio, and television publicity can be beneficial, but is not necessary for adequate compliance ratios.

Figure 11 depicts three-dimensionally how the vehicles entering at the Mockingbird ramp were distributed over the three lanes as they traveled toward the central business district. The dark portion at the bottom of the outside lane represents that portion of those entering vehicles which exited within the study area. A large percentage of all such exiting vehicles used only the outside lane and, as a result, those vehicles cannot be noticed in the two inside lanes.

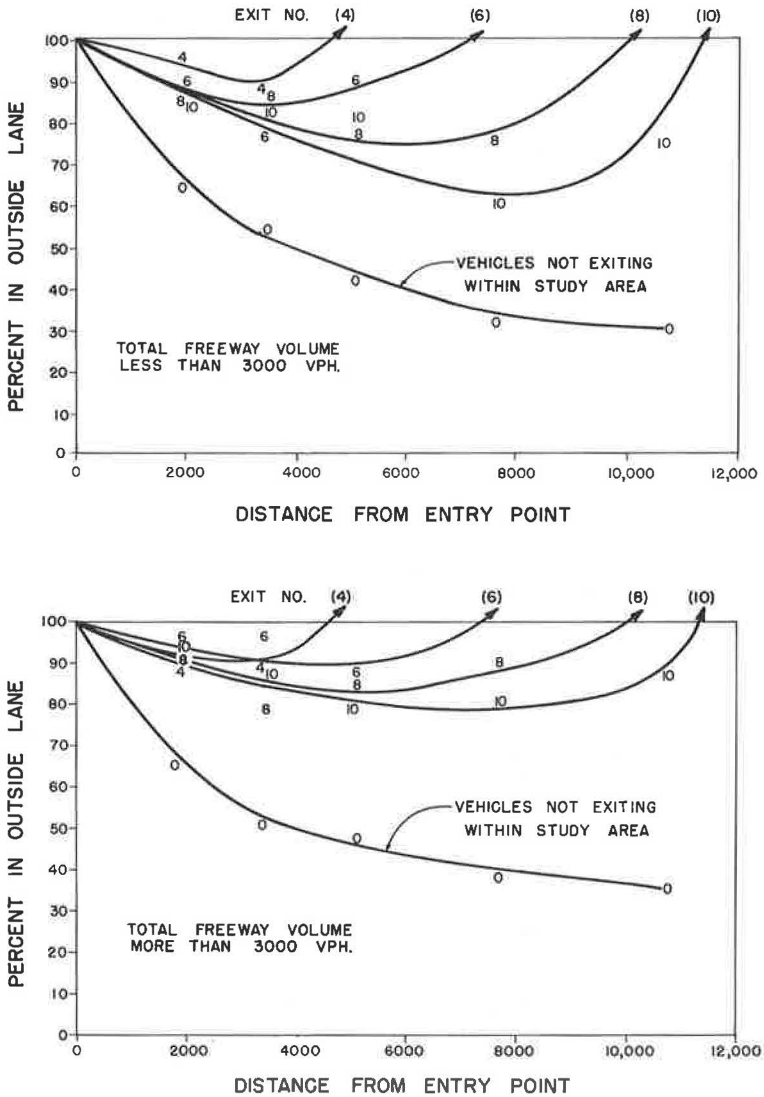

Figure 12. Outside lane use relationship with trip lengths.

The total volume of traffic on the freeway has an effect on the lane usage of entering traffic. As the total volume increases, entering vehicles are more restricted to the outside lane and motorists view a temporary lane change less worthwhile in view of the fact that another lane-change opportunity must be found to return to the outside lane prior to exiting. Figure 12 shows the relationship between trip length and the percent of the entering traffic in the outside lane. The upper portion of Figure 12 pertains to only light total freeway traffic volumes of less than 3, 000 vehicles per hour, one way. The lower portion of Figure 12 corresponds to conditions of moderate to heavy total freeway traffic volumes of more than 3, 000 vehicles per hour, one way. Although the observation points are widely spaced, it would seem that the restrictive effect of the higher volumes is noticeable.

Figure 12 indicates that "ordinary" or "equitable" lane use distribution is only attained by those vehicles traveling several miles on the freeway. Those vehicles traveling less than three miles cannot be expected to reach that "steady-state" lane distribution which is characteristic of "through" vehicles.

It must be pointed out that this is the result of a single study. It is expected that this study will provide a basis for further work of a similar nature.

CAPACITY CHARACTERISTICS

Theoretical Approach to the Capacity-Level of Service Concept

Both capacity and level of service are functions of the physical features of the highway facility and the interaction of vehicles in the traffic stream. The distinction is this: A given lane or roadway may provide a wide range of levels of service, but only one possible capacity. The various levels for any specific roadway are a function of the volume and composition of traffic. A given lane or roadway designed for a given level of service as a specified volume will operate at many different levels of service as the flow varies during an hour, and as the volume varies during different hours of the day, days of the week, periods of the year, and during different years with traffic growth. In other words, fluctuations in demand do not cause fluctuations in capacity, but do effect changes in the quality of operation afforded the motorist. In a very general way then highway planning, design and operational problems become a case of whether a certain roadway (capacity) can handle the projected or measured demand (volume) at an acceptable level of service (speed, etc.). Because of both observed and theoretical speed-volume relationships on freeway facilities, which are considered later, it is possible to anticipate to some degree just what level of service can be expected for a given demand-capacity ratio. The obvious weakness lies in the fact that most of the qualitative factors affecting level of service cannot be related directly to traffic volume.

Greater dependency on motor vehicle transportation has brought about a need for greater efficiency in traffic facilities. The motorist is no longer satisfied to be "out of the mud." In fact, fewer and fewer folks remember the days of unpaved roads. The freeway is an outgrowth of the demand for highways providing higher levels of service. The place that motor vehicle transportation plays in our society demands dependable service be provided by traffic facilities and the popularity or attraction to the freeway illustrates this point. It is very important that the engineer clearly understands the

The individual motorist seldom understands or appreciates efficiency of a facility in terms of volume accommodated. He evaluates efficiency in terms of his trip—the service to him. He evaluates the operating conditions of speed, travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, operating costs, etc. The level of service is a term which denotes the different operating conditions that occur on a given lane or roadway when accommodating various traffic volumes.

Recent contributions (7) to traffic flow theory regard traffic as a one-dimensional compressible fluid with a concentration, k, and a fluid velocity, u. The conservation of vehicles is explained by the following equation of continuity:

$$\frac{\partial k}{\partial t} + \frac{\partial (ku)}{\partial x} = 0 \tag{1}$$

If it is assumed that drivers adjust their speed in accordance with the traffic conditions about them as expressed by the general expression $k^n \partial k/\partial x$, the acceleration of the traffic stream becomes

$$\frac{du}{dt} = - c^2 k^n \frac{\partial k}{\partial x} \tag{2}$$

Solving Eqs. 1 and 2 for $u = f(k)$, and making use of $q = ku$ yields the following generalized equation of state for a traffic stream

$$q = ku_f \left[1 - \left(\frac{k}{k_j} \right)^{(n+1)/2} \right], \; n > -1 \tag{3}$$

where u_f is the free speed and k_j is the jam concentration. The exponent, n, provides some flexibility in fitting a theoretical flow concentration curve to a particular highway (8).

The speed of waves carrying continuous changes of flow through the stream of vehicles is given by the derivative of the q - k equation in Eq. 3

$$q' = u_f \left[1 - \frac{(n+3)}{2} \left(\frac{k}{k_j} \right)^{(n+1)/2} \right], \quad n > -1 \tag{4}$$

The concentration, k_m, at which flow is a maximum is obtained by setting Eq. 4 equal to zero and solving for k:

$$k_m = \left[(n+3)/2 \right]^{-2/(n+1)} k_j, \quad n > -1 \tag{5}$$

Repeating for $dq/du = 0$ gives the speed, u_m, at which flow is a maximum

$$u_m = \left[(n+1)/(n+3) \right] u_f, \quad n > -1 \tag{6}$$

It therefore follows that the maximum traffic flow obtainable on a roadbed (capacity) is

$$q_m = k_m u_m \tag{7}$$

It has been hypothesized (9) that discontinuities in traffic flow are propagated in a manner similar to "shock waves" in the theory of compressible fluids. The speed of a shock wave, U, is given by the slope of the chord joining the two points of the flow-concentration curve which represent the conditions ahead of and behind the shock wave

$$U = (q_2 - q_1)/(k_2 - k_1) \tag{8}$$

Application of the mean value theorem suggests that the speed of the shock wave is approximately the mean of the speeds of the waves running into it from either side

$$U = 1/2 \, (q'_1 + q'_2) \tag{9}$$

where q'_1 is given in Eq. 4.

The very strong analogy between traffic flow and fluid flow suggests that the conditions of continuity of momentum and energy should be fulfilled at the surface of a traffic shock wave, just as the equations of dynamic compatibility must be fulfilled in fluid dynamics. Multiplying Eq. 1 by u and Eq. 2 by k, then adding the two equations, we obtain

$$\frac{\partial(ku)}{\partial t} = - \frac{\partial \left(ku^2 + k^{n+2} \dfrac{c^2}{n+2} \right)}{\partial x} \tag{10}$$

Eq. 10 is the law of conservation of momentum in the differential form as applied to traffic flow. Comparing Eqs. 1 and 10 with the classical forms in hydrodynamics, we can complete the analogy between the fluid and traffic quantities. This correspondence is illustrated in Table 2.

Kinetic energy, ku^2, is the energy of motion of the traffic stream. The measure of the jerkiness of the driving in this stream is given by the standard deviation of the ac-

TABLE 2

CORRESPONDENCE BETWEEN PHYSICAL SYSTEMS

Factor	Hydrodynamic System	Traffic System
Variables	Mass density, p Velocity, v	Concentration, k Speed, u
Parameters	Momentum, pv Shock wave velocity, U Kinetic energy, pv 2/2 Internal energy, ϵ	Flow, ku Shock wave velocity, U Kinetic energy, ku 2/2 Acceleration noise, σ

celeration or acceleration noise, σ. The units of both parameters are those of acceleration. Energy, as expressed in these two quantities, is consistent with the level of service concept previously defined. Thus, the kinetic energy of the stream fulfills the first level of service factor (speed and travel time), whereas internal energy (acceleration noise) measures such level of service factors as traffic interruption and freedom to maneuver, and to some degree safety, comfort and operation costs.

Utilizing energy, rather than momentum, as the criteria for optimization depends on finding those values of k and u that maximize the kinetic energy of the traffic stream, E, and minimize the internal energy or lost energy, σ. Division of Eq. 3 by k, squaring the term and then multiplying by k yields

$$E = ku_f^2 \left[1 - 2\left(\frac{k}{k_j}\right)^{(n+1)/2} + \left(\frac{k}{k_j}\right)^{(n+1)} \right], \quad n > -1 \tag{11}$$

$$k'_m = (n+2)^{-2/(n+1)} k_j, \quad n > -1 \tag{12}$$

$$u'_m = \left[(n+1)/(n+2)\right] u_f, \quad n > -1 \tag{13}$$

and

$$q'_m = k'_m u'_m \tag{14}$$

Dividing Eqs. 5, 6 and 7 by Eqs. 12, 13, and 14, respectively, we get

$$\frac{k_m}{k'_m} = \left[\frac{2(n+2)}{(n+3)}\right]^{2/(n+1)}, \quad n > -1 \tag{15}$$

$$\frac{u_m}{u'_m} = \frac{n+2}{n+3}, \quad n > -1 \tag{16}$$

and

$$\frac{q_m}{q'_m} = 2^{2/(n+1)} \frac{(n+2)}{(n+3)}^{(n+3)/(n+1)}, \quad n > -1 \tag{17}$$

It is apparent that $k'_m < k_m$ and $u'_m > u_m$ which, from the point of view of the motorist, suggests that energy is a better criteria for defining optimum operation. Of course, this is accomplished by a sacrifice in traffic flow, since $q'_m < q_m$.

Traffic engineers have long been faced with the dilemma of relating possible capacity to level of service quantitatively. Much of the difficulty can be attributed to the fact that capacity is expressed in the units of volume, whereas the term level of service is highly subjective in nature. Volume is a logical measure of efficiency from the point of view of the engineer, whereas motion in the form of speed and the magnitude and frequency of speed changes is an important measure of level of service from the point of view of the individual driver.

The momentum-kinetic energy analogy seems to apply. If a moving mass strikes a stationary object without rebounding, as when the descending block of a pile driver strikes the pile, the resulting motion of the latter depends on the momentum of the block and not upon its kinetic energy. Therefore, if a fixed amount of energy is available, it is more effective to use a heavy mass moving at a relatively low speed than a lighter mass moving at a high speed. The momentum of the slow-moving heavy mass after falling a short distance is greater than that of a small rapidly moving mass which has been lifted higher by the expenditure of the same amount of energy.

Replacing mass in the previous discussion with traffic density, it is seen that efforts to measure efficiency in terms of momentum (traffic "throughput") must necessarily be achieved with a high traffic stream density and a low traffic stream speed. This is obviously not consistent with the level of service concept. However, because energy is a scalar quantity, the energy of a system such as a traffic stream is equal to the sum of the energies of its constituent particles and will be a maximum when internal friction caused by vehicular interaction is a minimum. Inasmuch as this internal friction reflects some compromise on the individual driver's freedom to maneuver, his comfort, and his safety, the energy concept includes most of the qualitative ingredients defined in level of service. Moreover, the energy concept affords the engineer the opportunity to treat level of service quantitatively.

If Eqs. 3 and 11 are expressed in terms of speed only, and then normalized, they become (for the special case of n = 1)

$$\frac{q}{q_m} = 4\left[\left(\frac{u}{u_f}\right) - \left(\frac{u}{u_f}\right)^2\right] \tag{18}$$

and

$$\frac{qu}{q_m u_f} = 4\left[\left(\frac{u}{u_f}\right)^2 - \left(\frac{u}{u_f}\right)^3\right] \tag{19}$$

The curves of Eqs. 18 and 19 are plotted in Figure 13. The right side of the graph is the well known volume-speed relationship normalized so that the abscissa is the ratio of flow to capacity and the ordinate the ratio of speed to free speed. Because division of the abscissa by the ordinate

$$\frac{q}{q_m} \div \frac{u}{u_f} = \frac{ku}{4k_j u_f} \div \frac{u}{u_f} = \frac{k}{4k_j} \tag{20}$$

it is apparent that the slope of any ray is one-fourth the normalized traffic density. The optimum density rays, using both the momentum and energy criteria, are plotted.

The left side of the graph shows the relationship between the kinetic energy and speed of the traffic stream. Division of the abscissa by the ordinate

$$\frac{qu}{q_m u_f} \div \frac{u}{u_f} = \frac{q}{q_m} \tag{21}$$

gives the flow-capacity ratio. Thus, the maximum abscissa gives maximum kinetic energy (located at $u'_m = 2/3\ u_f$), while the maximum slope gives the maximum momentum or flow (located at $u_m = 1/2\ u_f$).

The relationship between capacity and level of service is so fundamental to such practical aspects of traffic engineering as planning, design and operations, it is important that the distinction between these terms be appreciated. This can best be accomplished by a quantitative relationship, based on the energy-momentum analogy, as expressed in the following definitions:

Possible capacity is the maximum number of vehicles that can be handled by a particular roadway component under prevailing conditions. It is that product of the density and speed that maximizes the momentum of the traffic stream.

Level of Service refers to the quality of driving conditions afforded a motorist by a particular facility as reflected by (a) speed and travel time, (b) traffic interruption, (c) freedom to maneuver, (d) safety, (e) driving comfort and convenience, and (f) vehicular operating costs. Seven levels of service are described. (See Fig. 13.)

Level of Service A describes a free flow accompanied by low volumes, low densities, and high speeds which are controlled by the driver desires and physical roadway conditions (free speed). Although the variance in speeds is high, there is no restriction in maneuverability due to the presence of other vehicles, and drivers can maintain their desired speeds with little or no delay. This is the service expected in rural locations.

Level of Service B, C, and D describes the zone of stable flow. The upper limit is set by the zone of free flow, whereas the lower limit is defined by the optimum density, k'_m, and optimum speed, u'_m, based on maximizing the kinetic energy of the traffic stream. The conditions at u'_m and k'_m are acceptable for urban design practice. The divisions associated between levels B-C and C-D are arbitrary.

Level of Service E_1 & E_2 describes the zones of unstable flow. Zone E_1 is set between the optimum conditions described by the energy and momentum criteria. In this zone, small increase in volume is accompanied by both a large decrease in speed and kinetic energy leading to high densities and internal friction which can

a traffic density greater than k_m, yet with a vehicular flow greater than q_m. This type of operation cannot persist and leads inevitably to congestion.

Level of Service F describes a forced flow condition at low speeds and very high internal friction. Volumes are below capacity and storage areas consisting of queues of vehicles form. Normal operation is not achieved until the storage queue is dissipated.

Measurements and Relationships

The primary characteristics of traffic movement are concerned with speed, density and volume. These three fundamental characteristics are dependent on the geometric design of the roadway and the operational requirements of the traffic stream. Interest in these characteristics is manifest in the need for establishing representative possible and practical capacities for freeway sections.

While any set of speed observations may be influenced by such items as demand, capacity, design, weather and controls, it is not generally appreciated that the location at which measurements are made has a great deal to do with the adequate description of operating conditions. For example, traffic data taken just beyond an entrance ramp may reflect smooth and uniform operation when actually the traffic behind the ramp may be operating under stop-and-go conditions. Because congestion at one point may cause congestion for a great distance back along the freeway, a survey made at a "point" behind this critical ramp area will reflect poor conditions (low speed and relatively low volume) without a direct association with the cause of the congestion being possible.

Motion picture study procedures provide the advantage of having a view of a reasonably long section of freeway and reasonably precise measurements of numerous traffic characteristics such as speed, volume and density. However, even with the view of a section of freeway 1,000 to 2,000 feet in length, it is often difficult to determine accu-

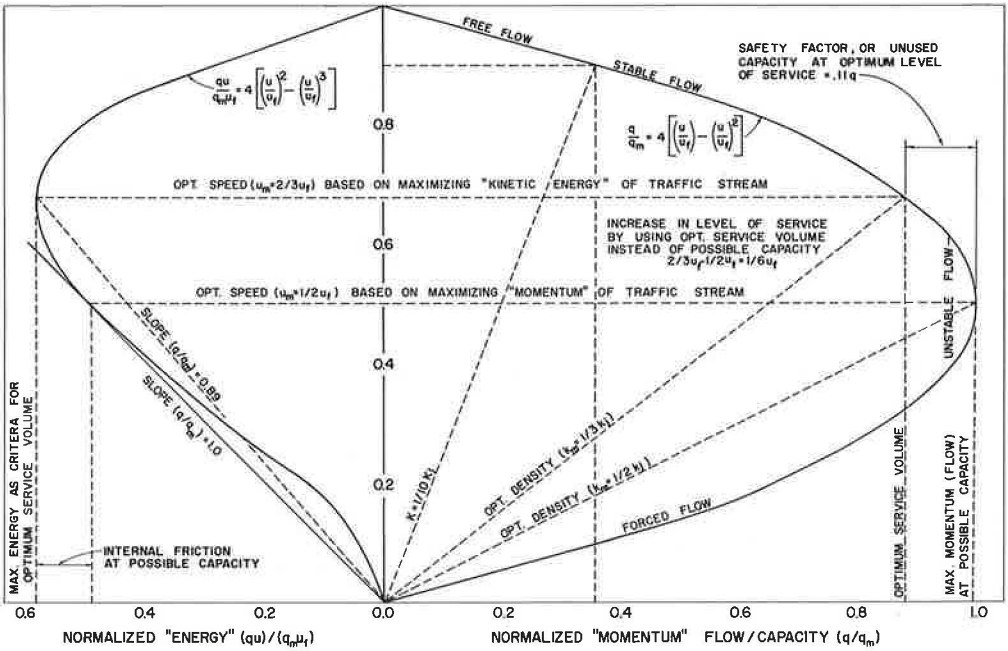

Figure 13. Quantitative approach to level of service-capacity relationship using energy-momentum analogy.

rately the cause of congestion. Congestion at one study area may actually be caused by conditions existing at a point farther along the freeway. Although the motion picture provides some possibilities of continually examining conditions throughout a section for possible influencing factors, the section studied from a single camera location is not always long enough to reveal whether congestion and "stoppages" are caused by conditions within the study area or by conditions ahead.

Two methods which have been successfully utilized and that do not rely on point survey data for describing flow characteristics are television camera surveillance and aerial photography. Pursuant to this research, two types of aerial photography have been studied (10): (1) strip photography where two continuous pictures are taken simultaneously over the entire study section, and (2) time-lapse photography where individual overlapping pictures are taken at short intervals of time. Time-lapse photography seems to be more suited for speed and density measurements, and can provide multiple speeds for each vehicle from which acceleration can be calculated. Of course, the shortcomings of the aerial approach, though distinct from the point survey approach, are nevertheless significant in that it is essentially an "instant" survey.

By careful design of a freeway study, either "point" or "instant" surveys can be used to give continuous coverage in both time and space. This two-dimensional interpretation is indispensable because all traffic characteristics vary in both time and space. "Contour maps" provide a means of illustrating these two-dimensional variations in the characteristics. These maps are drawn by using time as the ordinate, and distance along the freeway as the abscissa. If aerial photography is used (the "instant" survey approach), flight runs must be made at periodic intervals (about 10 minutes), and the speeds are averaged at about 600-ft intervals giving about 8.5 points per mile per 10 minutes. Interpolating in both time and space and connecting points of equal speed yield a speed contour map. Obviously, the same contour map could be obtained by interpolating between "point" surveys spaced at 600-ft intervals. Figure 14 illustrates speed contours for total inbound traffic on the Gulf Freeway, Houston, obtained from the aerial photographic survey method.

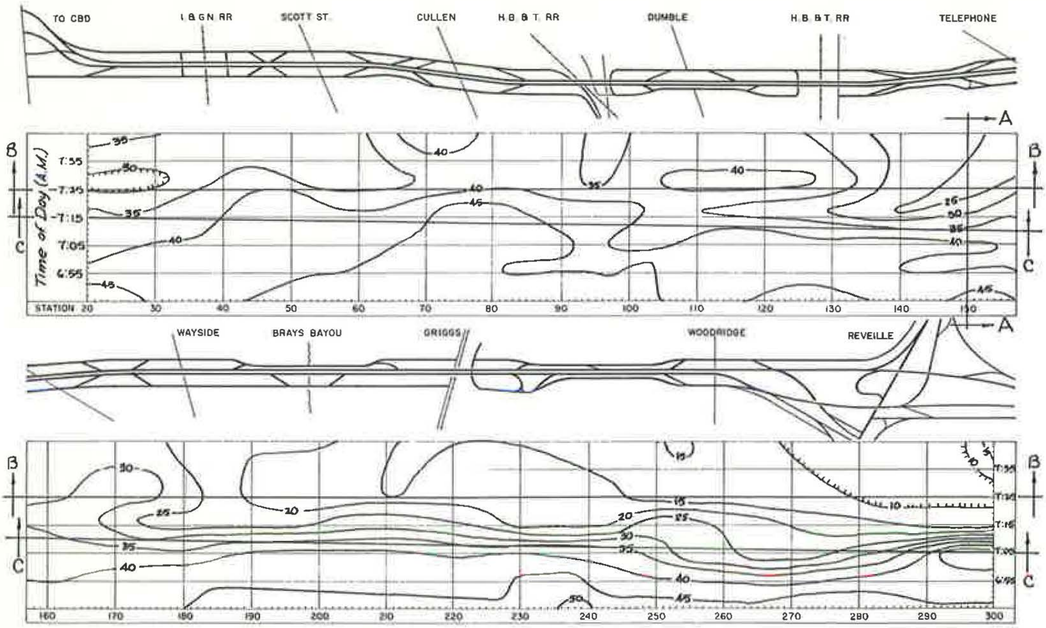

Figure 14. Speed contours (total inbound traffic).

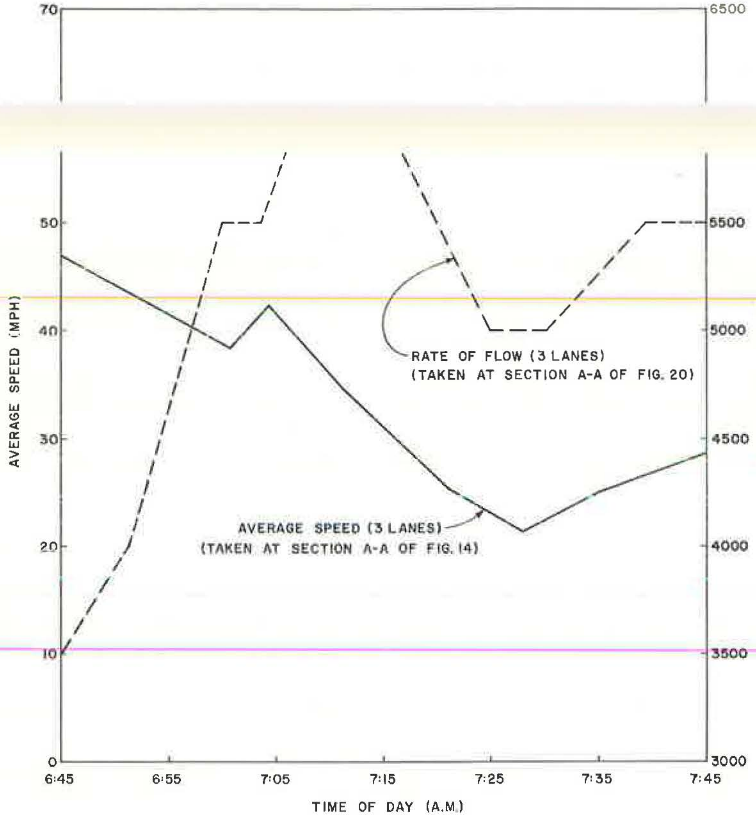

Figure 15. Speed-flow relationship as obtained from contour maps.

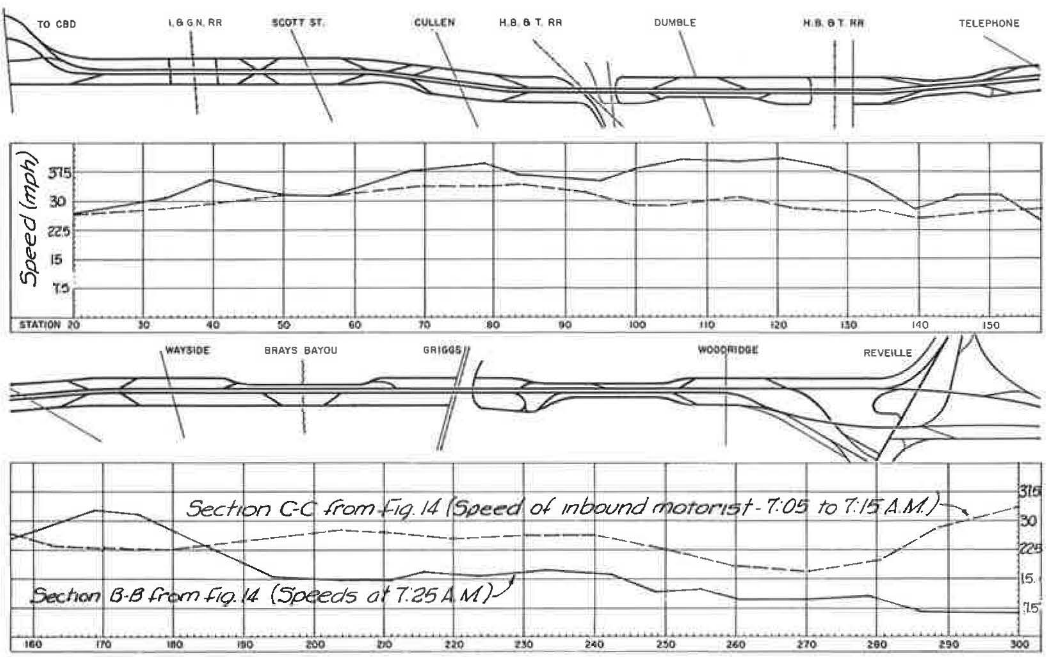

Figure 16. Speed profiles obtained from "contour map."

In addition to providing a continuous record of speed for an entire facility for a sustained period such as the peak hour, the contour map affords the opportunity to isolate critical locations and periods and then to study them in more detail. Consider, for example, the Telephone Interchange Entrance Ramp located at Station 150 + 00 (Fig. 14). The profile of section A-A is plotted in Figure 15. A speed-flow relation, obtained solely from contour maps, is illustrated. A profile plotted from section B-B' in Figure 14 is plotted in Figure 16. This profile illustrates the performance of the entire facility at 7:25 a.m., as reflected by speeds.

Although profiles at either a point or at an instance may have some conceptual appeal to the engineer in evaluating freeway operations, the level of service concept previously discussed is based on the driving conditions afforded an individual motorist as reflected, for example, by his speed. Section C-C in Figure 14 indicates the path of a hypothetical motorist traversing the freeway starting from the Reveille Interchange at 7:05 and arriving at the downtown distribution system at 7:15. This profile is also plotted in Figure 16.

It is generally accepted that the largest number of vehicles that can pass a given point in one lane of a multilane highway, under ideal conditions, is between 1,900 and 2,200 veh/hr. This represents an average maximum volume per lane sustained during the period of one hour. Figures 17, 18, 19 and 20 illustrate volume contours for the three inbound lanes on the Gulf Freeway, Houston, as well as the three-lane total for inbound traffic. It is evident that higher rates of flow exist for specific lanes or for short periods of time. Although the capacity of a freeway under ideal conditions is considered to be 2,000 veh/lane/hr, only in situations where the peak period demand extends very nearly for the entire hour will this capacity value be realized.

To illustrate the speed-density-volume relationship over shorter sections of freeway, the 14 graphs of Figure 21 were plotted for the inbound shoulder lane. Each graph represents the average conditions encountered throughout the study hour for the 2,000-ft length of freeway directly above the graph. The graphs' ordinate is speed in miles per hour and the upper abscissa is volume in vehicles per hour, while the lower abscissa is density in vehicles per mile. The numbers from 1 to 6 refer to the times of the 6

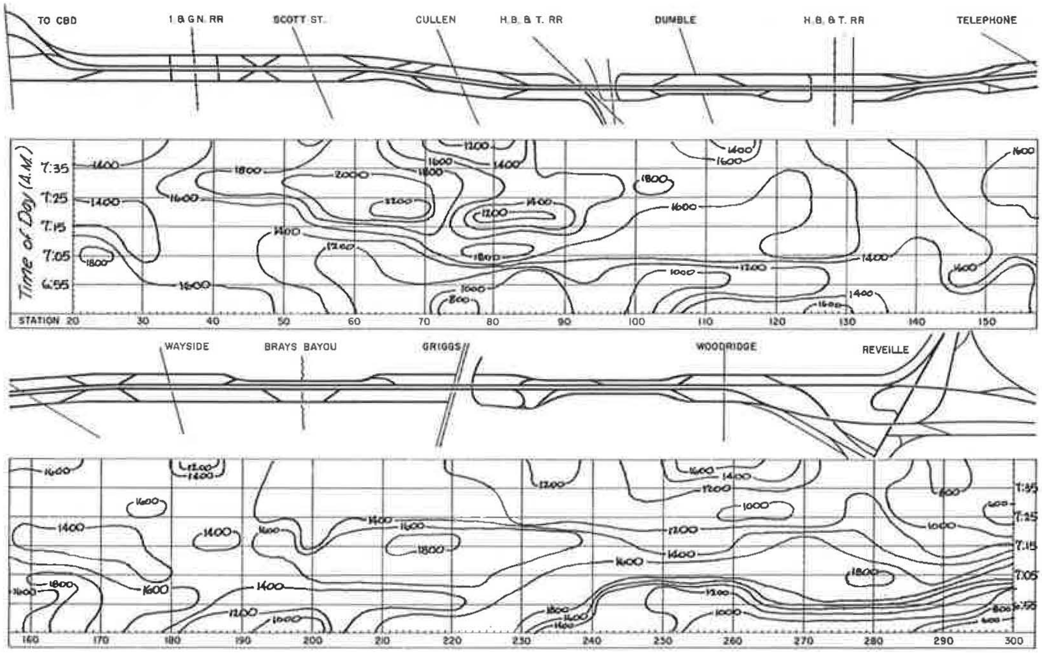

Figure 17. Volume contours (shoulder lane).

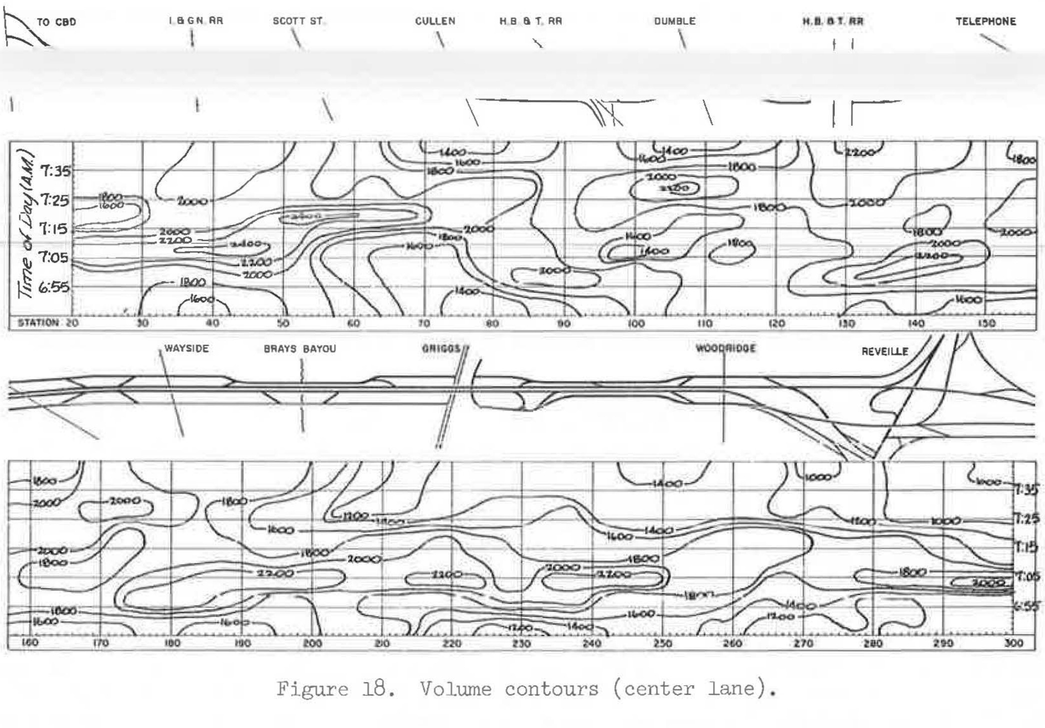

Figure 18. Volume contours (center lane).

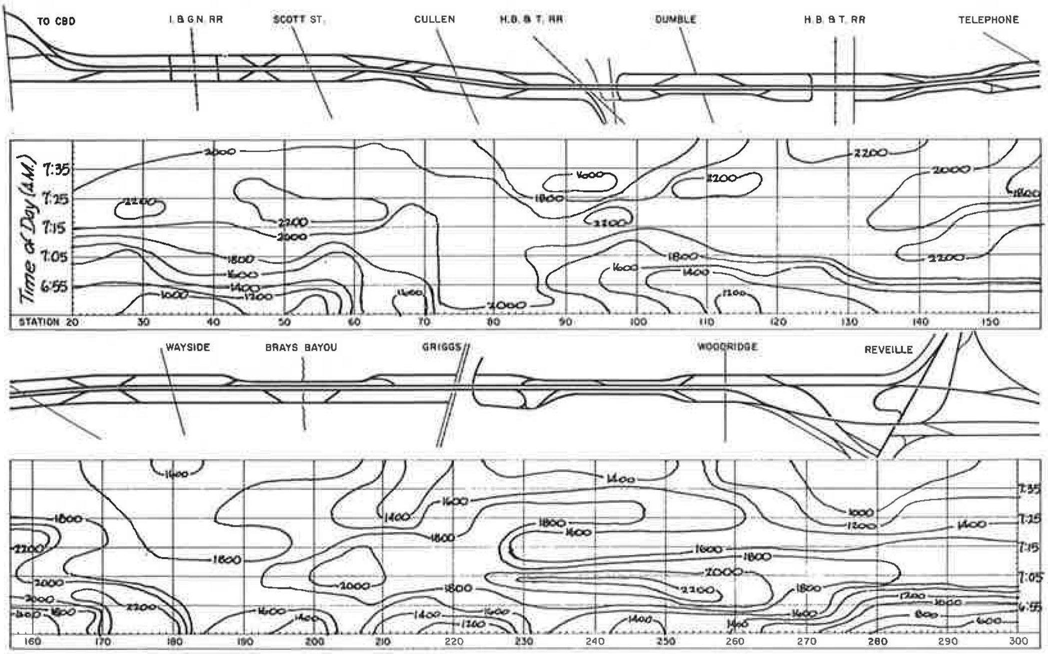

Figure 19. Volume contours (median lane).

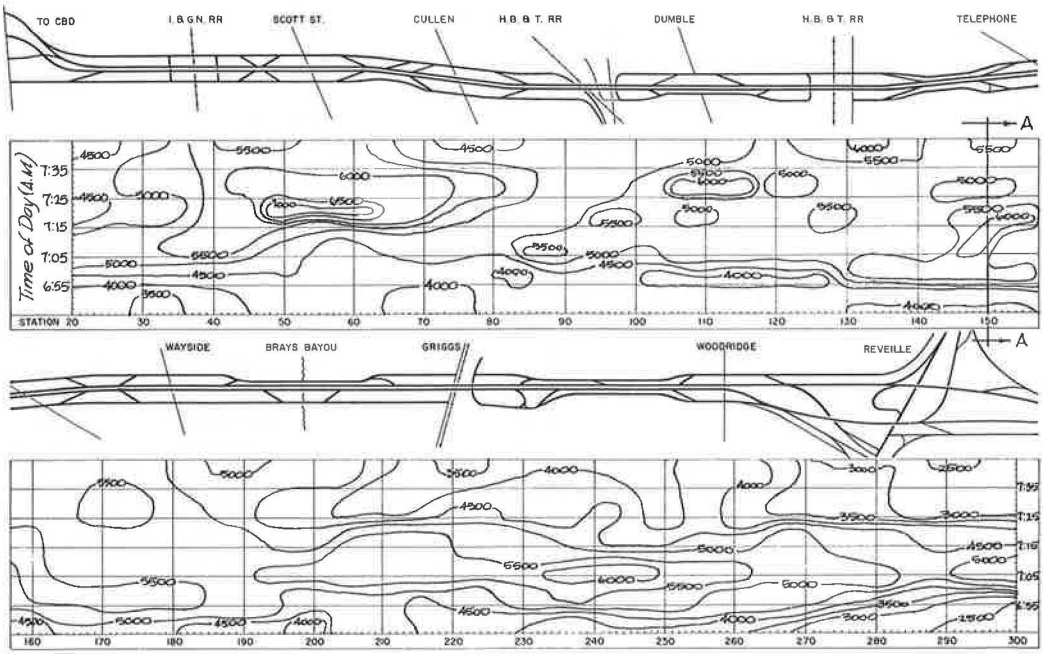

Figure 20. Volume contours (total inbound traffic).

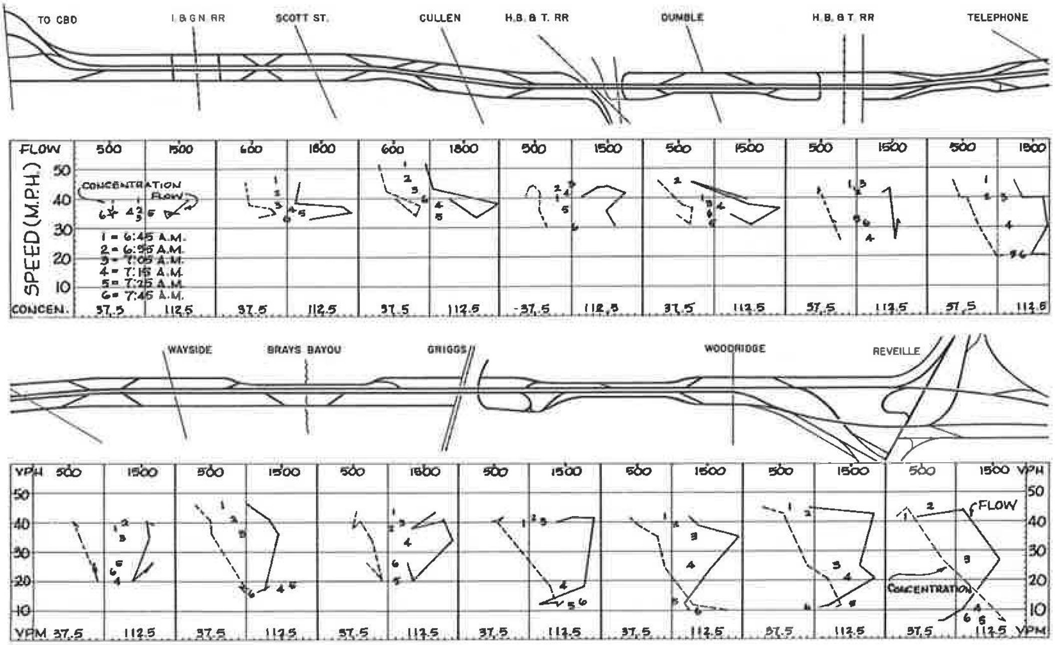

Figure 21. Observed speed-volume-density relationships (lane 1) in time and space.

flight runs, thus giving a chronological plot of data. The speed-density relationship is shown by a dotted line; the speed-volume relationship is shown by a solid line.

the interval within the Reveille Interchange, the range in average speeds varies from 45 to 5 mph over the study hour. At the other end of the freeway, however, average speeds remain at from 40 mph to 35 mph during the same period. The range of average speeds plainly decreases throughout the morning peak hour as one travels toward the CBD on the facility.

The volume-speed relationship is more difficult to explain. Decreases in speed do not always accompany increases in flow. However, several of the graphs exhibit a characteristic parabolic loop resulting from the decrease of speed at excessive flows. Keese, Pinnell and McCasland (2) explain that as peak flows build up, the average speed drops and generally does not recover to the original relationship with volume until the peak flow or demand has passed. Ryan and Breuning (11) utilize the concept of critical vs noncritical flow, with the dividing point being the maximum flow. They report that all three relations among speed, volume and density are linear within the noncritical flow region (before congestion sets in). May, et al. (12) define 3 zones which may be described as constant speed, constant volume and constant rate of change of volume with density. In zone 1, the speed of the vehicle is determined by the facility itself and the volume matches the demand. Zone 2 represents impending poor operations; average speed drops but the flow rates may be sustained at a high level. In zone 3 both speed and volume rates decrease, which in itself may serve as a definition of congestion.

The analysis of highway traffic is more than the making of measurements and collection of facts. Although this exploration of the true nature and characteristics of freeway traffic is the necessary beginning in providing new ways of improving performance, the freeway traffic phenomena are so complex that a collection of bare data tells us a little more than we already know. Earlier in this article, a hydrodynamic model was explained. An important aspect of this model is the momentum-energy analogy which is summarized in Figure 13. These concepts yield several parameters which provide the means of organizing and interpreting the traffic characteristics collected.

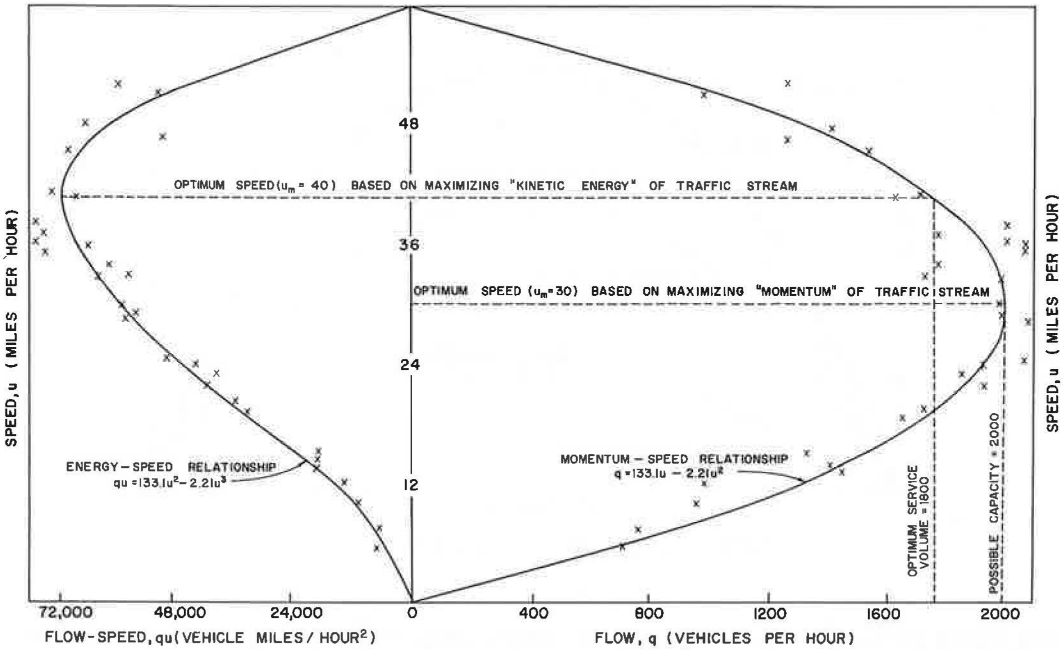

Figure 22. Verification of energy-momentum analogy.

Using Figure 22 as a model, volume vs speed and (volume × speed) vs volume data were plotted for lane 3 of the Gulf Freeway, inbound traffic, during the peak hour. The constants for a curve of the form

$$q = k_j u \left[1 - \left(\frac{u}{u_f}\right)\right] \tag{22}$$

was calculated by fitting a regression line to speed-density data taken from aerial photographs. Values of free speed, u_f, and jam density, k_j, are 60.3 mph and 133.1 vpm. Thus, the equation of the curve in the q - u plane becomes

$$q = 133.1\,u - 2.21\,u^2 \tag{23}$$

The equation for the "energy"-speed relationship is

$$qu = 133.1\,u^2 - 2.21\,u^3 \tag{24}$$

It can be observed that the curves provide excellent estimates of the points plotted, and verify the theoretical approach to the capacity-level of service concept suggested by the momentum-energy analogy of the hydrodynamic model of traffic flow.

The hydrodynamic model, so useful in describing the level of service-capacity concept, can be extended to aid in the explanation of a bottleneck (9). A bottleneck is a stretch of roadway with a flow capacity less than the road ahead (Fig. 23). The upper q - k curve in Figure 23 is for the roadway ahead of the bottleneck, and the lower one refers to the bottleneck itself. When the traffic volume reaches the capacity of the bottleneck, the velocity in the bottleneck, u_1, is less than ahead of the bottleneck, u_3. However, this difference in speed is not significant in urban area capacity problems

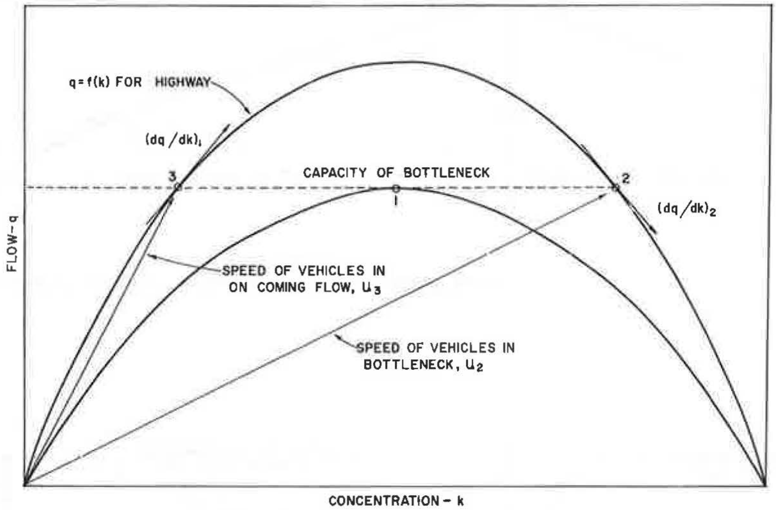

Figure 23. Traffic flow in a bottleneck.

and should not be used as a criterion for determining acceptable operation. But, any further increase in demand (volume) accumulates as a queue in advance of the bottleneck, and the traffic conditions in this region shift from those expressed by point 3 in Figure 23 to those expressed by point 2 (density changes from k_3 to k_2).

When a bottleneck is operating at capacity, the speed of traffic is independent of the geometric conditions in the upstream section. Because congestion may last much longer (Fig. 24) than that interval in which demand exceeds capacity, it is important that precautions be taken to prevent this. A stipulated rate-of-flow for a 5-min period

designer. This is illustrated by the design curves relating level of service to a 5-min rate of flow (Fig. 25). Thus, a service volume of 1,800 has a probability of 0.50 of guaranteeing "stable flow" during the peak 5-min period. On the other hand, there is a 50 percent chance of "unstable flow" occurring; and a 2.5 percent chance of "forced flow." Of course, forced flow is congestion, and unstable flow can lead to congestion, due to the statistical variability of the vehicle headways. It is interesting to note that a relatively small reduction of 100 in the service volume, to a flow of 1,700 vph, greatly increases the probability of maintaining stable flow. These curves represent an attempt to put such a decision in the hands of highway administrators and designers. After this choice is made, the service volume to be used for design for the peak hour would be obtained from Figure 1, depending on the population of the city. (See Table 3.)

Figure 25 was obtained by determining the probability of getting observed rates of flow greater than the predicted values utilizing the data shown in Figures 2 and 3 and assuming the errors are normally distributed. The first four columns in Table 3 are taken directly from Figure 25; the last four columns utilize the peaking relationships expressed in Figure 1.

APPLICATIONS

Freeway Design

Highway design is an engineering function—not a handbook problem. The engineer is faced with the problem of predicting traffic demands in future years and providing facilities that will accommodate that traffic under a selected set of operating conditions or levels of service. Too often highway design has been accomplished by adopting a set of handbook "standards" which when coupled with traffic "guestimates" have resulted in the construction of many seriously inadequate facilities.

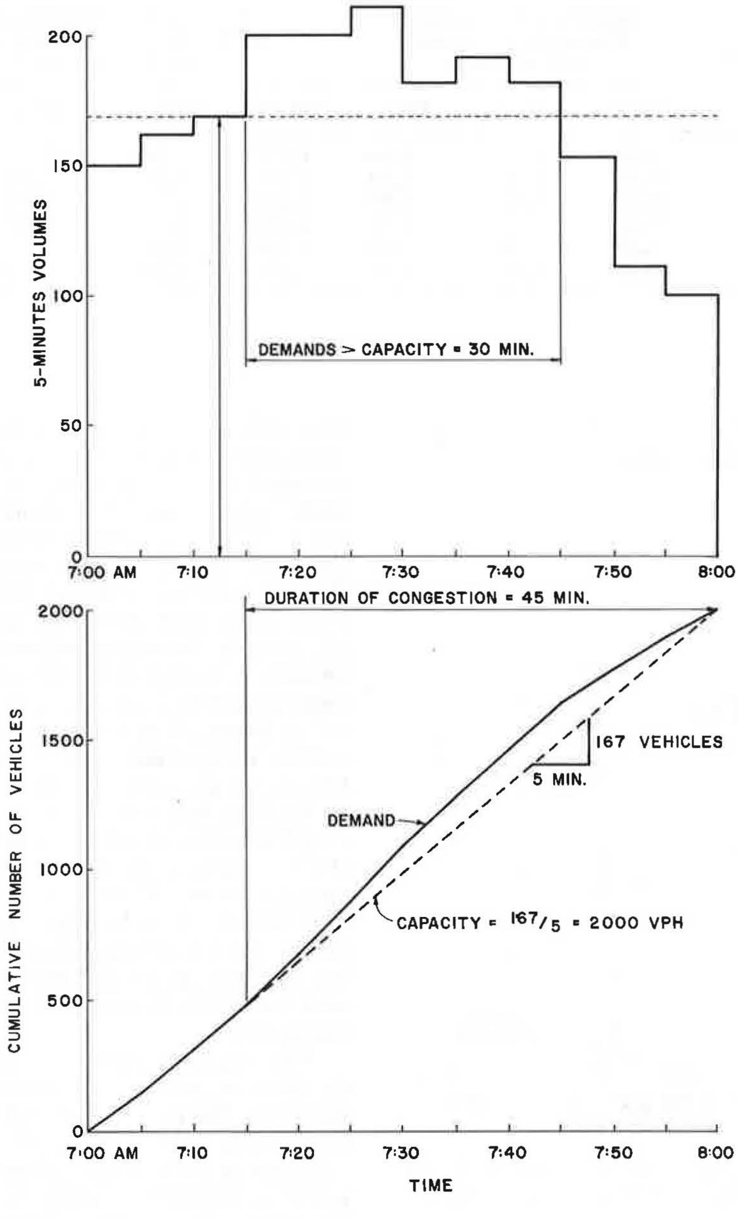

Figure 24. Relation between demand, capacity and congestion.

Traffic prediction, traffic operation and design have now developed to the point where it is possible for engineering (the application of science) to produce rather reliable results.

A freeway is not built for some date 20 years in the future. It must go to work the first day and serve efficiently all through its expected life. And, if history is not changed, many will be serving for quite a number of years beyond the "design" year.

The freeway is only one facility in a network of system of streets and highways. It has its place, but the system as a whole must be made to function efficiently. The day

TABLE 3

FREEWAY CAPACITY WITH CONFIDENCE LIMITS

Peak 5-Min Flow (vph)	Approx. Probabilities of Various Types of Flow in Peak 5-Min			Freeway Design Service Volume by Population of Metropolitan Area (total hourly vol/lane)			
	Stable	Unstable	Forced	100,000	500,000	1,000,000	5,000,000
1,500	1.00	0.00	0.00	1,100	1,200	1,300	1,300
1,600	0.98	0.02	0.00	1,200	1,300	1,300	1,400
1,700	0.85	0.15	0.00	1,300	1,400	1,400	1,500
1,800	0.50	0.48	0.02	1,400	1,500	1,500	1,600
1,900	0.15	0.69	0.16	1,500	1,600	1,600	1,700
2,000	0.03	0.47	0.50	1,500	1,600	1,700	1,800

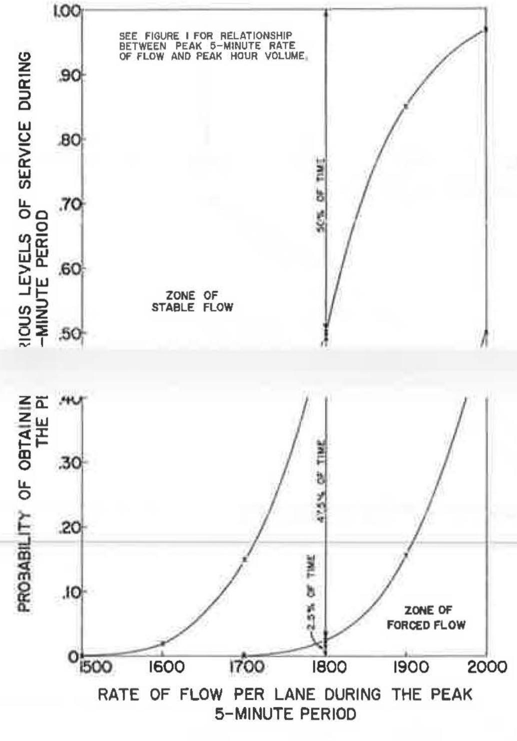

Figure 25. Design curves relating level of service to flows during the peak 5-min.

has gone when a freeway can be designed within the confines of two parallel right-of-way lines. Likewise, the day has gone when only the 20-yr "complete system" can be considered when designing a particular facility. Traffic projections and designs must be made on partial or incomplete systems if desirable service is to be obtained in the years before the whole system is completed. With the modern tools available, the designer should have at his disposal an accurate estimate of traffic demand for each stage of comple-

Engineering and management must be coupled in the selection of a level of service for design that is best adapted to the specific need. Economics and other factors will continue to play a major part in facility programming and even in design, but realistic projected service analysis will lead to more realistic priority programming.

The largest number of vehicles that can pass a given point in one lane of a multilane highway, under ideal conditions, is between 1,900 and 2,200 vph. This represents an average maximum volume per lane sustained during the period of one hour. Studies have found higher volumes for specific lanes or for short time periods. Where at least two lanes are provided for movement in one direction, and disregarding distribution by lanes, the capacity of a freeway under ideal conditions is considered to be 2,000 veh/lane/hr, as explained earlier in this report.

Where conditions are less than ideal because of reduced widths, sight distance, grades and commercial vehicles, etc., the capacity will be somewhat lower. Moskowitz and Newman (5) suggest some correction value to be used when these conditions are anticipated.

When the traffic volume equals the capacity of a freeway, operating conditions are poor. Speeds are low, with frequent stops and high delay. In order for the highway to

TABLE 4

LEVELS OF SERVICE AS ESTABLISHED BY ENERGY-MOMENTUM CONCEPT

Level of Service Zone	Description	Zone Limits[a]	
		Upper	Lower
A—free flow	Speeds are controlled by driver desires and physical roadway conditions; type of service expected in rural locations.	u_f	$0.9\ u_f$, $0.35\ q_m$
B, C & D—stable flow	Flow concentration, speed $^{-1}$. Conditions at u'_m and k'_m are acceptable for urban design practice; divisions between B-C and C-D are arbitrary.	$0.9\ u_f$, $0.35\ q_m$	u'_m, q'_m
E_1—unstable flow	A small increase in demand (flow) is accompanied by a large decrease in speed leading to high densities and internal friction which contribute to instability.	u'_m, q'_m	u_m, q_m
E_2—unstable flow	This type of high density operation cannot persist and leads inevitably to congestion.	u_m, q_m	$0.5\ u'_m$, q'_m
F—forced flow	Flows are below capacity and storage areas consisting of queues of vehicles form. Normal operation is not achieved until the storage queue is dissipated.	$0.5\ u'_m$, q'_m	0

[a]See Figure 13.

provide an acceptable level of service to the road user, it is necessary that the service volume be lower than the capacity of the roadway.

The level of service approach establishing levels of operation from free flow to capacity which is being considered by the HRB Committee on Highway Capacity is designed to allow the engineers and administrators to provide the highest level of service economically feasible. The momentum-energy analogy derived in the previous section is an effort to explain the capacity-level of service relationship rationally and quantitatively (Table 4). It must be recognized that highway traffic represents a stochastic phenomenon. Therefore, any highway facility, designed to accommodate traffic, must be designed with the realization that from time to time demand will exceed capacity. The organization of Table 3 is useful in that it provides the designer with confidence limits in determining the number of main lanes needed on a freeway.

After the determination of the number of freeway lanes, 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. An entrance ramp is an example of the first type of critical location, whereas exit ramps and grades can cause intervehicular conflicts.

It is interesting to note that a distinction can be made between the terms "critical location" and "bottleneck." A bottleneck is a section of roadway with a capacity lower than the adjacent upstream section. Thus, the volume input can exceed the capacity of the bottleneck, and the roadway upstream becomes a storage area whose level of service and rate of flow are governed by the capacity and operating conditions through the bottleneck. If traffic backed up at a bottleneck is required to stop, the capacity of the bottleneck becomes a function of vehicular departure headways from a stopped condition. Strictly speaking then, not all "critical locations" are "bottlenecks"; a bottleneck is only one form of critical location. The most important consideration is that operation in critical sections never drops below the adopted level of service. Often, this can be effected by not allowing the demand on the facility to exceed the bottleneck capacities.

As traffic operations deteriorate, vehicles tend to form platoons. In general, "platooning" is a function of the number of slow vehicles and the speed of slow vehicles. When an upgrade is introduced, speeds are reduced and platoon lengths increase. Accident potential and capacity mitigating lane change maneuvers are a direct result. The percentage of trucks in the stream, as well as the steepness and length of grade, will determine just how adverse this effect may be.

One possible solution to maintaining a level of service on the grade equal to a level grade, would be to add a climbing lane whenever the passenger car volume and speed varies below the adopted level of service. This, of course, is not always feasible on an urban freeway facility where lane reductions present severe operational problems. The policy of restricting trucks to the outside lane (predicated on the theory that if all trucks are traveling on the outside lane, then vehicles on the remaining lanes can maintain the operation levels achieved on level grade) ignores the fact that vehicles in the outside lane will not accept the same level of service as trucks and will attempt to change lanes. It is impossible for adjacent lanes to operate at drastically different levels of service. This influence of the operation of one lane on the operation of another is a well-established speed characteristic and is sometimes called "speed sympathy." Thus, the most acceptable solution to the problem of operating on freeway upgrades is to see that demand volumes never exceed the adopted level of service.

The traffic demand on a freeway can only change at entrance or exit ramps. Two of the most critical points on a freeway will be upstream from an exit ramp and down-

described at an upgrade, but can be much more severe where there is a back-up from the exit ramp onto the main roadway proper. Many exit ramp problems could be avoided by providing for the speed reduction on the ramp rather than on the shoulder lane of the freeway. Even where long parallel deceleration lanes are provided, they are not used because of the unnatural maneuver involved. Unfortunately, the close spacing of interchanges and use of frontage roads favor the use of short slip-type ramps. Where a high exit volume slip ramp is used, definite consideration should be given to placing yield signs on the frontage roads, thus preventing back-up from the exit ramp onto the freeway.

Entrance ramps may 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.

There are three basic procedures employed in determining the capacity of entrance 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 takes into consideration the distribution of freeway volumes per lane (discussed in the first section of this paper and also treated extensively by Hess (13)) and then limits the ramp volume to the merging capacity (assumed here to be equal to the service volume selected in Table 3) 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 (8). It is believed that the second method (Figs. 5 to 9) is the most practical in designing a

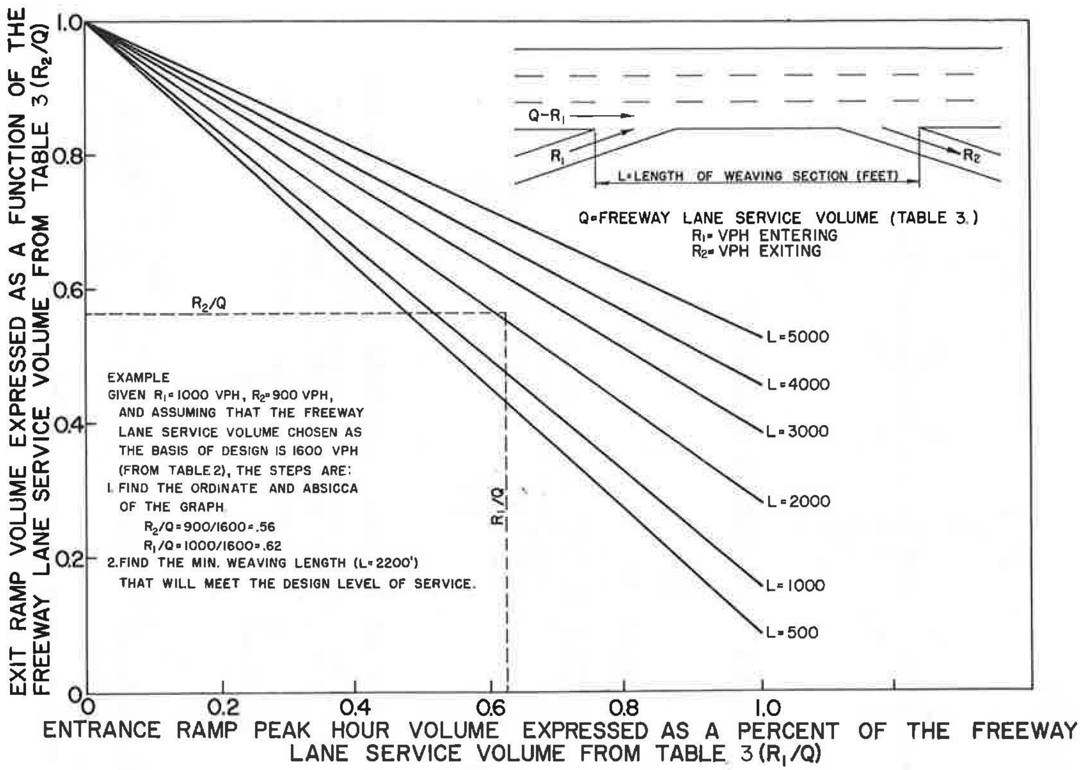

Figure 26. Determination of minimum length of weaving section to meet design level of service.

new facility. The first method is predicated on knowing the capacity of bottlenecks—something that is not known in the case of a new facility. Research concerning the third method is now underway; the advantage of this approach is that it recognizes that ramp capacity and operation must be affected by the geometrics of the ramps.

The last "critical location" to be considered is the weaving section. Weaving sections often simplify the layout of interchanges and result in right-of-way and constrution economy. The capacity of a weaving section is dependent on its length, number of lanes, running speed and relative volumes of individual movements. When large volume weaving movements occur during peak hours, approaching the possible capacity of the section, probably results are traffic stream friction, reduced speeds of operation, and a lower level of service. This can sometimes be avoided by the use of additional structures to separate ramps, reversing the order of ramps so as to place the critical weaving volumes on frontage roads, and the use of collector-distributor roads in conjunction with cloverleaf interchanges.

Weaving sections should be designed, checked and adjusted so that their capacity is greater than the service volume used as the basis for design. This is consistent with the level of service concept used in determining the number of main lanes and checking the merging capacities at entrance ramps. The determination of minimum length of weaving section to meet the controlling level of service is illustrated in Figure 26. These relationships were obtained by considering the outside lane use relation with trip length (Figs. 11 and 12). Referring to Figure 26, the maximum number of vehicles at an exit, R_2, cannot exceed Q - R_1, plus the number of entrance ramp vehicles that change lanes within the merging section.

Figure 27 illustrates four steps to be followed in the design of a freeway system, as follows:

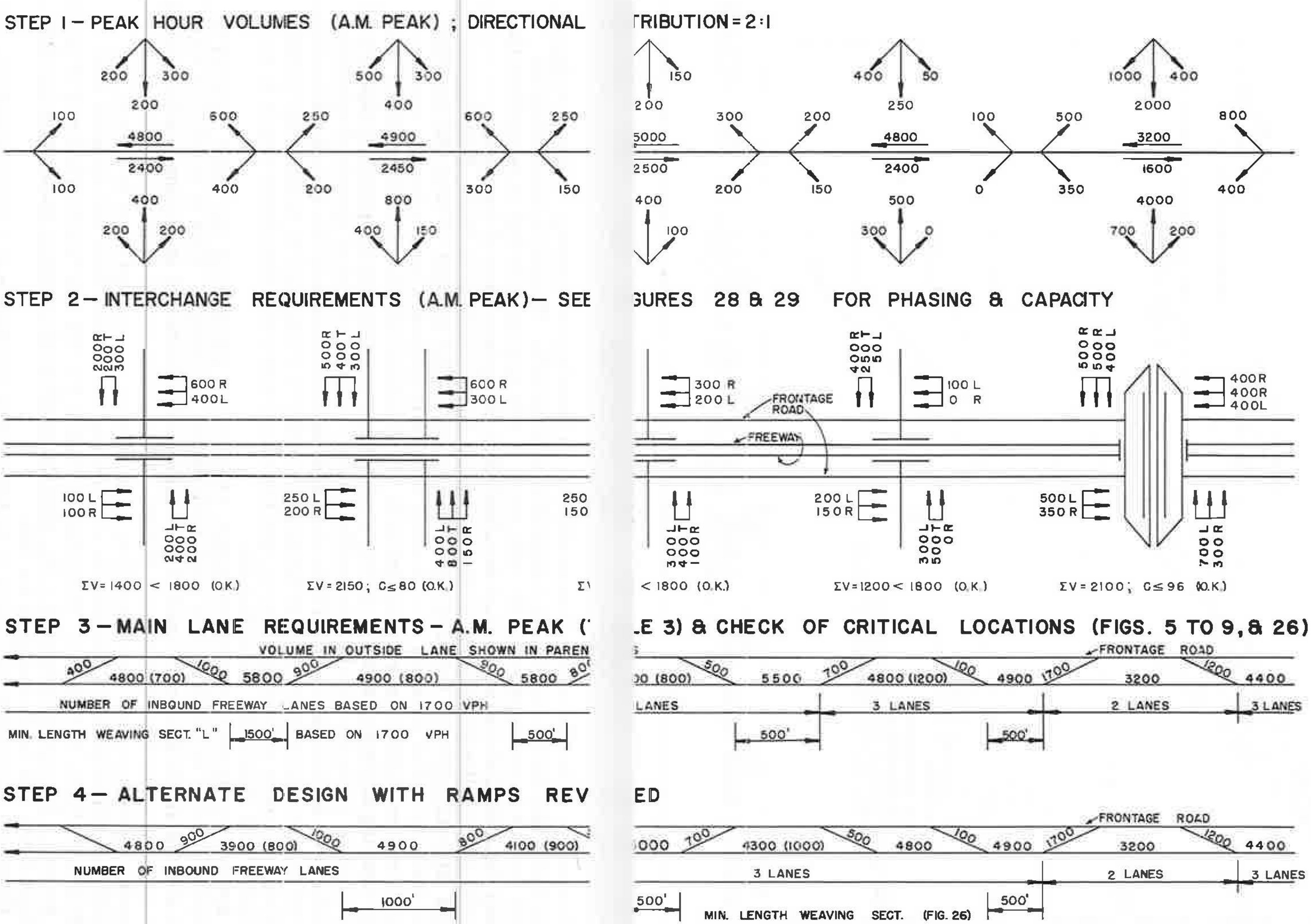

Figure 27. way design procedure.

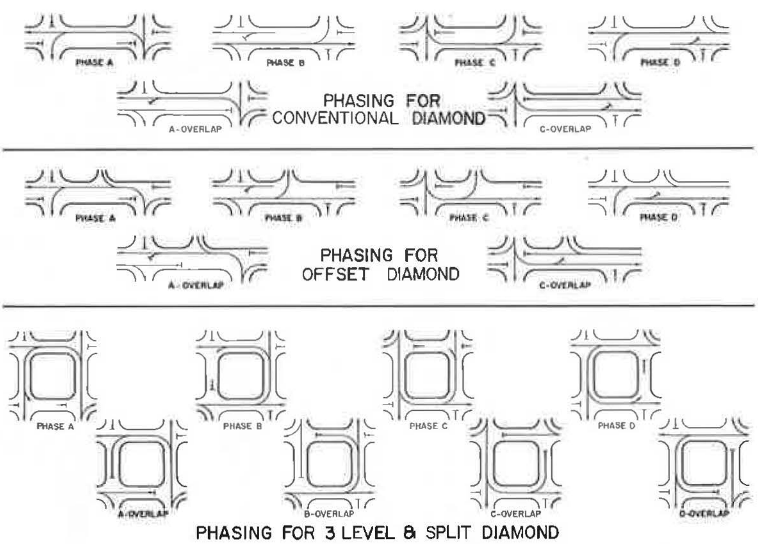

Figure 28.

Step 1—Determine the peak hour volumes through the application of the peak hour and directional distribution factors to the assigned daily traffic volumes. In an actual problem the PM peak would also be checked.

Step 2—Determine interchange requirements. It is important that this be done before freeway main lane requirements be investigated, because the number of ramps depends on the choice of interchange. Thus, a cloverleaf interchange and a directional interchange may have one or two entrance ramps and one or two exit ramps in each direction; whereas diamond interchanges have one entrance ramp and one exit ramp in each direction. If the interchange is to be signalized, a capacity check is made to see if the planned facilities will handle the traffic with reasonable cycle lengths (Figs. 28 and 29) (14). Should a facility be apparently underdesigned, additional approach lanes may be added or a higher type facility be substituted in its place.

Step 3—The number of main lanes depends on what service volume value is chosen as the design capacity. The freeway design service volumes in Table 3 enable the designer to judge what level of service can be expected for a given service volume based on the probability of obtaining various types of flow conditions during the peak 5-min period. For the purposes of this example a service volume of 1,700 vph is chosen. The operating conditions at critical locations must be checked to insure that the designated level of service is met at every point on the freeway. The critical sections considered in this paper are merging and weaving sections. Figures 5 to 9 provide the basis for determining if the merging capacities at entrance ramps are exceeded, where the merging capacity is defined as the service volume chosen in Table 3. Thus, since a total hourly volume of 1,700 vph is used as the basis for determining the number of lanes, then 1,700 vph would represent the merging capacity in this procedure. Figure 26 provides the basis for determining if weaving sections on the freeway meet the designated level of service.

Step 4—Alternate designs should always be considered. In Figure 27, one alternative is illustrated by merely reversing the order of entrance and exit ramps, resulting in 3 lanes in each direction instead of 4 lanes.

The level of service should be "in harmony" along the stretch of freeway being considered. Because operational problems at one point are reflected along the freeway for a distance depending on the volume-capacity relationship, it is not practical to consider a lower level of service at one or more critical points, rather the level of service selected for design should be met or exceeded at the critical or bottleneck points. This concept is referred to as balanced design and it is a must for freeways.

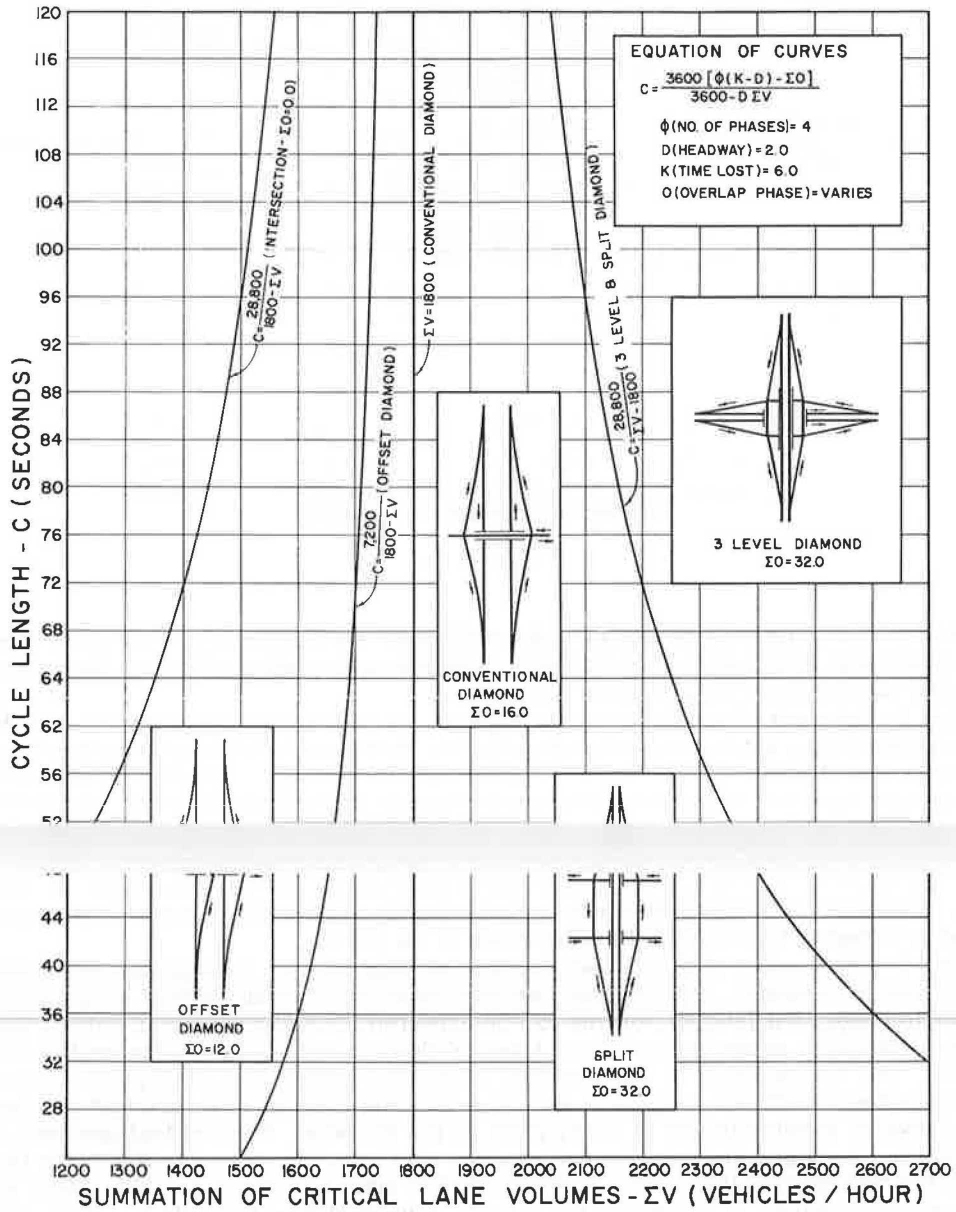

Figure 29. Design curves for 4-phase facilities.

Freeway Operations

The freeway motorist expects to have his needs anticipated and fulfilled to a much higher degree than on conventional roads. This expectation can sometimes be fulfilled by the application of capacity considerations to rational geometric design. More often than not, however, actual traffic and travel patterns differ from the projected values making constant freeway operational attention after construction a must.

Congestion occurs on a freeway section when the demand exceeds the capacity of that section for some period of time. Bottlenecks can be caused by changes in the freeway alignment (horizontal or vertical) or reductions in the freeway section (reduction of number of lanes, reduction in lane widths, the presence of an entrance ramp, etc.). Accidents, disabled vehicles and maintenance or law enforcement operations can also cause temporary bottlenecks by reducing the effective capacity or level of service provided.

Freeway design does not always eliminate the need for sound traffic regulation. A reasonably homogeneous traffic stream, particularly with respect to speed, is essential for efficient freeway operations. Pedestrians, bicycles, animals and animal-drawn vehicles are excluded from freeways. Motor scooters, non-highway (farm and construction) vehicles and processions, such as funerals, are also generally prohibited from the freeway. Towed vehicles, wide loads or other vehicle combinations such as trailers drawn by passenger vehicles which impede the normal movement of traffic may be barred during the peak traffic hours or during inclement weather.

Minimum speed limits are being increasingly used and have been found of great benefit, particularly on high-volume sections. The effect of this type of control is to reduce the number of major accident potential lane change maneuvers. The effects of slow-moving vehicles on both capacity and accident experience are so pronounced that a greater use of minimum limits appears probable. There is a need to eliminate all vehicles incapable of compatible freeway operation.

Increasing attention has been given to the possibility of and need for using variable speed control on urban freeway sections as a means of easing the accordion effects in a traffic stream as congestion develops. Drastic speed variations might be dampened by automatically adjusted speed message signs in advance of bottlenecks.

A properly designed entrance ramp with provision for adequate acceleration should allow the entering driver adequate distance to select a gap and enter the outside lane of the freeway at the speed of traffic in the lane. These merging areas operate best when there is a mutual adjustment between vehicles from both approaches. "Yield" signs impose rather drastic speed restrictions under the laws of a number of states thus causing operational problems, and are no longer mandatory on the Interstate system. It is generally felt that any speed restriction or arbitrary assignment of right-of-way should be avoided unless inadequacies in the design make it imperative.

It is generally agreed that one key to significant progress in operation of urban freeways lies in improved surveillance technique. In its most basic form, urban freeway surveillance is limited to moving police patrols. Recently, helicopters have been used for freeway surveillance in Los Angeles and other communities. Efficient operation of high density freeways is, however, more than knowing the locations of stranded vehicles; it may require closing or metering entrance ramps, or excluding certain classes of vehicles during short peak periods. Therefore, what is needed is a reliable, all-weather source of surveillance information with no excessive time lag.

Experimentation with closed circuit television as a surveillance tool was initiated on the John C. Lodge Freeway in Detroit. This offers the possibility of seeing a long area of highway in a short or instantaneous period of time, made possible by spacing cameras along the freeway so that a complete picture can be obtained of the entire section of roadway. Evaluation of the freeway operation depends mostly on the visual interpretation of the observers. However, many traffic people believe that this is not enough. The Chicago Surveillance Research Project, for example, is predicated on the assumption that trained observers offered no uniform objectivity. In other words, if an expressway is operating well, this quality can be detected by observing operating characteristics. When the characteristics drop below a predetermined level, action may be taken.

A traffic surveillance system should involve the continuous sampling of basic traffic characteristics for interpretation by established control parameters, in order to provide a quantitative knowledge of operating conditions necessary for immediate rational control and future design. The control logic of a surveillance system, or any system, is that combination of techniques and devices employed to regulate the operation of that system. The analysis shows what information is needed and where it will be obtained. Then, and only then, can the conception and design of the processing and analyzing equipment necessary to convert data into operational decisions and design warrants be described.

In research conducted during the past year by the Texas Transportation Institute on the Gulf Freeway Surveillance Project, the application of many control parameters to

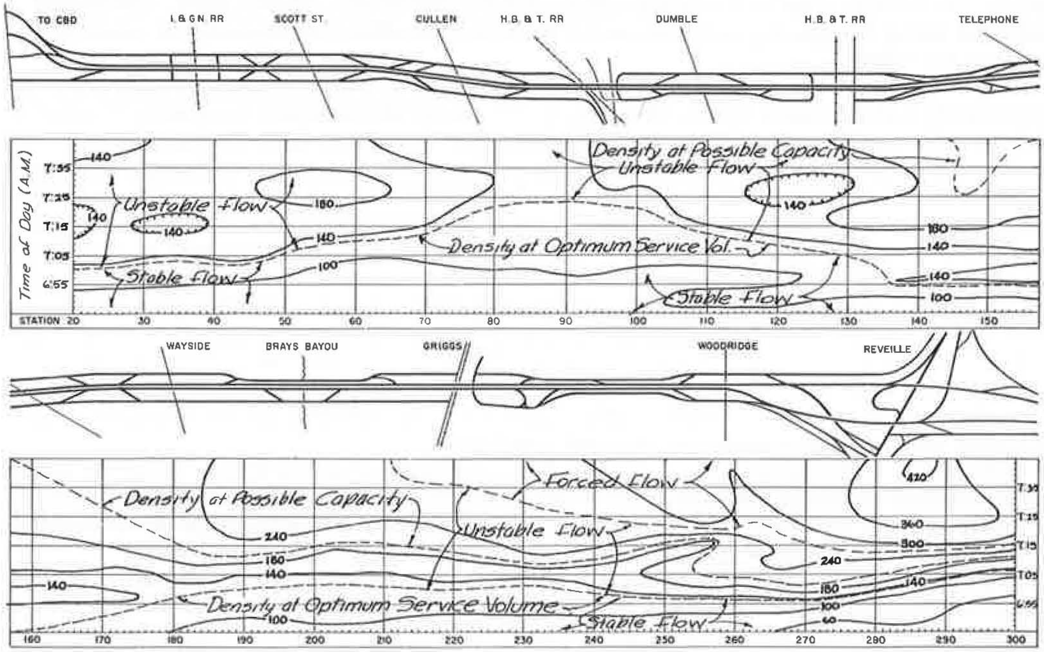

Figure 30. Density contours (3-lane total).

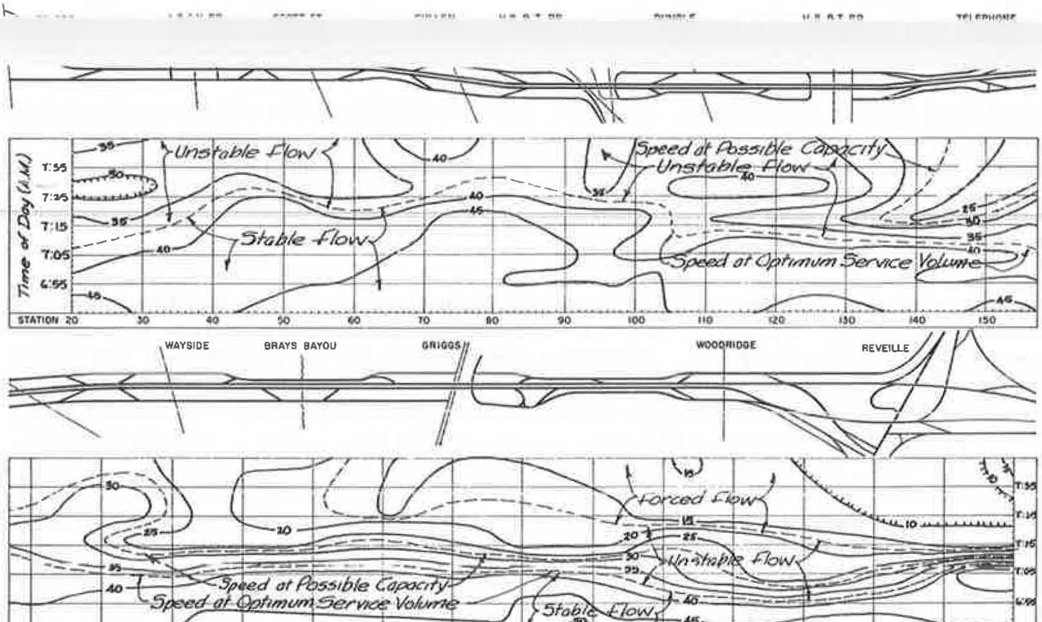

Figure 31. Speed contours (total inbound traffic).

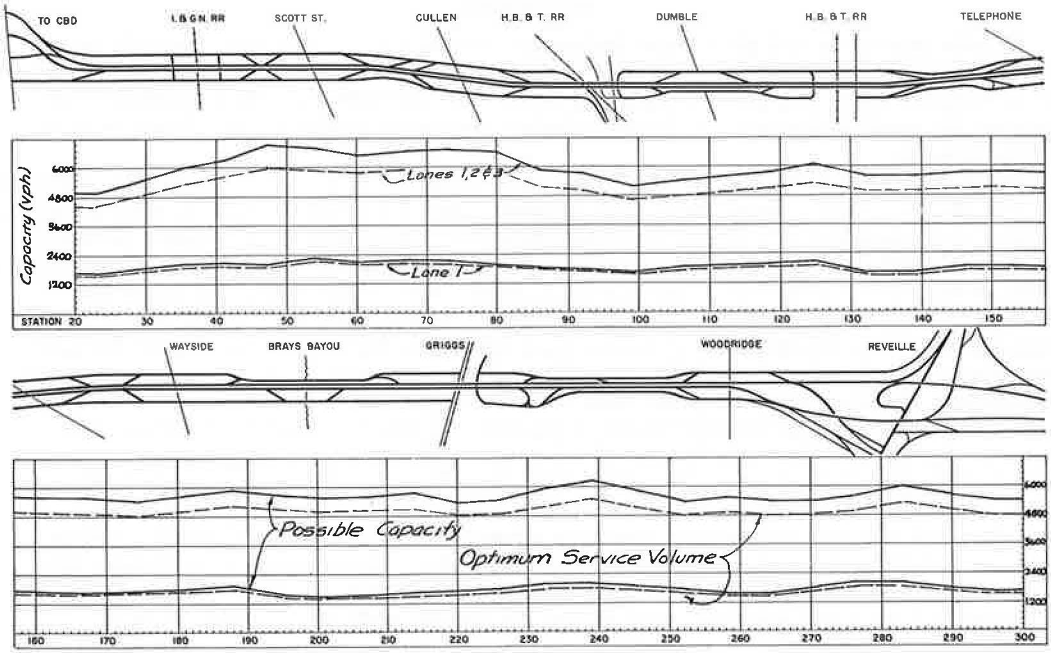

Figure 32. Capacity profiles (linear model).

the description and eventual control of freeway congestion was explored. Figures 30 and 31 illustrate the operation of the 3 inbound lanes of some six miles of the facility during the morning peak hour as obtained from time-lapse aerial photographic studies. Four control parameters, derived in the previous section, are superimposed on the contour maps: (1) the speed at possible capacity, u_m; (2) the density at possible capacity, k_m; (3) the speed at the optimum service volume, u'_m; and (4) the density at the optimum service volume, k'_m. These parameters afford a rational, quantitative means for describing the level of operation on the facility: stable flow, unstable flow and forced flow.

Figure 32 illustrates continuous profiles of the possible capacity, q_m, and the optimum service volume, q'_m, which were derived by applying the momentum-energy analogy to speed-density data taken from aerials of the facility. Thus, if stable flow is to be maintained on the facility, demand must be kept below the optimum service volume. Use of possible capacity as a basis for ramp metering or control places operation of the facility in the unstable zone of operation, and provides absolutely no safety factor against breakdowns due to statistical variability in demand.

Efforts to measure freeway operational efficiency in terms of traffic "throughput" (momentum) are obviously inconsistent with the level of service (energy) concept, because maximum throughput must necessarily be achieved with a high traffic stream density, a low traffic stream speed, and a level of operation typified by "unstable flow." On the other hand, the optimum service volume provides for speeds 33 percent higher, densities 33 percent lower, a level of operation typified by "stable flow," and with only a 10 percent reduction in flow. Actually, because there is less probability of attaining "forced flow" (congested flow inevitably accompanied by complete breakdown), the "throughput" from day to day might very well be higher because of less frequent breakdowns.

SUMMARY

Traffic operation and geometric design are essentially systematic attempts to resolve a demand-capacity relationship for a given facility in a manner that will provide an acceptable level of service to the motorist. This report deals with urban freeway volume (demand) and capacity characteristics, and their application to freeway design and operation.

An important volume characteristic which is discussed is the distribution of demand during the peak hour. Peak rates of flow within the peak hour exceed the average hourly rate of flow on urban freeways. It is important that the peak hourly volume presently used as a criterion for design be expanded to accommodate the higher rates of flow which exist over shorter intervals within the peak hour. This is true because a condition in which the demand exceeds the capacity can extend congestion for a much longer time than just the duration of the peak flow period (see Fig. 24). In Figure 1 this variation has been related to the size of the city, thus affording a means of estimating the highest 5-min rate of flow during the peak hour on a facility.

Another vital volume characteristic considered is the lane use distribution of vehicles on freeways. The outside lane of a freeway will, on the average, have a lower volume than the other lanes. The most important factors influencing the percent of the total freeway traffic in the outside lane immediately upstream from an entrance ramp are the total freeway volume and entrance ramp volume (see Figs. 5 and 9). Other factors are the sequencing of entrance and exit ramps, their spacing, and their volumes. (See the nomographs illustrated in Figs. 6, 7 and 8.)

A second aspect of lane use distribution discussed is the relationship between trip length and outside lane utilization obtained through the use of a "lights-on" study technique (Figs. 10, 11 and 12). This relationship was useful in establishing the minimum length of freeway weaving sections based on entrance and exit ramp volumes (see Fig. 26).

The second section of this report deals with freeway capacity characteristics. A theoretical approach to providing a rational relationship between capacity and level of service is formulated utilizing a hydrodynamic model and based on an energy-momentum [illegible] description of level of service is suggested in the preparation of a table of Freeway Design Service Volumes (Table 3) to be used in determining freeway main lane requirements. Fundamental volume-speed-density relationships (Fig. 21) and their measurements (Figs. 14, 15 and 16) on urban freeways are discussed. The importance of the contour format as a means of presenting a complete picture of performance over long sections of freeway and extended intervals of time is also described in this section.

It is significant that the first section of the body of the report deals with freeway demand and the second section with capacity. In the third section, applications are made of these demand and capacity characteristics to freeway operations (Figs. 30, 31 and 32) and to freeway design (Figs. 26, 27, 28 and 29), featuring a step-by-step design procedure.

ACKNOWLEDGMENTS

This paper is a summary of several years of research conducted by many researchers in the Texas Transportation Institute. We acknowledge the studies and unpublished work of the following individuals which have been incorporated in this paper: "Peaking Characteristics on Urban Freeways," by J. Royce Ginn; "Freeway Level of Service as Influenced by Peaking and Lane Use Characteristics," by J. J. Haynes; "Lights-On Study," by Dennis W. Jones; and "The Effect of Ramps on the Lateral Distribution of Vehicles on a Six-Lane Freeway," by John Lipscomb.

Sincere appreciation is expressed to Dr. Charles Pinnell, Supervisor of the Gulf Freeway Surveillance Project, for his inspiration concerning the theoretical approaches to capacity characteristics, and his cooperation in devoting one phase of the project research to the testing and application of these theories.

Gratitude is also expressed to the HRB Committee on Highway Capacity, and to many of the individual members whose ideas are included in this report.

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Discussion

ADOLF D. MAY, JR., Director, Expressway Surveillance Project, Illinois Division of Highways—This paper contributes significantly to the existing knowledge in freeway capacity and operations. The following discussion concentrates on three aspects of the paper: (1) linear speed density relationships, (2) density as an indication of levels of service, and (3) effect of levels of service on fuel economy.

The concepts of maximizing kinetic energy or traffic stream momentum as presented in Figure 13 are based on linear speed-density relationships, and because of the importance of these concepts some confirmation of the linearity of the speed-density relationship seems appropriate to mention in this discussion. Data obtained by the Chicago Area Surveillance Project has been analyzed (15) in order to evaluate the linearity of 1-min average speed and density observations, and the results are shown in Figure 33. Table 5 is a summary comparing the characteristics of the linear speed-density relationship obtained by the authors and the discussor.

The only significant difference between the two sets of data was that Drew-Keese's jam concentration was higher, and therefore the maximum flow was higher. The r^2 value of 0.88 indicates a rather strong linear speed-density relationship in that 88 percent of the variation in speed can be explained by density using a linear relationship. The σ value of 4.4 indicates that 95 percent of the observed minute speed measurements will lie within ± 8.8 mph of the speeds obtained by the equation. The new data introduced support the use of the linear speed-density relationship, although there are indications that well-selected nonlinear equations might provide even a higher correlation.

TABLE 5

Characteristics	Drew-Keese Data	Discussor's Data
Equation:	$\mu = 60.3 \left[1 - \frac{k}{133.1}\right]$	$\mu = 61.1 \left[1 - \frac{k}{117.7}\right]$
Mean free-flow speed (u_f)	60.3	61.1
Jam concentration (k_j)	133.1	117.7
Optimum speed (U_m)	30.2	30.6
Optimum density (K_m)	66.6	58.8
Maximum flow (Q_m)	2,006	1,798
Correlation coefficient (r)	0.97	0.94
Coefficient of determination $(r)^2$	0.95	0.88
Std. dev. of regression (σ)	3.2	4.4

TABLE 6

Level of Service Zone	Zone Limits	
	Upper	Lower
A	$K_j/10$ (13, 12)	0
B, C, D	$K_j/3$ (44, 39)	$K_j/10$ (13, 12)
E_1	$K_j/2$ (67, 59)	$K_j/3$ (44, 39)
E_2	$2K_j/3$ (39, 73)	$K_j/2$ (67, 59)
[illegible]	[illegible]	[illegible]

Table 4 of the Drew-Keese paper presents the levels of service as established by the energy-momentum concept. The authors use the combination of speed and volume to establish limits for the various levels of service. The consideration of this concept as a guide for operating a freeway surveillance and control system, and the authors comment in their introduction that congestion can quantitatively be defined in terms of vehicular density, led the discussor to investigate the use of density as the parameter to establish limits for the various levels of service for operating sections of freeways (see Fig. 34). The resulting density values for the various levels of service are shown in Table 6. The normalized limits in terms of k_j are given as well as the actual density levels in vehicles per lane-mile based on Drew-Keese's data and the discussor's data.

For the two sets of data, a lane density of approximately 40 veh/mi is the limit between stable flow (levels of service B, C and D) and unstable flow (level of service E_1), and is the optimum density based on maximizing kinetic energy. A lane density of approximately 60 veh/mi is the limit between the two levels of service (E_1 and E_2) which represent unstable flow, and is the optimum density based on maximizing traffic stream momentum. A lane density of approximately 80 veh/mi is the limit between unstable flow (level of service E_2) and forced flow (level of service F). It is interesting to note that a density of 40 veh/mi (15 percent occupancy) is currently used as the control parameter level for initiating ramp metering on the Eisenhower Expressway. If density continues to increase, the metering rate is reduced until at a density level of 67 veh/mi (25 percent occupancy) the most restrictive ramp metering rate is selected.

In the introduction the authors indicated that vehicular operational cost is one of the factors influencing level of service. In a recent study (16) on a 3.13-mi section of the

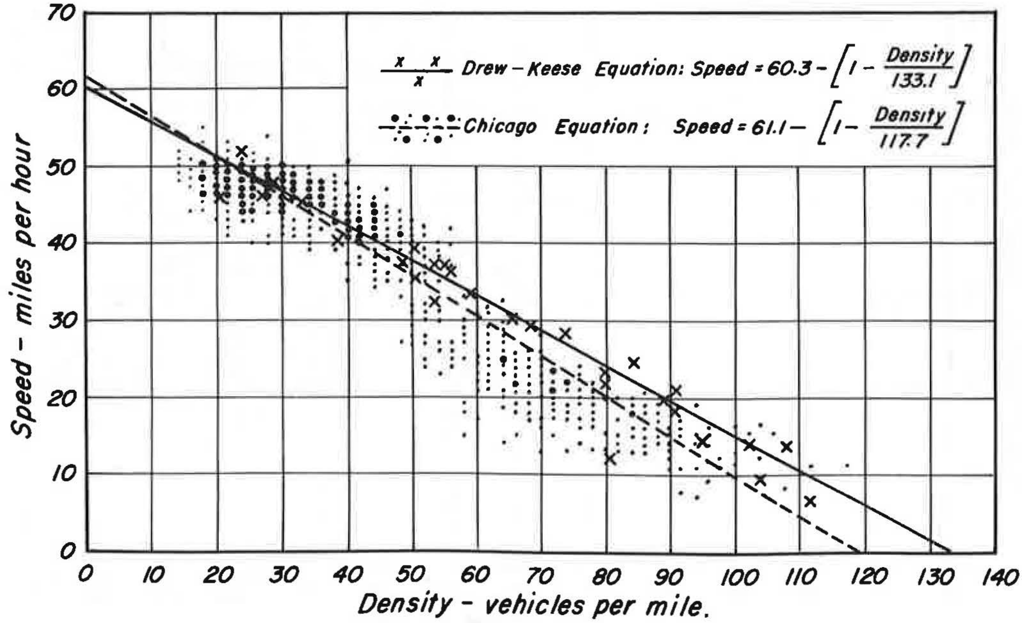

Figure 33. Speed-density relationship.

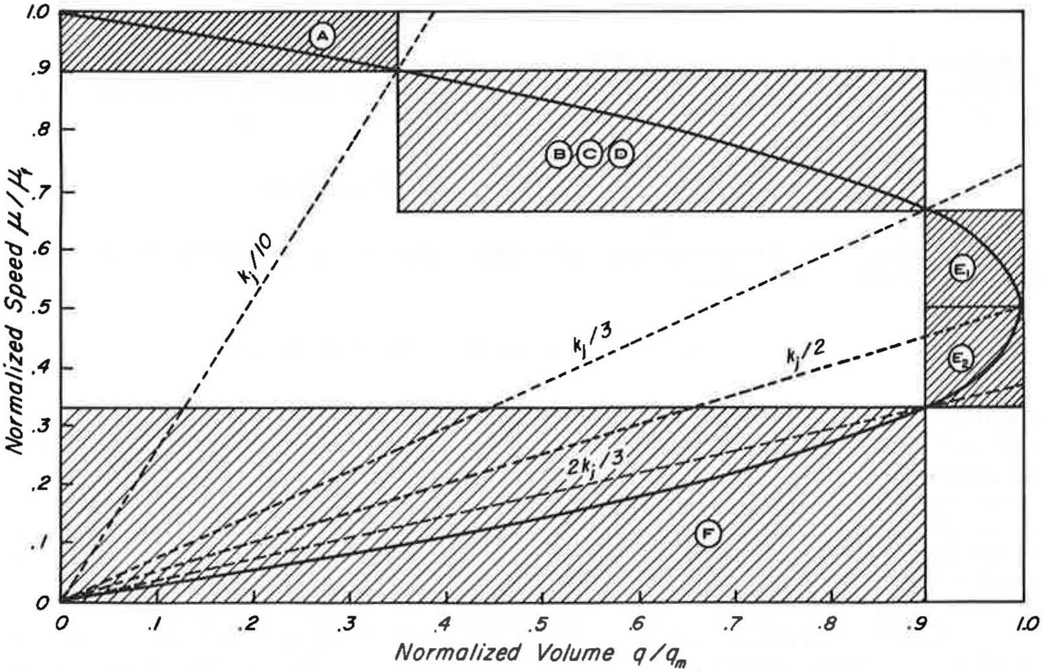

Figure 34. Levels of service related to density.

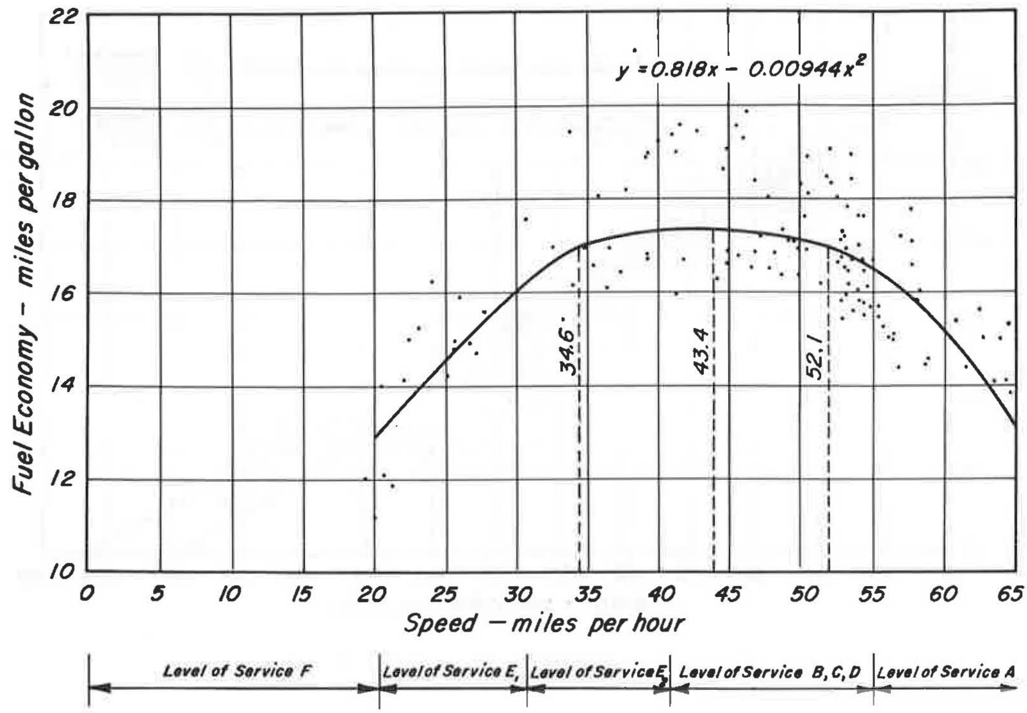

Figure 35. Fuel economy vs speed.

of fuel economy to speed versus speed. The resulting equation was

$$\frac{\text{Fuel economy}}{\text{Speed}} = 0.818 - 0.00944 \text{ (speed)}$$

and an r^2 value of 0.92 was obtained. The above equation was transformed into a fuel economy versus speed equation

$$\text{Fuel economy} = 0.818 \text{ (speed)} - 0.00944 \text{ (speed)}^2$$

and is shown in Figure 35. The highest fuel economy was obtained at a speed of 43.4 mph, and fuel economies greater than 17.0 mpg were obtained at speeds between 34.6 and 52.1 mph.

Using the density levels calculated in the earlier portion of this discussion, and the linear speed-density equation, the levels of service were determined and are shown in Figure 35. It can be seen that better fuel economy is obtained at the optimized speed based on the kinetic energy concept than the optimized speed based on the momentum concept. This would seem to further support the kinetic energy concept.

In summary, this discussion of the paper: (1) supports the authors' use of a linear speed-density relationship as a basis for the kinetic energy and momentum concepts, (2) indicates that the kinetic energy and momentum concepts can be used as guides for operating existing expressways, and (3) presents evidence that fuel economy is higher when kinetic energy is maximized than when momentum is maximized.

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KARL MOSKOWITZ, Assistant Traffic Engineer, California Division of Highways, Sacramento—The authors have made a significant contribution, not only by introducing a new concept, the "energy analogy," but in collecting and collating a great deal of previous work done by them and others.

Each mile of a freeway flowing at a rate of 5,400 vph (one direction) at 15 mph is providing 5,400 vehicle-miles of transportation per hour, and each mile of another freeway flowing at a rate of 5,400 vph at 50 mph is also providing 5,400 vehicle-miles of transportation per hour. The quantity of service seems to be the same, yet the quality is obviously different, and if a person wants to describe quality in terms of volume (or flow) he cannot do it.

The product of flow and speed suggested by the authors may be helpful in providing a number that would overcome this anomaly, but until people get used to it, the usefulness might be limited. Some of us might still find it aids our understanding to use more than one parameter to describe flow; i.e., to keep flow and speed separate.

Drew and Keese reproduce in Figure 24(b) a graph that this reviewer has found very useful in understanding the relation of flow and delay at (and upstream of) a bottleneck, which is a very common occurrence on freeways and all other traffic facilities, including nonmotor vehicle facilities. Many investigators get involved with speed-volume curves (such as the right-hand side of Fig. 22), or flow-concentration curves (such as Fig. 23), which are based on observations that may or may not be upstream of a bottleneck (the points on the curves representing higher speeds are seldom upstream of bottlenecks), and they do not always realize that inferences drawn from the speed-volume curve or q - k curve may not be compatible with the facts as shown in Figure 24(b).

One of the most useful things about Figure 24(b) (reproduced herein with added notes and dimensions as Figure 36) is that it shows flow as a rate (i.e., a mathematical slope) which is time-related, instead of as a point which is not time-related. Since by definition flow is a rate (i.e., n/t where n is number of vehicles and t is time), Figure 36 is much easier for an engineer to understand. Engineers have great difficulty discerning the slope of a point, or the meaning of a slope represented by a point on a curve, or worse yet, a point which might be anywhere within an envelope represented by a curve.

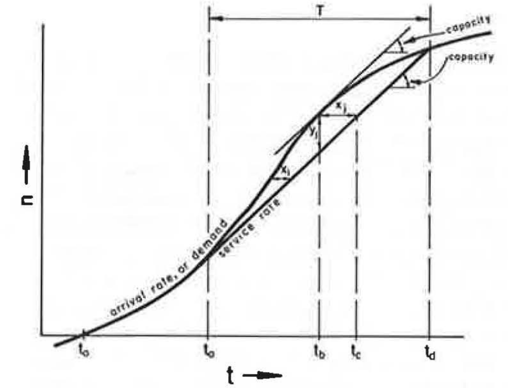

Figure 36. Relation between flow and delay.

Figure 36 represents cars going through a bottleneck as a function of time during a peak period. The slope of the upper curve at any point is the arrival rate and the slope of the lower curve is the service rate, or the actual flow. It can be seen that at t_a, the arrival rate begins to exceed the service rate. This might be 4:45 p.m. in a city like Sacramento, Calif., and many other cities in this country. At t_b, which might be 5:15 p.m., the arrival rate is equal to the service rate, and from t_b until t_d, the arrival rate is less than the service rate. At t_d, which might be 5:45 p.m., the congestion is over.

Some of our colleagues in the traffic assignment field use speed-volume curves, such as Figure 22 in Drew and Keese, to predict journey times as a function of demand. Yet it can be seen that the delay to an individual vehicle, x_j in Figure 36, is much greater at t_c (when vehicle j goes through the bottleneck) than it is at t_a, or at any other time of day between t_a and t_b, although the predicted demand is <u>less</u> at t_c than it is at any time between t_a and t_b. Using a speed-volume curve, a person would imply that the speed was close to 0, at $t_a + \Delta t$, just after the queue begins to form, because the volume/capacity ratio at $t_a + \Delta t$ is very high and is considerably greater than 1.0, whereas in fact the delay is very small, as can be seen by looking at the graph. The same person might infer that subsequent to t_b, speed would be very high because the demand/capacity ratio is less than 1, and yet a glance at Figure 36 shows that the j th vehicle, arriving at t_b, suffers the greatest delay of any vehicle in the universe we are here concerned with.

The actual flow rate stays nearly constant from t_a to t_d, but the journey time of any individual vehicle varies considerably, and the queue length (y_j) also varies. The spot speed immediately upstream of the bottleneck will be variable, from 0 up, on a moment-to-moment basis, and this is what Drew and Keese as well as other investigators call "forced flow." The spot speed in the bottleneck or downstream of it will be more uniform during time and will vary depending on how far downstream the observation is made, because cars will generally be accelerating. If the bottleneck is a merging area on a freeway, the volume or flow upstream of the bottleneck is less than the volume in the bottleneck by the amount added at the ramp; i.e., the ramp volume plus the freeway volume is equal to the downstream volume.

A third axis, namely distance, can be added to Figure 36 and the resulting graph would be a surface that truly portrays traffic flow in a single file, because speed would show. The slope of this surface in one plane would be flow (this is the plane shown in Figure 36); in the second plane would be speed and in the third plane, density. An examination of this surface during periods of forced flow would show that speed-volume-density relationships are far too complex to describe on a rate vs rate graph (i.e., flow vs concentration or flow vs speed).

But the loss of time, or delay, owing to this speed reduction is so small in comparison with the delay that occurs when the arrival rate exceeds capacity, that it really is not very important to know much about in urban freeway operational problems. The reduction in level of service, or subjective reactions of drivers, in this range is related more to nervous tension and necessity for continuous concentration on the driving task than it is to speed or delay.

J. W. HESS, <u>U. S. Bureau of Public Roads</u>—This report deals with volume characteristics and capacity characteristics which affect traffic operation, and thus the level of service, on freeways. Applications of the developed design tools are given. I am very impressed by the comprehensiveness and thoroughness of this report. The Texas Transportation Institute engineers working on the HPR "level of service" project in Texas have turned out a remarkable quantity and quality of research in this past year. I understand the material in this report has been under development for several years.

The subject of peaking is studied as related to the population of the city or urban area. For instance, the level of service D for a 6-lane freeway has a maximum 5-min volume rate of 5,250 when the design is for 4,800 vph for a city of 5 million metropolitan area population. A higher maximum 5-min rate would be found in other cities of smaller population (for instance, a 5,600-vph rate for a city of 1,000,000). My own studies have shown the importance of population in high volume merging. The population of a large city represents a steadier demand, greater driver experience, and more aggressive driver behavior. The latter should not be misconstrued as meaning drivers in large cities are more rude. I feel that they are actually more polite in freeway driving because they do not insist on their right-of-way and will help open a gap in the stream at merging areas.

Perhaps the indications are that smaller cities should design at a higher level of service because of greater peaking rates and poorer traffic performance. An alternative is to use lower-service volumes. Traffic performance should improve as the freeway systems are developed, but peaking will probably not change much.

The authors had trouble relating factors which might have an effect on the maximum rate of flow occurring in a peak hour. Such factors were physical size of the metropolitan area, distance of the study site from the CBD or principal traffic generator, the actual size of the peak period, itself, etc. Other engineers who have tackeled this subject have encountered the same trouble. Perhaps, the main reason for the lack of correlation is the incompleteness of traffic studies as far as a simultaneous study of traffic movement over the complete area of influence.

The design charts presented for use in determining lane 1 volumes at entrance ramp locations gave answers very similar to those obtained by my own nomographs. The only differences of any size were at successive on-ramps for 6-lane freeways where the authors' lane 1 volumes are 150-200 vph lower, and lane 1 volumes on 8-lane freeways where my volumes are lower. I have one specific question—does ± 8 percent connote ± 8 percentage points or ± 8 percent?

I was especially interested in the results of the "lights on" study because our weaving study crew made four such studies in Detroit in September 1963. The authors' study instructed the drivers to keep their lights on for 20 minutes, whereas our studies instructed drivers to keep their lights on for 2 miles. Does this mean that there is more congestion in Texas or do Texans tend to be "clockwatchers?" Seriously, we were studying the same thing—the lane usage of ramp vehicles downstream after merging. Our manpower allowed only a distance of 5, 000-6, 000 feet downstream for study purposes, whereas in the authors' study the distance was over 10, 000 feet. The curves were very similar, with the authors' study curve falling along the same line as our uppermost curve. One difference in the respective studies is that the authors' results showed more vehicles remaining in lane 1 (after merging) at volumes above 3, 000 vph, whereas our results showed less staying in lane 1 at the higher volume rates. Although it is more difficult to vacate lane 1 at higher volume rates, it also appears to be more desirable if the driver has the motivation of improving his position in the traffic stream. Northwestern University has recently done some similar work for left-hand ramps, both entrance and exit. The study method used by them utilized a series of time-lapse cameras mounted on overpasses. All of this information is very important in determining the operation to be expected for a given design, and it may well give us our best traffic evaluation for interchange spacing. Lane usage by prospective off-ramp vehicles upstream from their exit ramp is a vital cog still missing in this evaluation of interchange effects. I can also see an important use of this information in simulation of the traffic stream.

In the "Capacity Characteristics" section of the report, the following can be quoted—"Both capacity and level of service are functions of the physical features of the highway facility and the interaction of vehicles in the traffic stream. The distinction is this: A given lane or roadway may provide a wide range of levels of service, but only one possible capacity...." I wonder about the "one possible capacity." I believe that the interaction of vehicles can change, and with it, the possible capacity, for instance, at a ramp merging area where the ratio of lane 1 to the ramp might change considerably.

The energy-momentum analogy developed in this report to provide a quantitative approach to the level of service-capacity relationship should be studied by everyone interested in the developments behind the new Capacity Manual approach. This is not to say that the analogy led to the Capacity Manual approach—only that it helps explain some of the speed-volume-density relationships.

The last part of this section on "Capacity Characteristics" presents a very interesting "Probabilities of Types of Flow" table. It would be especially useful if eventually these probabilities could be related to the design standards or features of the freeway. The actual freeway design, especially at a bottleneck area, as well as the metropolitan population which is used should be a strong factor.

In the "Applications" section of this report, the authors discuss critical sections along the freeway such as merging and weaving areas. Though no procedures are given in this report, intervehicular conflict upstream from an exit ramp is also mentioned as a problem. Ramp vehicles tend to decelerate on the through freeway lane and this is a section of vehicle concentration. This problem is being considered in the new Highway Capacity Manual by the assignment of a service volume level to the lane 1 immediately upstream of the exit ramp. It is interesting to note that in the report's design problem of a length of freeway, application of the exit ramp equations points up at least three locations where the 1,700-vph service volume is exceeded, if only slightly. However, service volumes for diverges could logically be slightly higher than those for merges.

The "Graph for Determination of Minimum Length of Weaving Section to Meet the Design Level of Service" could be very useful. I note, by the slope of the weaving distance lines, that entrance ramp volumes are considered more critical than exit ramp volumes—this lends credence to my belief that lane 1 volume upstream from an exit ramp should have a higher service volume (perhaps 100 vph) than a merge.

Finally, the authors mention the gap acceptance method of evaluating entrance ramp capacity. I certainly agree that more research is needed in this area to cover, for instance, aside from geometrics, the effect of intensity of light, inclement weather, driver experience, etc. The measurement of gap arrivals and the metering of ramp vehicles to fill these gaps is to become of more importance. In this vein, we can no longer ignore freeway operation during more or less adverse conditions of weather. I guess we literally will have to get our feet wet in the near future if we are to take an all-conditions approach to freeway operation.

DONALD R. DREW and CHARLES J. KEESE, Closure—The authors are appreciative of the excellent reviews by Dr. A. D. May, Mr. Karl Moskowitz and Mr. Joseph W. [illegible] tics and operation (May), freeway capacity (Moskowitz), and ramp capacity (Hess), because these gentlemen have made particularly important contributions in these respective areas.

It is encouraging to note that Dr. May's data based on lane occupancy tend to substantiate the model developed in this paper, and that the optimum speed based on fuel consumption is in agreement with the energy-momentum concept for defining level of service. Vehicular operating cost is one of the factors considered in the level of service concept. The fuel data were collected by Donald G. Capelle for use in his Doctoral Dissertation at Texas A & M University. His studies were conducted on the Eisenhower Expressway in cooperation with Dr. May and will be reported in detail at a later date.

We concur with Mr. Moskowitz regarding the utility of the relationship between flow and delay illustrated in Figure 36 reproduced, in part, in our Figure 24. However, the addition of the top half of Figure 24 serves to dramatize the fact that the duration of congestion can be appreciably longer than the duration in which demand exceeds capacity.

It is felt that the ramp capacity analyses conducted by Mr. Hess substantiate the procedure developed in this paper which was based on several highly selective locations. Mr. Hess has analyzed a great deal more data covering numerous diverse types of facilities and conditions. The fact that his findings generally substantiate the theoretical approach and practical application developed in this paper indicates that this complex operational characteristic can be analyzed and used with reasonable confidence in the solution of freeway design and operation problems.

These reviews lend support to the philosophy that the level of service concept (or quality of traffic service) can be quantified for use in design of new facilities and in the application of control measures to improve operation on existing facilities.

The authors again express appreciation to Messrs. Ginn, Haynes, Jones and Lipscomb who conducted the individual studies and to the various members of the staff

of the Texas Transportation Institute who contributed to these studies. Gratitude is also expressed for the contribution of each member of the HRB Committee on Highway Capacity and particularly to May, Moskowitz and Hess for their reviews.

Deterministic Aspects of Freeway Operations and Control

DONALD R. DREW, Associate Research Engineer, Texas Transportation Institute

• AMONG the important problems arising from the population explosion is that of congestion. Although this overcrowding manifests itself in virtually every aspect of modern life, nowhere is it as dramatically exhibited as on our streets and highways. The most vigorous attempt to eliminate traffic congestion was the development of the freeway, a concept based on (a) the reduction of vehicle-to-vehicle conflicts, (b) elimination of vehicle-to-pedestrian conflicts, and (c) elimination of delay-producing traffic control devices. Still, practically all major cities are troubled with severe peak hour congestion on newly completed freeways.

Previous studies have shown that a relatively small increase in traffic demand on an already heavily loaded expressway can have a very detrimental effect on the operating conditions for all traffic on the facility. Speeds and volumes are reduced, densities and travel times are increased, and the highway immediately loses much of its efficiency. Theoretically, it seems desirable to either ration or completely deny access to the freeway at certain locations.

The automatic evaluation of freeway traffic flow will be a vital element of any future control system. Research must be directed toward the evaluation of the use of surveillance and sensing equipment, and the simultaneous investigation of those characteristics of traffic flow related to freeway congestion which can be determined and treated by such equipment. The complexities and manifestations of freeway traffic congestion are

comfort. These factors are influenced by such additional variables as traffic demand, traffic composition, lane occupancy, highway geometrics and the drivers' desired speeds. Before it can be decided just what level of efficiency is economically feasible, or stated another way, how much congestion should be tolerated during peak periods, congestion must be defined quantitatively in terms of known and measurable parameters of traffic flow theory.

In recent years, a number of descriptive theories of vehicular traffic have been put forward. These theories are based on mathematical models of two basic types: deterministic and stochastic. Included in the first category are the continuous flow models and individual vehicle models which describe the macroscopic and microscopic properties, respectively, of the traffic flow phenomena. Included in the second group are the probability distribution hypotheses and queueing theory.

GENERALIZATION OF DETERMINISTIC MODELS OF TRAFFIC FLOW

If vehicular traffic is assumed to behave as a one-dimensional compressible fluid of concentration (density), k, and fluid velocity, u, then the conservation of vehicles is explained by

$$\frac{\partial k}{\partial t} + \frac{\partial (ku)}{\partial x} = 0 \tag{1}$$

Taking the derivative of the product in the second term yields

Paper sponsored by Committee on Freeway Operations.

$$\frac{\partial k}{\partial t} + \frac{u\,\partial k}{\partial x} + \frac{k\,\partial u}{\partial x} = 0 \tag{2}$$

It is well established in the theory of traffic flow that vehicular velocity varies inversely with the concentration of vehicles,

$$u = f\,(k) \tag{3}$$

As a consequence of Eq. 3,

$$\frac{\partial u}{\partial k} = \frac{\partial u}{\partial x}\frac{\partial x}{\partial k} = \frac{du}{dk} = u' \tag{4}$$

Solving for $\partial u/\partial x$ from Eq. 4 and substituting in Eq. 2, one obtains the following equation of continuity for single-lane vehicular traffic flow,

$$\frac{\partial k}{\partial t} + [u + ku']\frac{\partial k}{\partial x} = 0 \tag{5}$$

Now, if it is assumed that a driver adjusts his velocity at any instant in accordance with the traffic conditions about him as expressed by $k^n \partial k/\partial x$, the acceleration of the traffic stream at a given place and time becomes

$$\frac{du}{dt} = -c^2 k^n \frac{\partial k}{\partial x} \tag{6}$$

Taking the total derivative of $u = f(x, t)$ gives

$$\frac{du}{dt} = \frac{\partial u}{\partial x}\frac{dx}{dt} + \frac{\partial u}{\partial t}\frac{dt}{dt} \tag{7}$$

where $dx/dt = u$ and $dt/dt = 1$. Substituting Eq. 7 in Eq. 6 yields

$$\frac{\partial u}{\partial x}u + \frac{\partial u}{\partial t} + c^2 k^n \frac{\partial k}{\partial x} = 0 \tag{8}$$

From Eq. 4, it is equally apparent that

$$\frac{\partial u}{\partial t} = u'\frac{\partial k}{\partial t} \tag{9}$$

By solving for $\partial u/\partial k$ from Eq. 4 and substituting in Eq. 8, substituting for Eq. 9 in Eq. 8, then dividing through by u', Eq. 8 becomes

$$\frac{\partial k}{\partial t} + \left[u + \frac{c^2 k^n}{u'}\right]\frac{\partial k}{\partial x} = 0 \tag{10}$$

which is the generalized equation of motion. The nontrivial solution of Eqs. 5 and 10 is obtained by equating the quantities within the brackets,

$$(u')^2 = c^2 k^{(n-1)} \tag{11}$$

Finally, because of the inverse relation between velocity and concentration,

$$u' = -ck^{(n-1)/2} \tag{12}$$

Greenberg (1) has solved Eq. 12 for n = -1 obtaining

$$u = c \ln (k_j/k) \tag{13}$$

The solution of Eq. 12 for n > -1 is as follows:

$$u = \frac{-2c}{(n+1)} k^{(n+1)/2} + C_1, \quad n > -1 \tag{14}$$

where the constant of integration is to be evaluated by the boundary conditions inherent in the vehicular velocity-concentration relationship. Thus, since no movement is possible at jam concentration, k_j,

$$C_1 = \frac{2c}{(n+1)} k_j^{(n+1)/2}, \quad n > -1 \tag{15}$$

and

$$u = \frac{2c}{(n+1)} \left[k_j^{(n+1)/2} - k^{(n+1)/2} \right], \quad n > -1 \tag{16}$$

Similarly, the implication exists that a driver is permitted his free speed, u_f, only when there are no other vehicles on the highway (k = 0). Therefore,

$$u_f = \frac{2c}{(n+1)} k_j^{(n+1)/2}, \quad n > -1 \tag{17}$$

and the constant of proportionality takes on the following physical significance:

$$c = \frac{}{2 k_j^{(n+1)/2}}, \quad n > -1 \tag{18}$$

Substitution of Eq. 18 in Eq. 16 yields the generalized equations of state,

$$u = u_f \left[1 - \left(\frac{k}{k_j} \right)^{(n+1)/2} \right], \quad n > -1 \tag{19}$$

$$q = ku = k u_f \left[1 - \left(\frac{k}{k_j} \right)^{(n+1)/2} \right], \quad n > -1 \tag{20}$$

Differentiation of Eq. 20 with respect to k equated to zero gives the optimum concentration, k_m, which is that concentration yielding the maximum flow of vehicles:

$$\frac{dq}{dk} = \left[1 - \frac{(n+3)\, k^{(n+1)/2}}{2 k_j^{(n+1)/2}} \right] u_f = 0$$

$$k_m = \left[(n+3)/2 \right]^{-2/(n+1)} k_j, \quad n > -1 \tag{21}$$

Substituting Eq. 21 in Eq. 19, one obtains the optimum velocity,

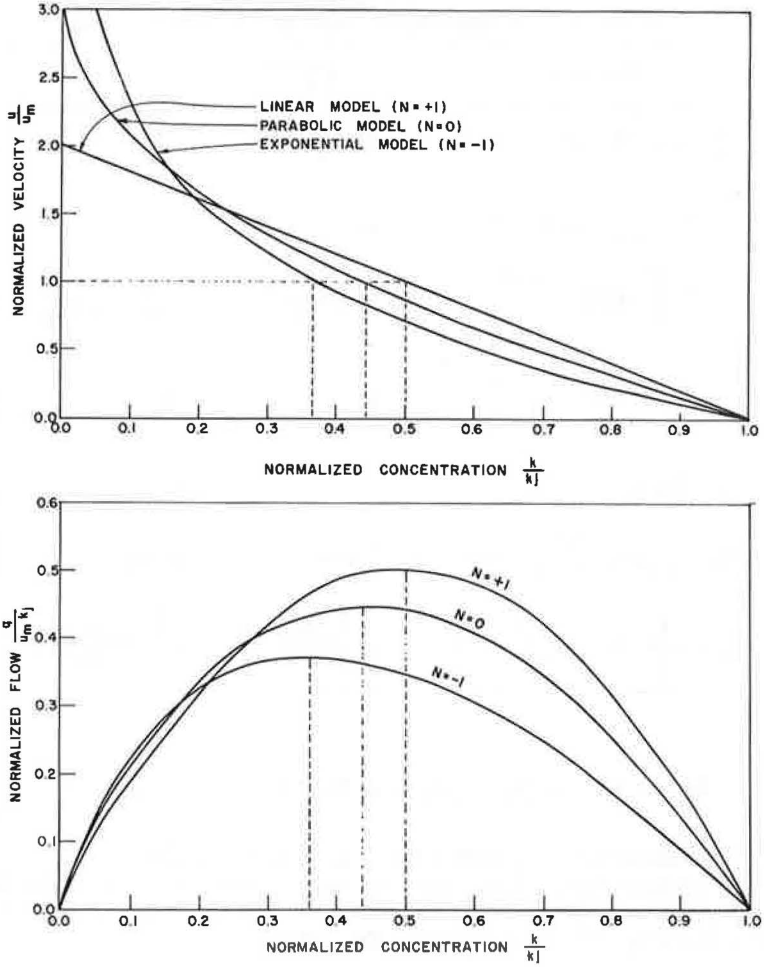

Figure 1. Solution of generalized equation of traffic motion, $\frac{du}{dt} + c^2 k^n \frac{\partial k}{\partial x} = 0$, for N = -1, 0, +1.

$$u_m = \left[\frac{n+1}{n+3}\right] u_f, \quad n > -1 \tag{22}$$

The maximum flow of vehicles of which the highway lane is capable (capacity) is obtained from the product of Eqs. 21 and 22

$$q_m = \left[\frac{(n+1)}{(1/2)^{2/(n+1)}\,(n+3)^{[2/(n+1)]+1}}\right] u_f\, k_j, \quad n > -1 \tag{23}$$

Some special cases of Eqs. 19 through 23 have proven to be of significance. Greenshields'(2) linear model is obtainable by setting n = 1, and Drew (3) has discussed the case for n = 0. These cases, as well as Greenberg's model, are summarized in Figure 1 and Table 1.

Typical of some of the car-following laws that have been proposed are those that express the performance of a vehicle in terms of its velocity and position with respect to the vehicle immediately preceding it,

TABLE 1

COMPARISON OF MACROSCOPIC MODELS OF TRAFFIC FLOW

Element	General (n > -1)	Exponential (n = -1)	Parabolic (n = 0)	Linear (n = 1)
Eq. of motion	$\frac{du}{dt} + c^2 k^n \frac{\partial k}{\partial x} = 0$	$\frac{du}{dt} + \frac{c^2 \partial k}{k \partial x} = 0$	$\frac{du}{dt} + c^2 \frac{\partial k}{\partial x} = 0$	$\frac{du}{dt} + c^2 k \frac{\partial k}{\partial x} = 0$
Constant of proportionality	$c = \left[(n+1) u_f\right] / 2k_j^{(n+1)/2}$	u_m	$u_f/2k_j^{1/2}$	u_f/k_j
Eq. of state	$q = ku_f \left[1 - \left(\frac{k}{k_j}\right)^{(n+1)/2}\right]$	$ku_m \ln\left(\frac{k_j}{k}\right)$	$ku_f \left[1 - \left(\frac{k}{k_j}\right)^{1/2}\right]$	$ku_f \left[1 - \frac{k}{k_j}\right]$
Optimum concentration	$k_m = \left[(n+3)/2\right]^{-2/(n+1)} k_j$	k_j/e	$4k_j/9$	$k_j/2$
Optimum speed	$u_m = \left[(n+1)/(n+3)\right] u_f$	c	$u_f/3$	$u_f/2$
Capacity	$q_m = \frac{(n+1)\, u_f k_j}{(1/2)^{2/(n+1)} (n+3)^{[2/(n+1)]+1}}$	$\frac{1}{e} u_m k_j$	$\frac{4}{27} u_f k_j$	$\frac{1}{4} u_f k_j$
Wave vel.	$\frac{dq}{dk} = u_f \left[1 - \frac{(n+3)}{2} \left(\frac{k}{k_j}\right)^{(n+1)/2}\right]$	$u_m \left[\ln\left(\frac{k_j}{k}\right) - 1\right]$	$u_f \left[1 - \frac{3}{2}\left(\frac{k}{k_j}\right)^{1/2}\right]$	$u_f \left[1 - \frac{2k}{k_j}\right]$

$$\ddot{x}_i (t + T) = a [\dot{x}_{i-1} (t) - \dot{x}_i (t)] [x_{i-1} (t) - x_i (t)]^{-m} \tag{24}$$

Eq. 24 states that the acceleration of a car, $\ddot{x}_i$, at a delayed time, T, is directly proportional to the relative speed of the car, $\dot{x}_i$, with respect to the one ahead, $\dot{x}_{i-1}$, and inversely proportional to the headway of the car, $x_{i-1} - x_i$. Since the right side of Eq. 1 is of the form dy/y^m, integration of Eq. 24 yields

$$\dot{x}_i (t + T) = a \ln [x_{i-1} (t) - x_i (t)] + C_1, \quad m = 1 \tag{25}$$

and

$$\dot{x}_i (t + T) = (-m + 1)^{-1} a [x_{i-1} (t) - x_i (t)]^{-m+1} + C_2, \quad m > 1 \tag{26}$$

The constants of integration are evaluated by observing that the velocity of a car approaches zero as its headway approaches the effective length of each car, L;

$$C_1 = a \ln L \tag{27}$$

$$C_2 = -(-m + 1)^{-1} a L^{-m+1}, \quad m > 1 \tag{28}$$

Substituting for C_1 and C_2, Eqs. 25 and 26 become

$$\dot{x}_i (t + T) = a \ln L^{-1} [x_{i-1} (t) - x_i (t)], \quad m = 1 \tag{29}$$

TABLE 2

COMPARISON OF MICROSCOPIC MODELS OF TRAFFIC FLOW

Element	General (m > 1)	m = 1	m = 3/2	m = 2
Eq. of motion	$\ddot{x}_i = \frac{a(\dot{x}_{i-1} - \dot{x}_i)}{(x_{i-1} - x_i)^m}$	$\ddot{x}_i = \frac{a(\dot{x}_{i-1} - \dot{x}_i)}{(x_{i-1} - x_i)}$	$\ddot{x}_i = \frac{a(\dot{x}_{i-1} - \dot{x}_i)}{(x_{i-1} - x_i)^{3/2}}$	$\ddot{x}_i = \frac{a(\dot{x}_{i-1} - \dot{x}_i)}{(x_{i-1} - x_i)^2}$
Constant of proportionality	$a = (m-1) u_f k_j^{-(m-1)}$	u_m	$u_f/2k_j^{1/2}$	u_f/k_j
Eq. of state	$q = ku_f \left[1 - \left(\frac{k}{k_j}\right)^{m-1}\right]$	$ku_m \ln\left(\frac{k_j}{k}\right)$	$ku_f \left[1 - \left(\frac{k}{k_j}\right)^{1/2}\right]$	$ku_f \left[1 - \left(\frac{k}{k_j}\right)\right]$
Macroscopic counterpart (see Table 1)	n = 2m - 3	n = -1	n = 0	n = +1

$$\dot{x}_i (t + T) = (m - 1)^{-1} a \left\{ L^{-(m-1)} - [x_{i-1}(t) - x_i(t)]^{-(m-1)} \right\}, \quad m > 1 \tag{30}$$

Eq. 29 is due to Gazis, Herman and Potts (4) who showed that the traffic equation of state could be derived from the microscopic car-following law just as the gas equation of state can be derived from the microscopic law of molecular interaction. Since the space headway is the reciprocal of concentration, k, Eqs. 29 and 30 become

$$u = a \ln (k_j/k) \tag{31}$$

and

$$u = (m - 1)^{-1} a \left(k_j^{m-1} - k^{m-1}\right), \quad m > 1 \tag{32}$$

The constant of proportionality is evaluated at $u = u_f$ and $k = 0$, giving

$$a = \left[\frac{(m-1)}{k_j^{m-1}}\right] u_f, \quad m > 1 \tag{33}$$

Special cases of Eq. 32, as well as the relationship of the macroscopic parameters c and n to the microscopic parameters a and m are given in Table 2.

APPLICATION OF DETERMINISTIC MODELS

The applicability of these deterministic models to freeway traffic was tested on the Gulf Freeway in Houston, Texas (Fig. 2). Time-lapse aerial photography with a 60 percent overlap was utilized to insure a given point on the freeway appearing on three consecutive photos (Fig. 3). Six flight runs were made in the direction of the traffic being studied, inbound during the morning peak. Since a given vehicle appeared on at least three consecutive photos, individual vehicular speeds, accelerations, and space headways were measured. The observations were compared (on a lane basis) to the three macroscopic models in Table 1 and the three microscopic models in Table 2.

Regression analyses based on the macroscopic hypotheses of Eqs. 13 and 19 (n = 0 and n = +1) are summarized in Table 3. Statistical tests were, in general, highly significant on each of the three freeway lanes, as well as on the total traffic on all three lanes. The microscopic analyses, however, were inconclusive. A constant of proportionality, a, was calculated for every freeway vehicle based on its performance and position with respect to the vehicle in front of it. The physical significance of a is indicated in Table 2 for the three microscopic models tested. The values obtained were

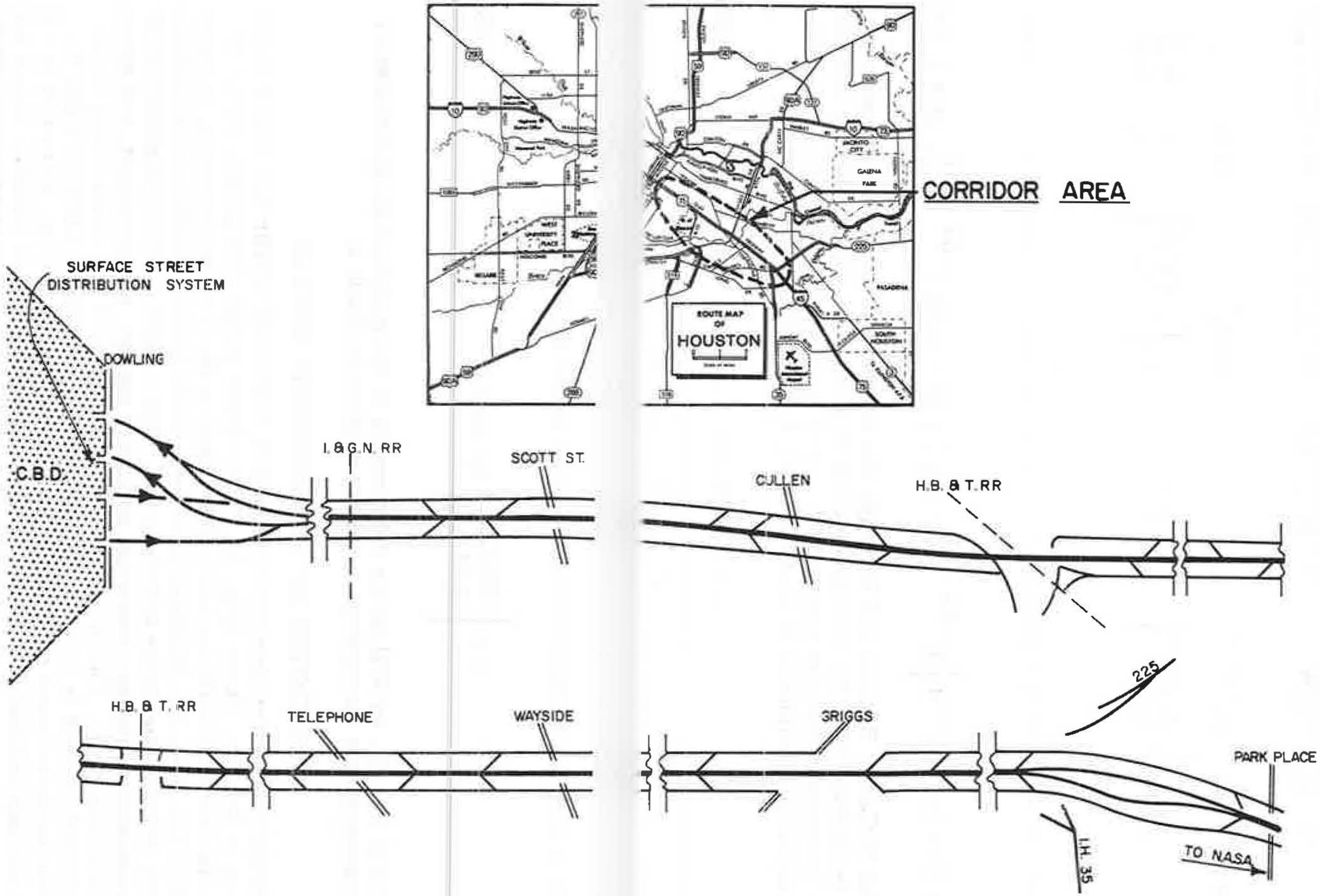

Figure 2. Study [illegible], Gulf Freeway, Houston.

TABLE 3

REGRESSION ANALYSES OF EQUATIONS OF STATE (3 LANE TOTAL)

Station	$u = a - bk$			$u = a - bk^{1/2}$		$\ln k = a - bu$	
	b	a	t	b	a	b	a
306-288	0.129	52.8	7.52**	3.32	71.0	0.044	6.20
299-281	0.115	50.9	32.16**	3.14	69.2	0.050	6.35
292-274	0.112	52.0	18.37**	3.20	72.0	0.047	6.41
286-268	0.132	54.3	40.60**	3.47	74.6	0.046	6.35
280-262	0.131	53.3	30.60**	3.34	72.2	0.048	6.38
273-255	0.142	55.3	13.49**	3.74	78.1	0.041	6.26
267-249	0.141	56.0	11.87**	3.89	81.3	0.038	6.25
261-243	0.102	46.7	4.98**	2.09	54.4	0.069	6.77
254-236	0.143	58.0	7.79**	4.03	84.8	0.035	6.20
248-230	0.173	66.1	20.77**	4.66	95.5	0.032	6.18
241-223	0.175	64.6	11.53**	4.56	92.6	0.032	6.15
235-217	0.181	63.0	10.93**	4.67	91.9	0.032	6.09
229-211	0.167	59.7	4.85**	4.32	86.6	0.032	6.06
223-205	0.182	64.5	15.84**	4.91	96.5	0.030	6.08
216-198	0.205	67.8	10.00**	5.33	101.5	0.028	6.00
210-192	0.176	62.3	8.82**	4.45	89.2	0.035	6.17
204-186	0.190	65.5	19.03**	4.86	95.4	0.032	6.12
197-179	0.176	64.3	14.32**	4.55	92.6	0.033	6.20
190-172	0.197	66.4	7.99**	5.11	99.0	0.028	6.04
183-166	0.200	65.9	4.97**	5.01	97.0	0.027	5.97
176-158	0.181	63.2	7.12**	4.57	91.8	0.032	6.16
169-151	0.179	63.0	11.14**	4.31	88.5	0.036	6.28
162-144	0.154	60.0	8.78**	3.59	79.7	0.043	6.51
155-137	0.157	60.1	4.18*	3.78	82.3	0.034	6.23
148-130	0.167	61.3	6.06**	4.03	85.1	0.035	6.25
141-123	0.158	61.2	5.03**	3.68	82.2	0.038	6.41
134-118	0.140	58.3	4.00*	3.18	76.0	0.042	6.50
128-110	0.145	58.1	3.60*	3.19	75.4	0.041	6.44
121-103	0.051	45.6	0.90	1.08	51.2	0.029	5.91
115-97	0.153	57.8	3.93*	3.14	73.7	0.049	6.71
108-90	0.222	66.4	4.85**	4.75	91.7	0.034	6.12
101-83	0.194	64.8	2.67	4.14	86.8	0.028	5.94
95-77	0.165	61.9	1.65	3.35	78.7	0.023	5.67
89-71	0.176	64.0	1.47	3.79	84.2	0.021	5.62
82-64	0.126	58.0	2.87*	2.75	72.7	0.048	6.81
76-58	0.121	58.0	3.16*	2.67	71.7	0.053	6.99
69-51	0.127	57.7	2.73*	2.98	74.8	0.043	6.62
63-45	0.114	55.7	2.80*	2.67	71.1	0.049	6.86
56-38	0.110	55.5	1.95	2.48	69.1	0.039	6.51
50-32	0.131	57.6	3.79*	2.88	73.1	0.051	6.91
44-26	0.122	54.2	3.37*	2.62	67.9	0.053	6.88
37-19	0.137	54.2	4.60**	2.96	69.8	0.053	6.75

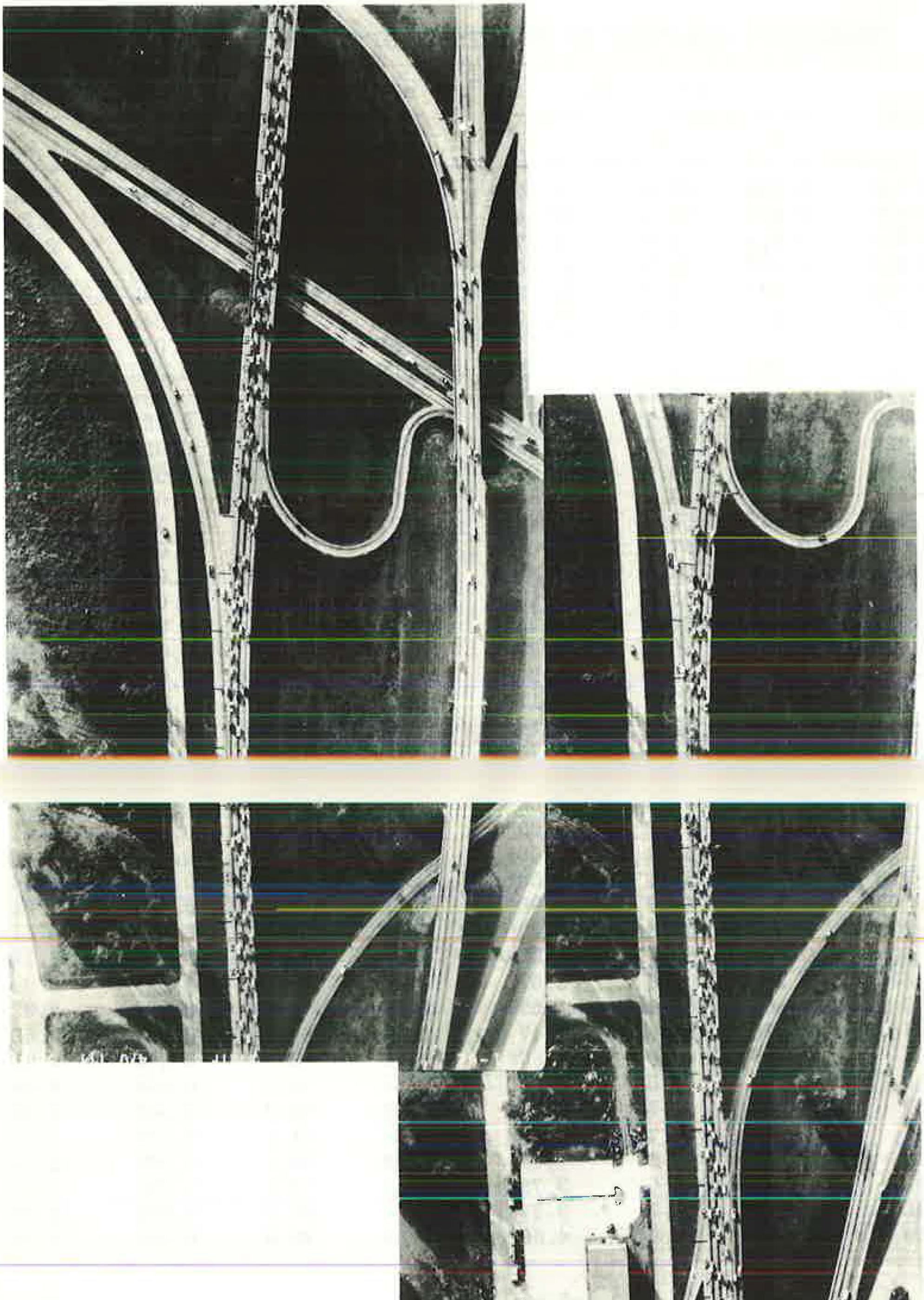

Figure 3. [illegible]-lapse photography.

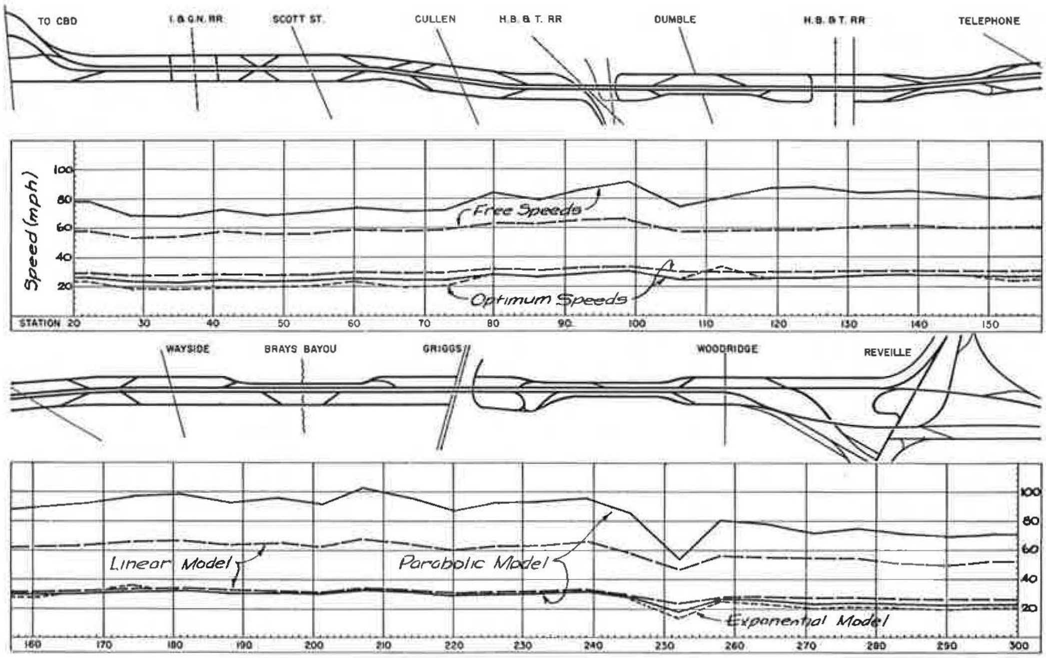

Figure 4. Speed profiles (total inbound traffic).

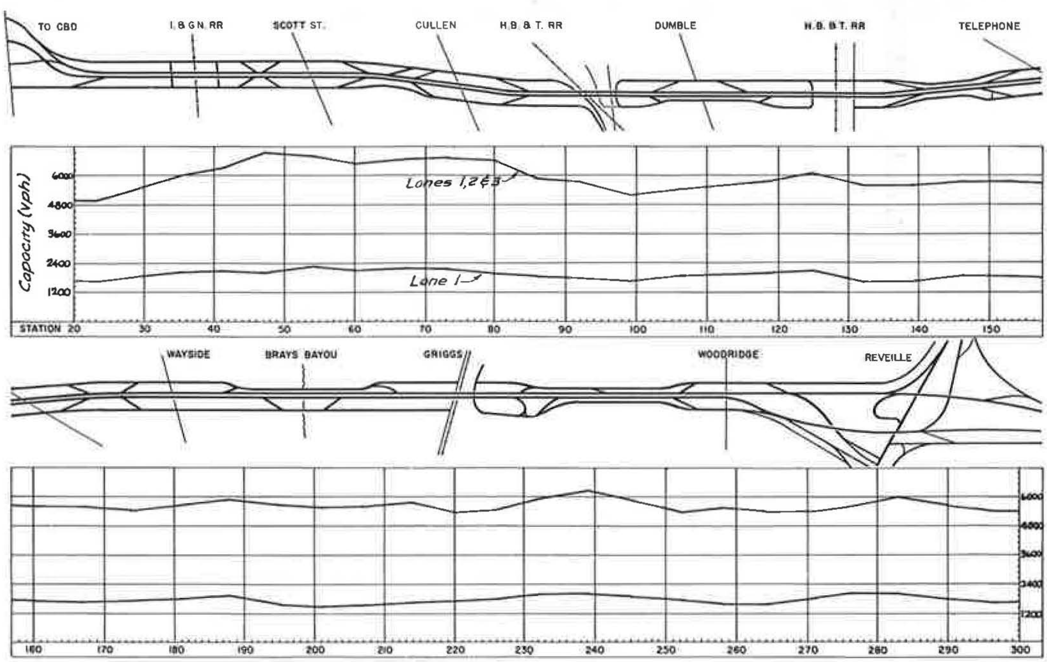

Figure 5. Capacity profiles (linear model).

extremely variable; approximately one-eighth of the values were negative indicating that, even under conditions of heavy traffic, the opportunity for changing lanes reduces a driver's necessity to respond to the performance of the car in front of him.

Essential to the development of freeway control techniques is the determination of suitable control parameters. Among the many techniques for controlling freeway traffic, ramp metering at entrance ramps and changeable advisory speed limit signs located on the freeway itself offer the most promise. Capacity, q_m, and optimum speed, u_m, represent two ideal control parameters. Figures 4 and 5 show continuous speed and capacity profiles for the outside lane of the 6-mi stretch of the Gulf Freeway. Free speeds, u_f, are also shown in Figure 4 for the linear and parabolic models ($u_f = \infty$ for the exponential model).

Because control of vehicles entering the freeway, as against 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. Entrance ramp metering may be oriented to either the freeway capacity or freeway demand. A capacity-oriented ramp control system restricts the volume rate on the entrance ramps to prevent the flow rates at downstream bottlenecks from exceeding the capacities of the bottlenecks. Figure 5 shows a capacity profile for traffic on all three inbound lanes of the Gulf Freeway. Bottleneck sections along with their respective control capacities are evident.

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Some Considerations of Vehicular Density on Urban Freeways

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•A WIDESPREAD urban freeway problem is that of the overcrowding or congestion which results from the peak traffic demands. Work traffic is customarily associated with the peak demand so that for a short time each weekday morning and afternoon many urban freeway sections offer a poor level of service to the motorists.

Although control of freeway traffic is, in itself, an anomaly, it has become increasingly apparent that some regulation or control of the traffic during such critical periods is necessary. Investigations are being made of the effect of metering or restricting input to freeways, and speed advisory signs for the traffic on the freeways are being used and evaluated.

Whatever the control action may be, there is a need for practical, reliable, and efficient information which will actuate or initiate the control measure or measures. Control systems will consist of an input sensor component which will supply the necessary information, a logic component which will translate input information into a course of action, and a control component which will enforce the chosen course of action. An iterative series of the foregoing phases will continuously sample, decide, and act throughout a period when control may be necessary.

Surveillance systems combine the first and part of the second components of a control system. These systems can be thought of as preludes to control systems. A television surveillance system uses television cameras and pictures as the sensor component and human beings as the logic component. Traffic stream element detector systems are also used as surveillance devices. Electronic vehicle and speed detectors are used in typical element detector systems as the sensing components, and analog electrical circuitry is used as a part of the logic component.

Although traffic stream element surveillance systems have the obvious limitation of not showing all of the traffic situation, they can be better adapted to an automatic control system. Until the present time, only the time-based elements of the traffic stream have been utilized, or sensed, by these element systems, i.e., volume (veh/hr) and/or speed (mph). It is possible with some of the systems to measure the percent occupancy which is related to density but is a point-obtained value and must be based on a time interval.

In the general traffic stream equation, $q = kv$, q is the flow (or volume) in vehicles per unit of time, v is the space-mean-speed of the vehicles in the traffic stream in distance per unit of time, and k is the concentration (or density) of vehicles in a length of roadway in vehicles per unit of length. If any two of these three traffic stream elements are known, the third is uniquely determined. Density, or concentration, has generally been considered the dependent element because the other two elements have been the measured elements. There is, however, no single dependent element but only a relationship between the elements. It is helpful in visualizing the basic traffic stream equation to consider the surface representing the equation plotted on mutually perpendicular axes (Fig. 1). The locus of all possible points is a surface which is infinite in extent; however, there are practical limitations which have been rather well established by many previous studies.

Congestion is a qualitative term which is used in traffic engineering to indicate a condition of traffic and traffic movement. Density is the quantitative measure of con-

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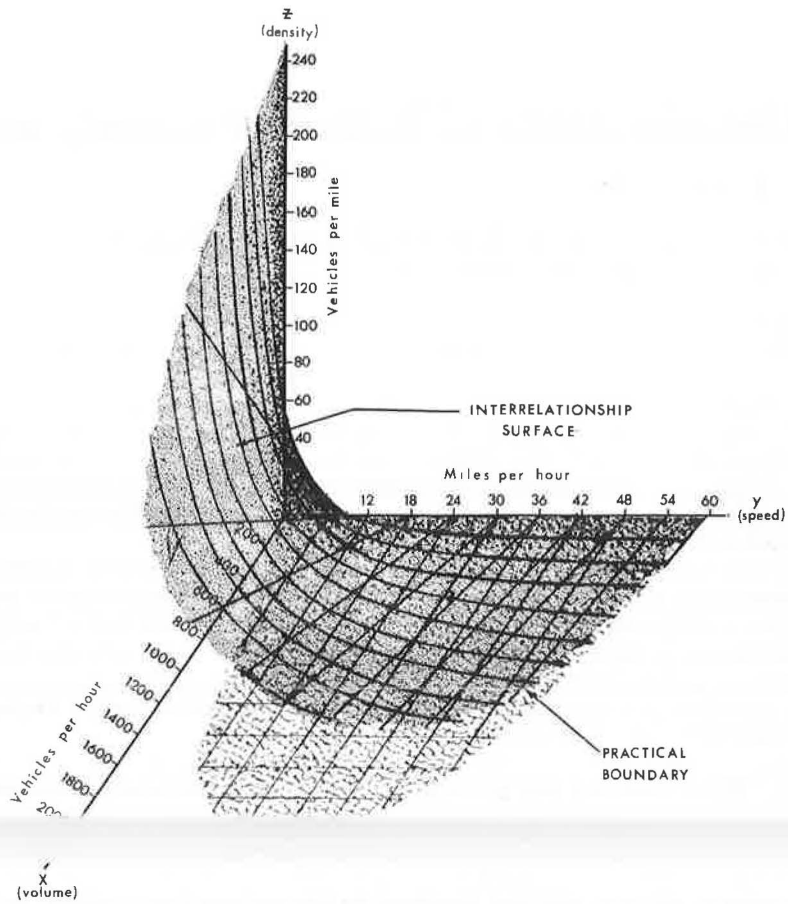

Figure 1. The speed-volume-density surface.

gestion and, thus, should be the most desirable element to use in freeway operation control. High volumes of traffic or high average speeds are not objectionable from an operational standpoint. Actually, high volumes and speeds are desirable in themselves, but it is known that sustained high volumes can lead to lower speeds and, hence, high densities or concentrations of vehicles on the roadway, which are undesirable. Unfortunately, continuous densities have not been directly measurable. Volumes and speeds have been measured for many years by a variety of means.

Some of the factors influencing the interrelationship of density, volume, and speed are the methods of measuring each. Density is, by its very nature, a space element of the traffic stream; volume is a time-point (nonspatial) element; speeds are sometimes point elements (spot speeds or instantaneous speeds) or are sometimes based on travel times over a finite, short distance (space-mean-speeds). Sensing devices have been used to determine speeds and volumes at a point (1) (or over a very limited length of roadway), and densities have been rapidly approximated at short time intervals, by electronic means, on the basis of such point information. This process, in effect, extrapolates speed and volume information obtained at a point to density over a distance of up to 1 mi. Density fluctuates continuously and becomes critically high in certain spaces on a freeway in connection with the creation of bottlenecks. Investigations by Keese, Pinnell, and McCasland (2) have shown that traffic in the near vicinity of entrance ramps becomes congested enough to reduce speeds as much as 50 percent or more during regular peak flow periods. From the fundamental relationship, volume is equal to density times speed, it is obvious that if a given volume of vehicles slows down, the density must increase, resulting in more congestion.

It would be desirable to sense density directly over a given length of roadway. From a control standpoint, it is hypothesized that density sensing offers greater promise than the current methods of computing density on the basis of speed and volume information.

If by some satisfactory means density were sensed, there would remain the problem of determining the proper locations of sensors, the required lengths of roadway to be sensed for density, and the critical density values to be used for the controlled operation of freeway traffic. The characteristics of density must be carefully studied by themselves before density is used as a control element. Because density has been the dependent or calculated element heretofore, little has been developed which would enable the study of the basic nature of density.

SCOPE

This report includes parts of a general study of the various aspects of vehicular density for use in the control of freeway traffic (3). Density is considered singly as a possible control element of a freeway operational system. This study provides information which may be useful for freeway control methods.

The scope of this report specifically involves principal study areas described as follows:

1. The principal features of existing methods used to measure or estimate density are reviewed. There are two basic methods involved. One is a process in which density is estimated on the basis of speeds and volumes sensed at a point. The other method, which is not yet operational, involves the actual measurement of the density, or concentration of vehicles in a space. The undesirable features and limitations of existing equipment are listed, and the general features which are desirable in density sensors are described.
2. Results of aerial photography studies of the Gulf Freeway in Houston, Texas, are utilized to show how density may be related to volume as well as to certain geometric features of the freeway facility.
3. A field study method is described which yields continuous values of vehicular concentration on certain sections of a freeway. The results of several of these density-trap studies are analyzed for the purpose of determining the variability and frequency distributions of freeway concentration and relating the length of sensing sections to the variability of concentration.
4. The analysis of the data demonstrates a means of establishing optimum, or critical, freeway concentration values and provides a means of identifying critical, or bottleneck, sections at a freeway which exhibit recurring high densities.

DEFINITION OF TERMS

Bottleneck—a section which has a smaller capacity for accommodating vehicles than adjacent sections upstream or downstream.

Concentration—accumulation, or number, of vehicles within a section or roadway less than 1 mi long.

Congested Operation—operation at densities, or concentrations, greater than critical density.

Critical Density—density at which maximum flow rate, or volume, occurs.

Density—number of vehicles within a 1-mi length of roadway.

Density Trap—a section of roadway for which the input and output volumes are measured synchronously.

Light-Volume Operation—uncongested traffic operation involving volumes less than maximum and densities less than critical.

Rate of Flow—number of vehicles passing a given point on a roadway in a period of time less than 1 hr.

Section Length—length of roadway under consideration for vehicular concentration.
Sensing—automatic detection of some aspect of traffic flow.
Space-Mean-Speed—mean, or average, speed of vehicles within a given space of roadway.
Time-Mean-Speed—mean, or average, speed of the vehicles passing a given point during some period of time.
Volume—number of vehicles passing a given point on a roadway in a 1-hr period.

DENSITY SENSING EQUIPMENT

There are presently several different firms which supply electronic density computers. Actually, the systems available at the present time are capable of measuring both speeds and volumes, and density is computed on the basis of these values.

The computers utilized in density computations usually generate analog functions which may be recorded as analog data, displayed on output meters, or used in connection with control systems. The input information necessary for density computation by a computer requires time to accumulate. The passage of a number of vehicles or the time passage of several seconds or minutes is necessary for the traffic stream to generate data to be evaluated.

The electronic circuitry involved in existing traffic surveillance analog computers is beyond the scope of this study. It is possible, according to some of the manufacturers, to modify or adjust the output of such systems for a wide variety of purposes. A preselected time period has been mentioned as a necessary constant for computing volumes and average speeds. In many instances, manufacturers will offer several different time intervals which may be used.

An altogether different type of density sensing is being developed in England by the Road Research Laboratories. Charlesworth, Head of the Traffic Section of the laboratories, replied to this author's inquiry about the actual sensing of vehicles in a space, rather than computing the concentration on the basis of speed and volume. In the reply, it was stated that there had been a development of a system which "meas-

stated that the detector produced an output dependent on the number of vehicles adjacent to its detector loop and has been used to determine when the "level of traffic" in a traffic circle exceeded a critical value. It was further stated that this instrument would be commercially available in the "near future."

The detector loop referred to in connection with the instrumentation is a known and utilized device in this country. Vehicle presence detectors which operate with loops installed just below the pavement surface are used at tollgates, traffic signals, etc.

These vehicle detectors, however, are used only to detect the presence or absence of a single vehicle adjacent to (just above) the loop. The output is a two-step function only, indicating that an event is or is not occurring. The development of the loop detectors to indicate the number of vehicles adjacent to a large loop (1,000 ft or more in perimeter) would involve an analog function output which would be indicative of the number of vehicles. In this sense, the Road Research Laboratory is developing a true density (or concentration) sensor rather than a density computer.

Present Limitations

The density (or occupancy) computers now available involve the limitations of point sensing and time lag which are not actually limitations of the density element but are imposed by the means now utilized in the computer-sensor scheme. As stated earlier, density is not presently sensed by available equipment; it is computed on the basis of volume and speeds, both of which are time-dependent elements of the traffic stream and must be measured at a point on a facility. The information obtained at a point is then extended such that, assuming unchanging conditions exist downstream from the point, a density or concentration of vehicles is estimated on the basis of a required, preselected time interval.

Density is a function of vehicles and roadway length only; time is not a dimension of this element. Density exists continuously at all instants in time. It is the one element

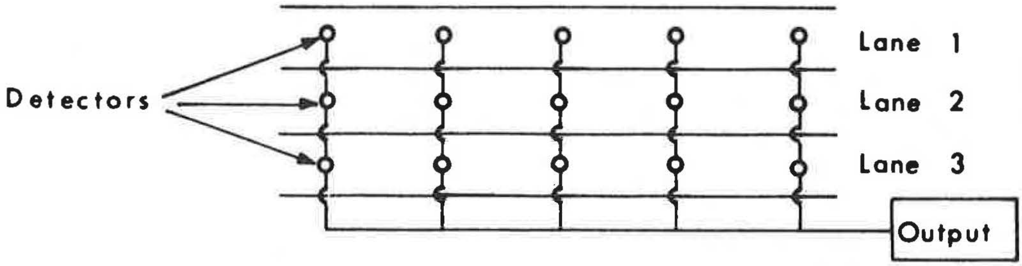

Figure 2. Desirable summation gridwork of sensors.

which can be obtained at any instant without counting for a predetermined time interval and, thus, would be more readily available for control decisions.

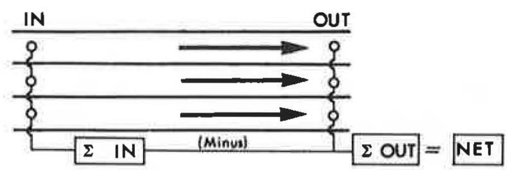

Figure 3. Undesirable scheme utilizing difference of counting sums.

Conditions do change from section to section along a freeway and conditions at one point do not accurately predict or represent conditions at all other adjacent points. Density is more nearly related to congestion and it should be sensed directly in order to measure congestion accurately. The devices now in use only predict, with limiting assumptions, what the density might be downstream, and only then after an arbitrary time period of counting. It would seem that improved methods are forthcoming and that true density sensing will be done in the future.

Some Desirable Features

Economy is a feature to be desired in a density sensing device. Methods are now possible which are too costly to consider. To utilize density as an element of control information, it must be economically feasible to sense it. A determination of the economic justification of density sensing or the benefit-cost ratio of traffic surveillance systems is perhaps difficult at the present time. It is presumed that in the developmental stages, some research and traffic surveying systems are not immediately economical but may result in long-term savings in terms of lives, dollars, time, and information. It remains, however, to develop an economical means of sensing density.

The systems developed should, of course, be density sensors (or concentration sensors) and not concentration computers or estimators. Such systems should be highly reliable. As in any automated device, it is desirable to incorporate "fail Safe" features or positive indications when the system is out of order.

The scheme of operation of a density sensor should be such that the failure of one component or sub-part will not create an accumulative error in the output of the system. Schemes involving a continuous counting routine are typical of those which accumulate an error if one counting element fails. More specifically, it is more desirable to feed the continuous output of a gridwork of vehicle detectors into one total output, such as that shown in Figure 2, than to deduct periodically the out-detector sums from the in-detector sums in an arrangement similar to that shown in Figure 3. The failure of one sensor in the gridwork scheme would cause a small nonaccumulative error in the output, whereas the failure of one sensor in the scheme of counting shown in Figure 3 would result in an ever-increasing error. An additional feature of the desirable system shown in Figure 2 is that it requires no starting technique. It would yield continuously, from the time the sensors were activated, the concentration of

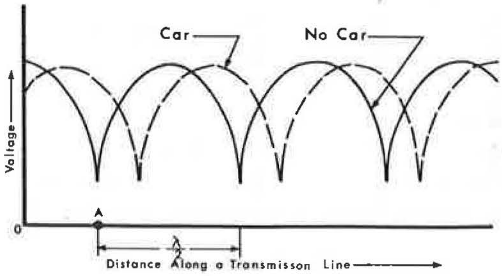

Figure 4. Standing wave variation along transmission line.

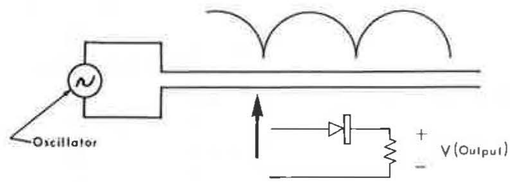

Figure 5. Transmission line circuitry scheme.

vehicles within the section. The undesirable scheme shown in Figure 3 requires some special starting technique for the determination of the number of vehicles in the section at the time the counting begins.

It is highly desirable that the density sensors be unaffected by the speed of the vehicle. Some vehicle detector loops and radar detectors will not operate if the vehicular speeds are less than 5 mph. Detector loops may cease to detect the presence of a stopped vehicle after a short period of time. The system should sense vehicles at high speeds as well as those which have stopped.

If vehicles could be sensed according to lanes, the system would have more utility. It might also be desirable to let the influence of large trucks be represented in the output of the system.

A system should be durable and easily installed and maintained. The utility of a density sensing device would be increased if a wide range of roadway lengths could be sensed.

In a very limited experimentation program undertaken at Arlington State College, certain studies concerning the development of density sensors were made. Attempts were made to utilize low-frequency electromagnetics which depend on an electromag-

properties of the low-frequency field resulted in unreliable detections because the flux was not controlled, or confined, to a desired region. The use of magnetometers was considered; however, a review of the cost of such a system revealed that this method would be too expensive to investigate in this study. The method of sensing impedance shifts of a transmission line placed along the centerline of a lane was investigated and was promising in a revised application.

A system which appears to be feasible because of its economy, reliability, and simple circuitry is one in which equally spaced, short transmission lines are placed at right angles to the centerline of a lane. An oscillator supplies energy to the transmission line and a standing wave pattern is set up adjacent to the line. Figure 4 illustrates the relationship between the voltage and the distance along the line.

In the absence of any vehicle adjacent to the transmission line, a voltage detector fixed at point A (Fig. 4) would read some small minimum voltage. The presence of a vehicle near the line causes a shift in the standing wave pattern which would result in an increase in voltage at point A. The direction of the wave shift is of no concern because the wave pattern is symmetrical and a small shift in either direction would result in the same voltage increase.

A schematic diagram of a transmission line circuit is shown in Figure 5. The line itself is comprised of simply two ordinary parallel wires. An oscillator is used which operates in the 100 megacycles per second range and is rather inexpensive, costing about $20. The rectifier serves as an envelope detector and the output voltage is largely d. c.

Multiple detectors may be incorporated into a sensing system similar to that shown in Figure 2. The voltage outputs of each detector system can be combined to operate a voltmeter which could be calibrated to read in vehicles rather than volts.

AERIAL PHOTOGRAPHY STUDY METHODS

The Houston Aerial Photography Study

In September 1962, aerial photography studies were made of a 5-mi section on the Gulf Freeway (US 75) which extended from the edge of the central business district in Houston southeast to the Reveille Interchange, where Texas 225 and 36 intersect the freeway. Two methods of aerial photography, time-lapse and continuous-strip, were incorporated in these studies under the direction of the Texas Transportation Institute,

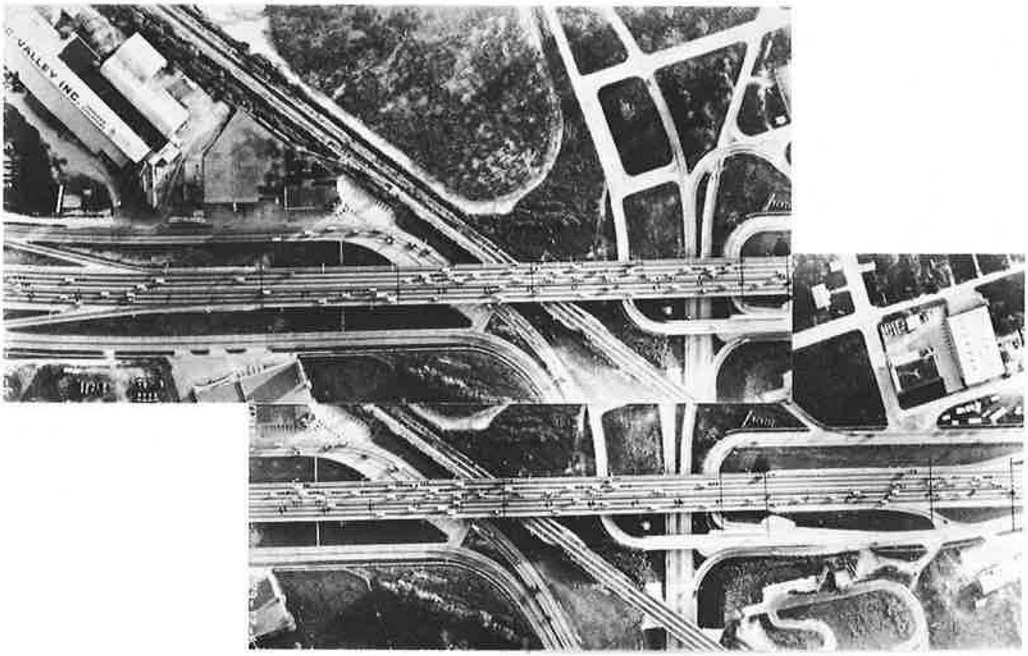

Figure 6. Typical time-lapse photographs.

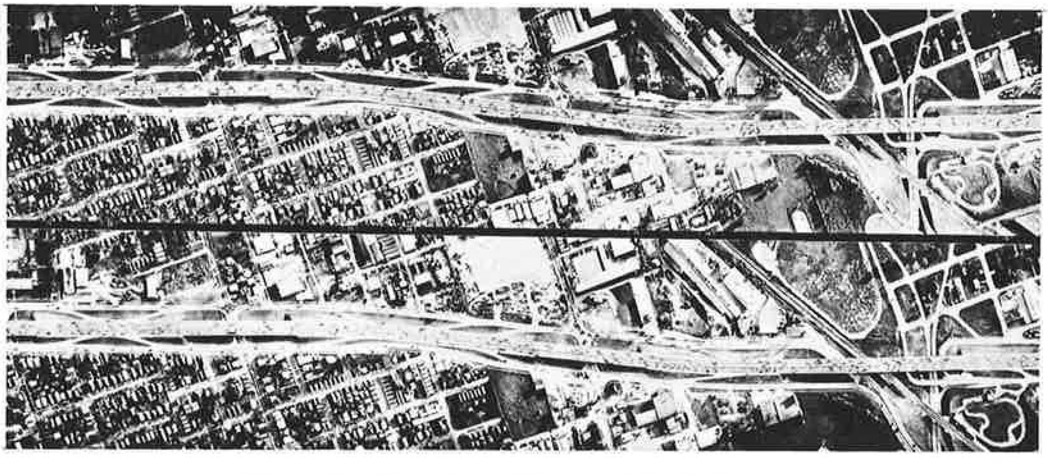

Figure 7. Typical continuous-strip photograph.

in cooperation with the Texas Highway Department and the U. S. Bureau of Public Roads.

There were two principal objectives of the aerial photography study of the Gulf Freeway. The two aerial photographic methods were to be compared for their applicability to aerial traffic surveys; also, considerable information concerning the operational characteristics of the freeway was expected. The work was contracted to two aerial photography firms, each utilizing a Cessna 195 fixed-wing aircraft. Each company began flights at about 6:30 a. m. and continued until about 8:00 a. m. The two planes were required to be separated by at least a 2-min interval. The time-lapse plane was required to make nine runs and the strip-film plane was required to make as many runs as possible and, to repeat its schedule of runs from 6:30 a. m. until 8:00 a. m. on a morning as soon thereafter as practicable. This arrangement resulted in the nine time-lapse runs and a total of 22 continuous-strip film runs. The planes were requested to fly only one outbound run on the first filming day and the strip-film plane was requested to repeat this procedure on its second filming day.

During the filming runs, ground observers made volume counts at several points along the 5-mi section of freeway. Several control vehicles were specially marked on the roof and made runs in and out continuously during the filming sequences for which they recorded their travel times. No communication between the ground stations and the airplanes was provided for. There was a synchronization of watches ½ hr before the start of the filming flights.

Each of the 22 strip-film runs was developed and furnished as positive film transparencies at a scale of about 1 in. to 300 ft. The nine time-lapse film runs were developed and printed at a scale of 1 in. to 100 ft. Figures 6 and 7 illustrate, to a reduced scale, these furnished photographic types.

Data Reduction

McCasland (4), in reporting on the data reduction techniques used on the Houston aerial photography study, emphasized that it was decided that each method and each run would be subjected to as complete a reduction of data as possible. Had it been desired only to analyze the films for a particular characteristic, such as density, the [illegible] completed in the spring of [illegible] and was in a general form on punched cards which could be utilized for many different types of analysis.

Density-Volume Relationship

Since it is the volume which ideally should be kept as high as possible, the relationship between volume and density should be well established if density is to be used as a control element. Each freeway may exhibit a different characteristic volume-density relationship. It is furthermore likely that different sections along a freeway will have their particular volume-density characteristics. Factors which can influence this relationship are not only the geometric features of the freeway such as lane widths, grades, median widths, shoulder widths, curvature, sight distance, and entrance and exit ramps, but also such features as the posted speed limits, size of the metropolitan area, and distance from the central business district.

Any specific section of a particular freeway facility can be studied for the volume-density relationship. The continuous-strip aerial photography described earlier was utilized to make a study of this type. A test section was selected such that no entrance or exit was possible within the section, but all vehicles entering at one end exited at the other end or stopped within the section.

The number of vehicles within the test section was obtained for each of the 22 different flights. The speeds of the vehicles were averaged (space-mean-speeds) and the volumes were then computed on the basis of the density and the speeds. Table 1 gives the results of the 22 runs.

The 22 points obtained in this study are plotted in Figure 8, which shows the volume-density relationship for this particular study section. A curvilinear regression, using a second degree parabolic relationship, yielded the equation of best fit:

TABLE 1

CONTINUOUS-STRIP AERIAL PHOTOGRAPHY DENSITY STUDY TABULATION

Run	Vehicular Concentration (per 2,000 ft)				Density	Space-Mean-Speed	Vol.
	Lane 1	Lane 2	Lane 3	Total			
1	4	5	8	17	44.9	53.6	2,407
2	7	10	9	26	68.6	38.3	2,627
3	9	11	6	26	68.6	51.5	3,533
4	8	10	11	29	76.6	50.5	3,868
5	14	22	18	54	142.6	46.9	6,688
6	13	18	14	45	118.8	43.3	5,144
7	11	19	18	48	126.7	43.0	5,448
8	15	24	17	56	147.8	41.7	6,163
9	14	21	20	55	145.2	38.6	5,605
10	12	14	15	41	108.2	45.1	4,880
11	9	11	15	35	92.4	49.1	4,537
12	8	14	12	34	89.8	48.7	4,373
13	9	14	10	33	87.1	50.2	4,372
14	9	14	11	34	89.8	52.9	4,750
15	10	15	13	38	100.3	47.1	4,724
16	15	20	17	52	137.3	43.1	5,918
17	14	21	23	58	153.1	33.2	5,083
18	12	20	15	47	124.1	40.2	4,989
19	13	14	15	42	110.9	39.8	4,414
20	16	18	19	53	139.9	40.8	5,708
21	15	20	19	54	142.6	43.2	6,160
22	15	19	18	52	137.3	40.9	5,616

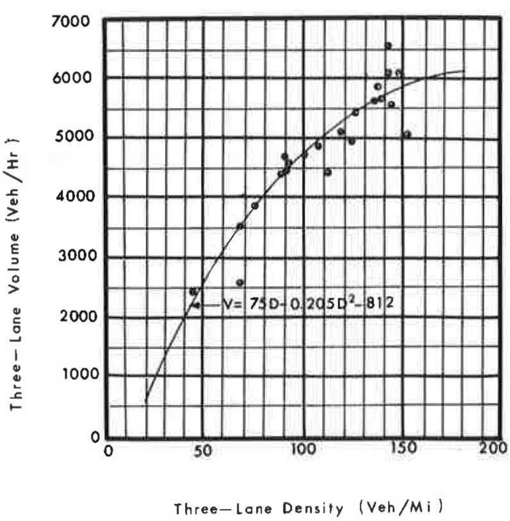

Figure 8. Volume-density relationship obtained in continuous-strip aerial photography.

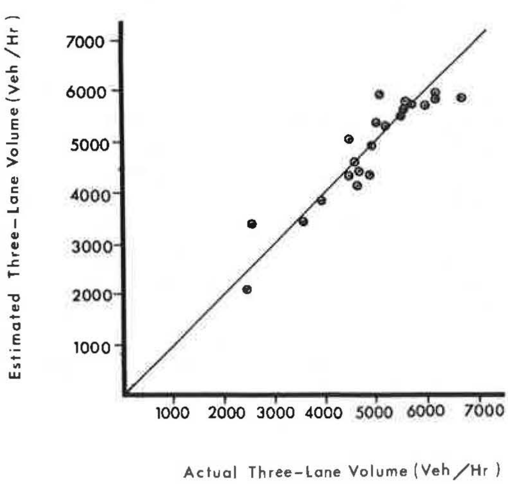

Figure 9. Estimated vs actual volumes obtained in continuous-strip aerial photography study.

$$V = 75\,D - 0.205D^2 - 812 \qquad (1)$$

in which V is three-lane volume (veh/hr) and D is three-lane density (veh/mi). This regression analysis placed no restrictions on the constant term; thus, when the density in this equation is zero, the volume is -812. The true relationship is such that volume is zero when density is zero. The equation, however, is the best second degree curve fit for the points shown, all of which are in the range of 2,500 to 6,500 veh/hr.

The volumes as predicted by Eq. 1 were computed for each of the 22 densities obtained in this study and plotted vs the volumes obtained in the study (Fig. 9). The coefficient of correlation for these points is 0.906 and R-square is 0.82, or it could be said that the second degree equation accounts for about 82 percent of the variation in actual volume.

If the parabolic expression is differentiated with respect to density, $dV/dD = 75 - 0.41D$ and $dV/dD = 0$, then $D = 75/0.41 = 183$, which is the density associated with the maximum volume. The maximum volume would then be $V = 75(183) - 0.205(183)^2 - 812 = 13{,}725 - 6{,}865 - 812 = 6{,}048$ veh/hr for three lanes.

DENSITY TRAP STUDY METHOD

Because of the expense of aerial photography methods of studying freeway traffic flow characteristics and because only a few instantaneous states of density were available from such studies, another study procedure had to be developed to provide more, and preferably continuous, density information. Economy and mobility were important factors in the development of this procedure. It is emphasized that the study method developed was not basically a method to be developed for automatic density sensing; some desirable features of automatic density sensing have already been discussed. This procedure involved a means of obtaining a continuous record from which actual densities, or more correctly, vehicular concentrations within a section of the freeway were derived after a data reduction of the record.

The density trap involves a selected length or section of roadway. The upstream end of the lanes of one-direction traffic constitutes the beginning of the trap or the in

counted in and out of the section or trap. The simplest case is that involving no entrance or exit points within the trap. The number of vehicles, k, in the trap at any time, t, can be expressed as:

$$k = k_o + \sum_{T=t_o}^{t} I(T) - \sum_{T=t_o}^{t} O(T) \qquad (2)$$

where k_o is the number of vehicles in the trap at some beginning time t_o, I(T) is the number of vehicles passing in as a function of time, and O(T) is the number of vehicles passing out as a function of time.

Several practical problems exist when attempts are made to put this principle into actual use: (a) the vehicles must be counted very accurately because any error made in counting will be thereafter reflected in the concentration, k; (b) since the number of vehicles passing in and out have the same time base, there should be no error in synchronization of the beginning time or lapsed time after beginning; (c) the number of vehicles, K_o in the trap at a beginning time, t_o, is not readily determinable; and (d) the record of such values as k, I(T), and O(T) is continuously changing in time and requires some dependable recording scheme.

Description of Method

As a matter of expediency, it was decided to count the vehicles manually. Adequate numbers of personnel were available with experience in counting traffic, and it was felt that automatic vehicle counters were not economically justified. The installation of the counters would have also been time consuming and they would not have had the mobility of human observers. It was realized that a method of checking would have to be developed, regardless of whether automatic or manual counters were used.

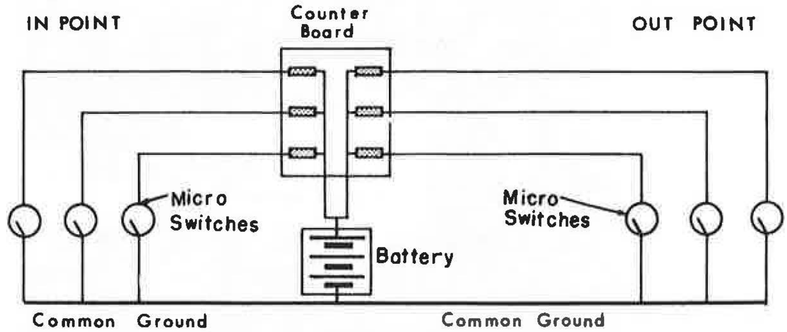

Figure 10. Wiring scheme for density-trap apparatus.

To obtain a common time base and to synchronize properly all in and out counts, it was decided that each observer, or traffic counter, would operate an electrical switch wired to an electrical counter mounted on a centrally located counter board. All counters for both the in and out points were mounted on the same counter board, thus could be observed simultaneously at any instant in time. Figure 10 shows a wiring scheme for the density-trap method.

To ascertain accurately the number of vehicles, K_0, in the trap at some beginning time, t_0, a span of pace cars were required to maneuver such that they were abreast of each other when traveling through the trap, thus preventing any vehicles from passing them or their passing any vehicles.

The pace cars were the first cars counted by every observer; therefore, when the out points began counting, the difference between the total in and the total out was the actual concentration of vehicles within the trap.

A check on the accuracy of the counting was provided by having the pace cars pass through the section a second time to end the count. The pace cars passed through the section abreast and were the last vehicles counted by every observer. The final totals of the vehicles in and the vehicles out were identical if no mistakes had been made in counting.

The data were recorded by photographing the counter board with a 16-mm moving picture camera at a rate of 10 frames per second. This system provided a continuous record of the density-trap studies, each of which lasted from about 6 to 18 min. The reaction time of the observers was found to vary about 0.2 sec. To determine this variation, several observers were required to actuate their counting switches for the same vehicles passing a given point. This variation in counting could have been overcome by using automatic vehicle counters; however, for the purposes of this study of vehicular densities, manual counting was considered sufficiently accurate. The camera speed of 10 frames per second was deemed sufficiently fast, consistent with the variation in manual counting accuracy.

The counters were mounted flush with a small plywood board which comprised the face of a folding-leg frame. An electric clock from an automobile was mounted on the face of the panel and was operated by a 12-volt dry-cell battery. The clock provided a record of the time of day and, by observing the second hand, an accurate check was made of the number of frames per second taken by the moving picture camera. The camera was equipped with a 1,200-ft film magazine and an electric drive. The power for the camera drive motor was provided by a portable, gasoline-powered generator. Figure 11 shows the camera in position for filming the counter board during a study at the Gulf Freeway in Houston.

Radio communication between the pace car drivers, the filming station, and the counter stations was provided. Ideally, there would be a total of four transceivers for such a study: one in a pace car, one at each counting station, and one at the filming station. Each transmitter should have a range of several miles to insure communication

Figure 11. Filming station for density-trap study.

Figure 12. Manual count station at edge of freeway.

Figure 13. Manual count station adjacent to bridge.

with the pace car which might be required to travel considerable distances from the trap section. Field telephones could be utilized between the ground stations and, thus, a minimum of two radios would suffice. Figure 12 shows a group of three observers and a supervisor standing in position just off the shoulder of the freeway. In some instances, it was necessary to make counts at a location on a bridge where the sidewalk was too narrow to accommodate the observers safely. In such cases, use was made of a hydraulic lift platform, as shown in Figure 13. The Texas Highway Department and some city traffic departments utilize such trucks in the maintenance of signs and traffic signals.

The moving picture films, after being developed, were analyzed on a 16-mm time-motion projector. These projectors are equipped with frame counters and can be advanced so that the film can be studied frame by frame. In the data reduction of the film, it was necessary to record each counter actuation and the associated frame number. Clock times were recorded every 1,000 frames, or approximately every 100 sec, to confirm the filming speed. The counters rotated half way toward the next digit on contact closure and completed the rotation when the contact was broken. Attention was given to recording the precise frame number of the initial contact closure.

An added feature of the density-trap method was incorporated during the studies made in early 1964 in order to obtain the travel times of several vehicles through the trap section. Two additional counters were used for this purpose and were also mounted on the counter board. Additional circuits with switches were provided similar to the counting circuits for each of the two additional counters. The observers using these counters were stationed at each end of the trap section near the volume count observers and were provided with communication with one another. Either field telephones or portable radios could be used for this purpose. The observer stationed at the in point selected a "floating" vehicle and actuated his switch when the vehicle entered the trap section. He then described the vehicle to the other observer at the out point, who, in turn, actuated his counter when the described vehicle exited the trap. The filmed record of these two counters provided sufficient information to calculate the speeds of these vehicles through the trap.

It should be pointed out that the trip times through the trap were not necessary, but were obtained as a possible source of verification of speeds calculated from the volume and density information obtained in the study.

A total of six density-trap study runs were made in Houston on Wednesday morning, Nov. 27, 1963. The study site was selected on the Gulf Freeway in the region of Station No. 100. The inbound lanes of the facility have no entrance or exit ramps for a distance of about 1,800 ft at this particular location (Figs. 6 and 7). The out count station was set up at Station 86 + 50 and the in count stations were set up at three different points to vary the trap length. In point stations at 92 + 30, 95 + 40, and 102 + 80 provided trap lengths of 580, 890, and 1,630 ft, respectively. Personnel from the Texas Transportation Institute and the Texas Highway Department were utilized in making the Houston studies. The run numbers and lengths of trap sections and the difference in the final in and out counts obtained are given in Table 2. Only runs 1, 4, and 5, which tallied exactly, were reduced frame by frame for complete analysis. The film reduction, frame by frame, required two persons about 42 hr of moving projector time and about 84 man-hours.

No serious congestion occurred during any of the six study runs. Information concerning volumes of traffic associated with high densities was desired.

A new study site for congested conditions was sought which would provide vantage points for observers from which they could make accurate counts without calling attention to themselves. The North Central Expressway in Dallas is a partly depressed facility; that is, it passes under many of the major streets. The embankments in the regions of overpassing major streets or the overpassing structures themselves provide positions from which observers can accurately count traffic from less obvious positions. At the Haskell St. overpass, the North Central Expressway provides a section about 1,200 ft long with no entrance or exit ramps. This section of expressway was observed to congest rather heavily each day on the outbound lanes during the afternoon peak. On Friday afternoon, Feb. 7, 1964, personnel from the Texas Transportation Institute

TABLE 2

SUMMARY OF DENSITY-TRAP STUDY METHOD RUNS IN HOUSTON

Run No.	Time (a. m.)	Run Duration (min)	Trap Length (ft)	Total Veh In	Total Veh Out	Count Error
1	7:40	15	580	1,268	1,268	0
2	8:30	12	890	749	746	3
3	9:15	12	1,630	476	477	1
4	10:45	12	1,630	499	499	0
5	11:10	11	890	466	466	0
6	11:40	11	580	419	421	2

and Arlington State College made a study at the Haskell St. location on the North Central Expressway. The study involved two runs, hereafter referred to as runs 7 and 8. Run 8 involved a difference of 9 vehicles in the in and out count and was not analyzed frame by frame. Run 7 was made at about 5:00 p.m. and had a duration of 6 min. The trap length was 900 ft and the total vehicles counted was 459, at both the in and out stations.

ANALYSIS OF DENSITY DATA

Density Data Concept

Volume, when reduced to its most elemental form based on time gaps between successive vehicles, is quite widely variable. Of course, when longer periods of

might be represented as shown in Figure 14. Although the shortest term volume rates in and out may be quite variable, the density of vehicles within the section will be less variable if there is an average of over two vehicles in the trap. The volume out is not independent of the volume in. If an average travel time through the trap were Δt, there would be similarity between the two volumes if the origin of the volume were shifted an amount equal to Δt. As the trap length increases, this similarity between volumes tends to diminish. The longer the trap length, the less variable the density will be. This is comparable to a longer counting period for volumes entering (and leaving).

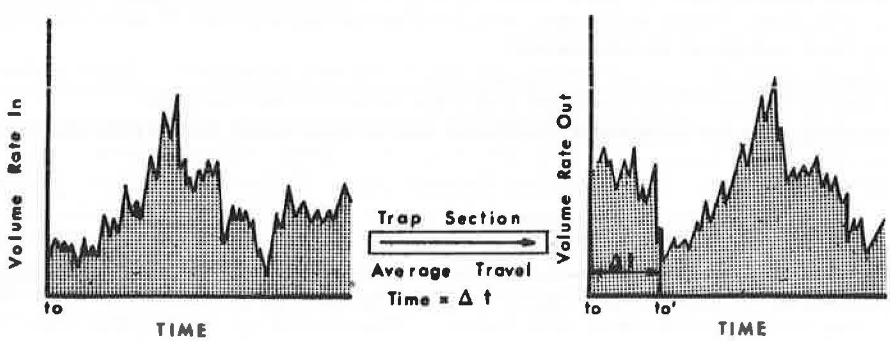

Figure 14. Volume rates in and out of density trap.

TABLE 3

DENSITY-TRAP STUDY METHOD TRAVEL TIME AND SPACE-MEAN-SPEED TABULATION

Run No.	Avg. Travel Time (hr)	Avg. Speed (mph)
1	0.00300	36.6
4	0.00680	45.4
5	0.00357	47.2
7	0.01070	15.9

Density-Volume Relationship

The density-trap study method provided considerable data suitable for establishing the relationship between density and volume. The approach in this analysis involved the assumption that a short-term rate of flow in immediately preceding any specific concentration should be compared to that concentration. Furthermore, the short-term rate of flow out immediately succeeding any particular concentration was compared to that concentration. The basis for this concept is illustrated in Figure 14. The time involved in the rates of flow was based on the approximate average travel time through the trap. The average travel times through the trap were either measured, as described earlier, or obtained from the time required for the pace cards to traverse the trap at the beginning and end of any run. Since a rate of flow was being computed which involved dividing the number of vehicles observed during the time period by that time period, the precise time interval to be used was not of absolute importance as long as the time interval was generally about the average travel time for the trap. The time intervals used for the various runs analyzed are given in Table 3.

A computer program was written to determine the rates of flow in and out for each concentration obtained in runs 1, 4, 5, and 7 and the rates of flow were extended to volumes, in vehicles per hour, and the concentrations were expanded to density, in vehicles per mile. A curvilinear regression involving a second degree parabola resulted in the equation

$$V = 65.5D - 0.179D^2 - 80 \tag{3}$$

Only every twentieth value of density and the corresponding value of volume is shown in Figure 15. The large number of obtained points renders plotting every point impractical.

When Eq. 3 is solved for the maximum volume, by differentiating and setting $dV/dD = 0$, the maximum volume is 5,930 veh/hr for three lanes and the optimum density associated with this maximum volume is 183 veh/mi for three lanes.

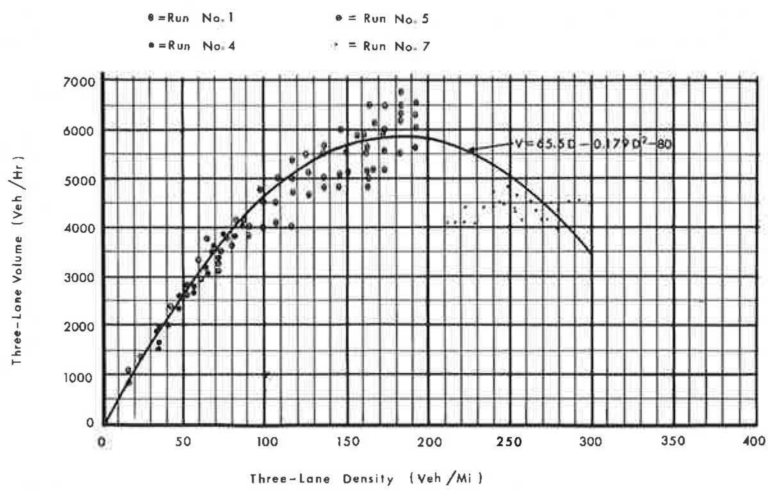

Figure 15. Volume-density relationship obtained from density-trap studies.

The differential of this equation, $dV/dD = 65.5 - 0.358\,D$, represents the slope of the volume-density curve at any point and when density is zero, the slope is 65.5. The slope at zero density is what might be termed the "free-flowing" speed of the section of freeway since the slope is volume, in miles per hour, divided by density, in vehicles per mile, which is equal to the space-mean-speed in miles per hour. To be completely correct, the curve should pass through the origin; however, this curve comes considerably closer to the origin than the one developed in the continuous-strip aerial photography study.

The true shape of the volume-density curve past the maximum volume point is difficult to ascertain. The volumes associated with extremely congested traffic conditions are known to be quite small and the density reaches its maximum value when the stream of vehicles has been halted in a bumper-to-bumper stoppage. For the purposes of this study, it is not necessary to determine this branch of the curve; the maximum volume and its associated density is the limit of importance in using density as an element of control. It is believed that the density-trap method, however, might be a useful study procedure for determining the congested, or right, branch of this curve.

Variability of Concentration

Although density is, by current usage, defined as the number of vehicles in a 1-mi length of roadway, concentration is taken to mean the number of vehicles in a length of roadway less than 1 mi long. The two important factors affecting the variability of concentration are the length of section involved and the volume of traffic.

A simulation of freeway traffic with an IBM 709 digital computer (3) indicated that distributions of concentrations could be approximated rather closely by the Poisson distribution, particularly for light to medium volumes of traffic. This simulation indicated that the mean concentration was directly proportional to the length of section involved for any given volume. The simulation furthermore indicated that the variability of concentration was inversely proportional to the length of section involved for any given volume. Specifically, the standard deviation of concentrations was related

$$\sigma_1 = \sqrt{\frac{L_1}{L_2}}\,\sigma_2 \tag{4}$$

where σ_1 is the standard deviation of the concentrations observed in a length of section L_1, and σ_2 is the standard deviation of the concentrations observed in a length of section L_2. The lengths of sections investigated ranged from 500 to 2,000 ft.

The standard deviations were observed in the simulation program to be a function of volume; however, for a section length of 1,000 ft, σ ranged between 3 and 4 veh for all volumes. The actual validity of these simulated variations was established rather well by the field studies.

The frequency distributions of concentrations obtained from the density-trap field studies were calculated on a time basis. The frequency, in other words, refers to that portion of the time that a particular concentration was observed. Figure 16 shows the frequency distributions for runs 1, 4, 5, and 7, and the Poisson distribution with the same mean value is superimposed on each plot. Runs 4 and 5, as previously stated, involved moderate to light volumes of traffic and it is seen that the concentration distributions are quite similar to Poisson distributions. The standard deviations of these two runs are approximately equal to the square root of the mean, which would be the case for a Poisson distribution. Run 1 involved high volume flow, or generally optimum volume of flow. The distribution of density for this run is observed to be significantly less variable than the corresponding Poisson distribution. Run 7 involved congested flow conditions with volumes less than optimum. The standard deviation obtained in this run was considerably less than would have been obtained with a Poisson distribution.

It should be pointed out that there was no control of volume in these field studies and only the lengths of traps were varied. The durations of the runs were compara-

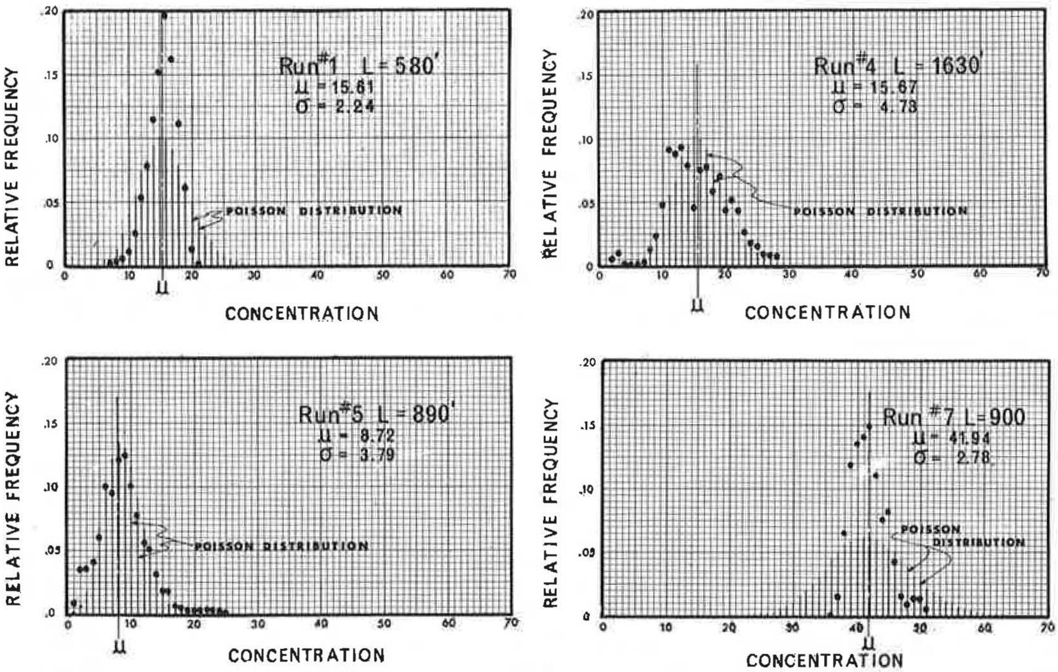

Figure 16. Concentration distributions obtained from density-trap studies.

TABLE 4

STANDARD DEVIATIONS OBTAINED IN DENSITY-TRAP STUDIES

Run No.	σ_m	$\sigma_{1,000}$	Approx. Vol.
1	2.24	2.94	5,050
4	4.73	3.71	2,550
5	3.79	3.94	2,650
7	2.78	2.88	4,300

tively short, however, and it is unlikely that a general change in the volume occured during any specific run. The lengths of the traps had an obvious effect on the mean value of concentration for any similar volume conditions. For example, the mean concentration value, considering moderate to light volumes of traffic, of a trap length of 1,630 ft in run 4 was 15.67 veh, as compared to a mean of 8.72 in a trap length of 890 ft in run 5. The standard deviation of the concentrations in run 4 was approximately equal to the square root of the ratio of the trap length used in run 4 to the trap length used in run 5 times the standard deviation of concentrations in run 5, or

$$\sigma_4 = \sqrt{\frac{L_4}{L_5}}\,\sigma_5 \tag{5}$$

This would appear to be a fairly reliable relationship between the lengths of sections involved and the variability of concentrations for light and medium volumes. For the greatest volumes as well as for the congested flow conditions, the variability of concentration was observed to be considerably less than for the light to moderate volumes. It might be pointed out, however, that for runs 1 and 7, the relationship

$$\sigma_7 = \sqrt{\frac{L_7}{L_1}}\,\sigma_1 \tag{6}$$

did happen to be very nearly correct.

The standard deviations obtained in these four field studies were all calculated for an arbitrary trap length of 1,000 ft by using the relationship

$$\sigma_{1,000} = \sqrt{\frac{1,000}{L_m}}\ \sigma_m \tag{7}$$

These values, along with the approximate volumes, are given in Table 4. The values in these studies indicate that the standard deviation of concentrations observed in a 1,000-ft section of three lanes of freeway will be about 3 for high volume flow and also for congested lower volumes and, for light to moderate traffic flow conditions, will be about 4. That is, the standard deviation of concentrations observed in a 1,000-ft section was found to be about 4 for lower concentrations and 3 or less for medium to high concentrations.

SELECTION OF LENGTHS AND LOCATIONS FOR DENSITY SENSING

Locations of Critical or Bottleneck Sections

Aerial photogrammetry studies of traffic flow characteristics have been analyzed for density contours by May, Athol, Parker, and Rudden (5). In such an analysis, the concentrations of vehicles along the roadway are obtained for successive time intervals of about 5 min. A contour is obtained by plotting the densities on a chart with the station numbering (100-ft stations) as an abscissa and time of day as ordinate. Studies made during peak traffic flow periods will show the position, or station numbering, of high-density locations and the time of day the high densities exist. The duration of high density, or congested conditions, can be determined by noting the vertical height (time) of any particular density contour at a particular location along the freeway. If studies made on several different days have similar locations of high-density conditions, it is reasonably certain that the particular locations are critical or bottleneck sections.

The aerial photogrammetry studies made on the Gulf Freeway in the fall of 1962 were analyzed for the purpose of plotting density contours. Drew (6) has shown that

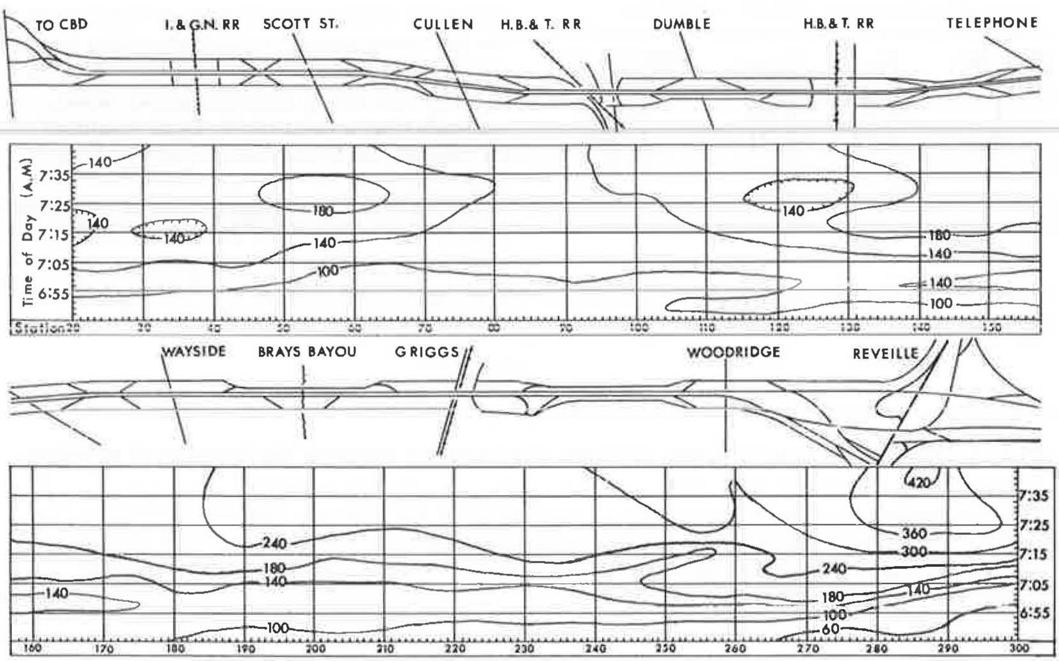

Figure 17. Density contours (three-lane total).

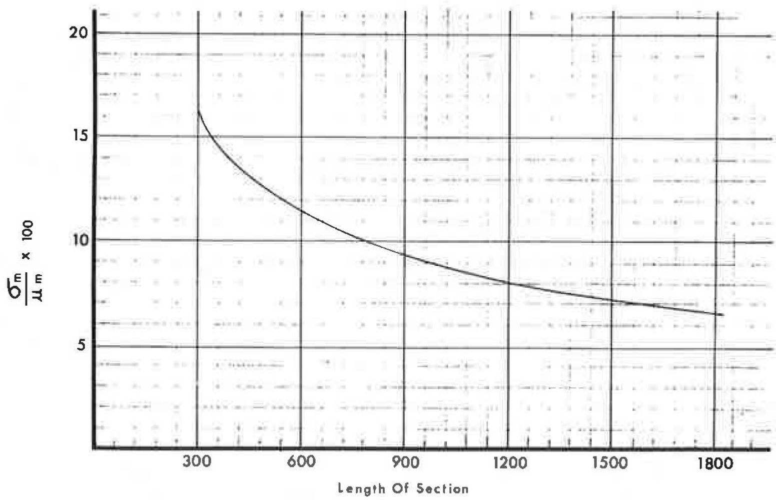

Figure 18. Relationship between standard deviation percent of mean concentration and length of sensing section at high volumes.

the morning peak flow toward the city of Houston results in several locations of high-density flow. Figure 17 shows the results of the three-lane density contour for the Gulf Freeway from 6:45 to 7:45 a.m. A line sketch of the freeway, frontage roads, entrance ramps, and exit ramps is shown above the contours.

It can be seen that in the region of Station 285 + 00 extremely high densities did occur. A three-lane density of over 420 veh/mi corresponds to almost a complete stoppage of vehicles, or, at best, a stop-and-go or crawling-speed condition. The merging of two state highways with the Gulf Freeway at this location presents a serious problem on the facility. Subsequent studies made on the Gulf Freeway indicate that the critical sections shown on this density contour generally tend to appear at the same locations more or less regularly.

This study procedure appears to provide a useful basis for the selection of density sensing locations. The contours also seem to indicate that lengths of 1,000 ft or less might suffice for density sensing trap lengths.

Considerations of Density Sensing Section Lengths

The volume-density relationship obtained from field studies of a section of the Gulf Freeway indicated that the density associated with the maximum flow rate was about 180 veh/mi for three lanes. The concentration in a section 1,000 ft long corresponding to this optimum density would be about 34 veh. The variability of concentration in a 1,000-ft section indicated that the standard deviation of concentration would be about three vehicles for moderately heavy or heavy volumes and that the distribution of concentrations would be approximately normal. A standard deviation which is only 9 percent of the mean concentration indicates the high degree of reliability which is inherent in sensing concentration (or density), and, as it has been pointed out, this sensing can be continuous, thus affording instantaneous indications of the traffic flow condition.

It may not be economically feasible or even necessary to sense density continuously along a freeway facility. A density contour analysis would seem to provide a basis for the selection of sections to be sensed. There is an indication that the standard deviation of the concentration, for any specific volume of traffic, is related to the length of section being sensed by Eq. 4:

$$\sigma_1 = \sqrt{\frac{L_1}{L_2}}\,\sigma_2$$

As the length of a sensing section increases, the concentration of vehicles increases proportionally. The standard deviation increases in proportion to the square root of the section length; thus, the ratio of the standard deviation to the mean decreases as the length of section increases. Figure 18 shows this relationship. It would appear that little increase in confidence would result from sections longer than 1,500 ft and that acceptable sensing reliability may be possible with sections as short as 500 ft.

CONCLUSIONS AND RECOMMENDATIONS

This study of vehicular density has singled out the one element of the traffic stream which, because of the difficulty of measuring it directly, has been largely relegated to be the dependent variable in the expression q = k v. By focusing attention on this element, this study offers these conclusions, subject to the recommendations which follow.

Conclusions

1. Density, or concentration, is an independent element of the traffic stream which is subject to direct measurement, is feasible to sense directly and is most directly related to the congestion of traffic. The q-k-v surface, as shown in Figure 1, is a useful concept in considering the relationship between speed, volume, and density.
2. Reasonably close agreement was found between the maximum or optimum density and theoretical optimum densities. It is possible, with field studies, to establish the optimum density, associated with a maximum volume, for particular sections of a freeway, and these optimum densities may not be the same for all sections along a freeway.
3. Frequency distributions of densities were found to be closely approximated by the Poisson distribution, or a random spacing of the vehicles, for light and medium volume conditions, or uncongested conditions; the distribution of densities was found to be considerably less variable, however, for heavy volume or congested flow conditions. These relationships agree with several theoretical considerations of traffic [illegible]
4. [illegible] which was approximated rather well by a parabolic relationship for the light traffic volumes up through the maximum volumes of flow.
5. The mean concentration was also proportional to the length of the section considered.
6. The variance of the concentrations was approximately proportional to the length of the section involved. Thus, the standard deviation was approximately proportional to the square root of the section length. For any particular volume, an increase in section length will result in a standard deviation of concentration which is a smaller percentage of the mean concentration.
7. Density sensing systems can offer the unique advantage of having continuous and instantaneous values for use in the control of freeway traffic systems and will not depend on some time interval for counting and averaging before a new output is available for control purposes.
8. It is possible to develop a density sensing system utilizing high-frequency, transmission-line type sensors which make use of voltage shifts caused by vehicles adjacent to the sensors.
9. Aerial photography methods of studying traffic characteristics can be utilized to determine volume-density relationships; such methods are useful in obtaining the density contours for a facility. The density contours can be used to identify the regular bottleneck sections which recur daily.
10. The density-trap method of study offers advantages in studying the continuously variable nature of density and the relationship between density and speed and volume.

Recommendations

An actual density sensing system needs to be developed and produced for evaluation. Such a system should have a continuous output proportional to the sum of the vehicles in the section at any instant and should not be based on the result of a subtraction process or data processing which subtracts a downstream count from an upstream count. A proper system would not involve a cumulative or increasing error if a particular sensor in the system failed, but would continue to give an output in error only by the amount attributed to the faulty sensor.

A density sensing system which has been properly evaluated should be installed at a section or sections on a freeway which have been previously selected as critical locations by the density contour method of analysis. The system should then be studied as an operational control system. The critical values of density predetermined from traffic studies should be corrected for any observed inaccuracies. It will be necessary to consider the entire freeway system in establishing critical densities for control to insure a balanced or optimum operation of the system.

Studies should be made to determine what minimum portion of the total length of controlled freeway should be sensed for density to obtain the required degree of reliability for operational control. A combination of volume sensing and density sensing could be considered in the determination of the minimum proportion of freeway to be sensed for density. Although density sensing can be useful in freeway operation control, a lengthy freeway section probably should have volume sensing used in combination with density sensing over certain lengths of the total section. A very long section sensed for only density would not present information concerning the bunching of vehicles within the sensed section; volume information would be useful in combination with density information in this respect.

Little has been done to relate the nature of one-lane traffic flow characteristics to multiple-lane flow characteristics. It is possible to sense density by lane as well as by total of all lanes. Relationships between individual lane densities and total densities should be established by further studies.

The density-trap method of studying traffic could be improved. Automatic vehicle counters could perhaps be utilized to replace the manual counters. The entire system could be operated wireless, or by radio, rather than by multi-wire cable and the voice communication could then be incorporated into the radio counters. In special locations where closed circuit television surveillance systems are in operation, it is possible to make density-trap studies by viewing the monitors. This study method offers direct information concerning concentration, volume rate, and speed and is a useful study procedure for determining the relationships between these basic elements of a traffic stream. The process of data reduction from the motion picture film is rather tedious. Improvements could be made in this process, particularly if the records were made on punched tape rather than motion picture film. A machine reduction of the information would then be possible.

The congested portion of the volume-density relationship should be investigated further. The density-trap method should prove to be a useful procedure for such studies. The right branch, or congested portion, of the volume-density relationship should be less influenced by the geometrical features of the roadway or the geographic location of the way drivers space themselves at slow speeds in congested conditions.

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Ramp-Freeway Terminal Operation as Related to Freeway Lane Volume Distribution and Adjacent Ramp Influence

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This report is a continuation of the report "Capacities and Characteristics of Ramp-Freeway Connections," contained in Highway Research Record No. 27. It terminates the analysis of the data collected during the nationwide Freeway Ramp Capacity Study, a project sponsored jointly by the Highway Research Board and the Bureau of Public Roads. It also contains an analysis of considerable data collected in a 1963 nationwide study of weaving areas by a crew of Bureau of Public Roads' junior engineers.

The emphasis in this report is on equations for determining traffic volumes in land 1, the right-hand freeway lane, at merging and diverging sections along the freeway. The main premise is that if the lane 1 volume can be kept at an acceptable level, the freeway as a whole will be operating free-flow. The equations take into account freeway volume, ramp volume, and traffic action on adjacent ramps which have an effect on the lane volume at the ramp under consideration. Once the lane 1 volume is determined upstream from an on-ramp, it is possible to determine the allowable ramp volume which in combination with the lane 1 volume will meet a specified service volume for the merge. At off-ramps, the lane 1 volume is that calculated upstream from the exit point, before divergence takes place. Thus, it is possible to forecast when overloading will occur on a section immediately upstream from an exit ramp, given certain freeway and exit ramp volumes.

Seventeen nomographs, derived from the equations, are introduced to provide a fast graphical solution to design and operational problems. Volumes ranging from below practical capacity up to possible capacity can be handled. This makes possible the solution of problems at different levels of service at the administrator's or designer's discretion.

Auxiliary lane usage between on- and off-ramps is discussed and a method of capacity analysis is illustrated with a sample problem. Curves are used to give the cumulative percentage of ramp vehicles on and off the auxiliary lane, related to the distance traversed.

Two-lane on-ramp operation is analyzed from the standpoint of the multiple merges taking place. Two-lane off-ramps are analyzed to show the ramp lane volume distribution and the diverging volume movement which takes place as off-ramp vehicles leave the main traffic stream at the ramp nose. The contribution of the deceleration lane to capacity and smooth operation is stressed.

Paper sponsored by Committee on Highway Capacity.

While data on major fork operation were very sparse, one equation was derived to enable forecasting of lane volume distributions upstream from a diverging major fork where three lane split into two, two-lane roadways.

Finally, preliminary results are given of on-ramp vehicle freeway lane usage studies made in Detroit on the 6-lane Edsel Ford Expressway. On-ramp vehicles were traced downstream after merging by means of the "lights on" technique. Analysis indicates that a stabilization of lane usage percentages occurs at approximately 6,000 feet downstream from the on-ramp nose, with 20 percent of the ramp vehicles in lane 3 at this point, 40 percent in lane 2 and 40 percent in lane 1.

•MUCH OF THIS REPORT is based on findings of the nationwide Freeway Ramp Capacity Study sponsored jointly by the Highway Research Board and the Bureau of Public Roads. The initial concept of a freeway ramp capacity was developed jointly in August 1958, by the late O. K. Normann, Chairman, HRB Committee on Highway Capacity and by its Subcommittee on Ramps, under Leo G. Wilkie. Field work was begun by the States starting in July 1960. The last of the collected data was received by the Bureau's Traffic Research Branch in December 1963.

Considerable data used in the analysis contained herein were collected in late 1963 by a group of junior engineers on a selected assignment in the Bureau of Public Roads' training program. Their task was primarily to study weaving sections as part of the Urban Weaving Area Capacity Study of the Bureau's Traffic Research Branch. This group, under direction of David Goss, a master's trainee, included Richard Fairbrother, Paul Heffern, Gunther Lerch, Manuel Marks, Barry Nunemaker, and John Ryan. Local State and municipal personnel provided welcome assistance to these men in many of the field studies.

GLOSSARY

added between an on-ramp and an off-ramp for speed change and weaving purposes.

2. Diverge: The traffic volume in lane 1 upstream from an off-ramp. The volume includes lane 1 through vehicles and off-ramp vehicles which expect to exit from lane 1.
3. Expected Diverge: The computed lane 1 volume upstream from an off-ramp.
4. Expected Merge: The computed lane 1 volume at an on-ramp nose plus the ramp volume moving onto lane 1 from the on-ramp.
5. Free-Flow Merge: Condition where freeway traffic is moving in a uniform manner somewhere in the 35-60-mph range. Large fluctuations in speeds are few and traffic is experiencing no conflicts severe enough to cause intermittent braking or congestion. Ramp traffic flow, though possible slower in speed than the freeway, is continuous without backup on the ramp. The merge of the two streams (lane 1 + ramp) is normally smooth within the usual adjustments in speed necessary for this maneuver. No specific overall speed should be associated with "free-flow," because the design and type of interchange will have an important effect on the speed at any one location.
6. Lane 1: The right-hand lane of the freeway.
7. Lane 2: The second lane from right-hand edge of the freeway.
8. Lane 3: The third lane from the right-hand edge of the freeway. This would be the median lane of a 6-lane freeway.
9. Lane A: In the case of a major fork, the land of the diverging roadway closest to the adjacent roadway.
10. Lane B: In the case of a major fork, the land of the diverging roadway farthest from the adjacent roadway.
11. Level of Service: This term, broadly interpreted, denotes any one of an infinite number of differing combinations of operating conditions that may occur on a

given lane or roadway when it is accommodating various traffic volumes. It is a qualitative measure of the effect of a number of factors which include: speed and travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs. In practice, selected specific levels are defined in terms of particular limiting values of certain of these factors.

A given route or route segment will normally consist of a number of roadway components. In addition to the through lanes, these components may include weaving areas, ramps, ramp terminals, auxiliary lanes and intersections. These various roadway components should provide operating characteristics in harmony with the specified level of service for the through lanes which comprise the basic framework for the overall route or route segment.

A given lane or roadway may provide a wide range of levels of service. The various levels for any specific roadway are functions of the volume and composition of traffic and of the speed attained. A lane or roadway designed for a certain level of service at a specified volume will actually operate at many different levels of service as the flow varies during an hour, and as the volume varies different hours of the day, days of the week, periods of the year, and during different years with traffic growth. Further, different types of highways, roads and streets, such as freeways, expressways at grade, major multilane highways, local 2-lane rural roads, urban arterial streets, and downtown streets, nearly always provide different levels of service that cannot be directly related to one another because each must be measured by a different standard or scale.

From the viewpoint of the driver, low flow rates or volumes on a given lane or roadway provide higher levels of service than greater flow rates or volumes on the same lane or roadway. Thus, the level of service for any particular lane or roadway varies inversely as some function of the flow or volume, or of the density.

The new Highway Capacity Manual includes narrative descriptions of prevailing traffic flow conditions which represent several levels of service. These levels encompass a working range of volumes from a condition of free-flow to a condition of capacity (formerly known as possible capacity). No attempt is made to describe any particular level as best, or to define any specific level as design level. Rather, it is the intent of the Committee that the new manual serve as a source for the basic criteria from which the user can select a volume which corresponds to the level of service best adapted to the specific need.

12. Ramp Lane A: The ramp lane closest to the freeway in the case of a 2-lane ramp.
13. Ramp Lane B: The ramp lane farthest from the freeway in the case of 2-lane ramps.
14. Rate-of-Flow or Hour Rate: The volume for a short period of time, such as five minutes expanded to a vehicles-per-hour figure by multiplication by $\frac{60}{\text{short period (minutes)}}$.
15. Service Volume: The maximum number of vehicles that can pass over a given section of a lane or roadway in one direction on multilane highways (or in both directions on a 2- or 3-lane highway) during a specified time period while operating conditions are maintained corresponding to the selected or specified level of service. In the absence of a time modifier, service volume is an hourly volume.

COLLECTION OF DATA

Sources and Locations

The data incorporated in this report came from the following sources:

1. State and municipal highway departments which collected data in 1960-1962 for the nationwide Freeway Ramp Capacity Study (1). These departments were: California, Colorado, Connecticut, Florida, Georgia, Illinois, Indiana, Kansas, Maryland, Michigan, Minnesota, Missouri, New Jersey, New York, Oregon, Pennsylvania, Rhode

Island, Texas, Virginia, Washington, District of Columbia, Port of New York Authority and Cook County, Illinois.

2. The Bureau of Public Roads Urban Weaving Area Capacity Study was made in late 1963 by junior engineers of the Bureau, assisted by local State and municipal personnel. These studies utilizing the "lights on" method of tracing vehicle paths were made in Washington, D. C., Philadelphia, Union and Paramus, New Jersey, Long Island and New York City, Detroit, Chicago, Sacramento, San Francisco, San Jose and Los Angeles. Weaving studies were the primary task of this group, but several studies were also made of successive on-ramps, of vehicle storage and operation on 2-lane cloverleaf loops, and of on-ramp vehicle paths downstream from their merging points. Some of the data collected by this group are incorporated in this report.

3. Local studies made by junior engineers as training projects on Virginia's Shirley Highway, the Baltimore-Washington Parkway, and in Baltimore on the Beltway and the Jones Fall Expressway. These studies were intended to fill some gaps in the basic data collected under items 1 and 2.

Field Procedure

The procedures used to collect data for the Freeway Ramp Capacity Study are given in detail in Highway Research Record No. 27, and will not be repeated herein.

The field procedures used in the Urban Weaving Area Capacity Study were similar in vehicle classification, but differed in other respects. Counts were made at entrances to and exits from weaving sections. In most of the studies, signs were erected along the ramp or minor roadway leading into the weaving area, requesting drivers to turn on headlights and keep them on for a distance of two miles. "Lights off" signs were erected at points downstream from the last counting location. Weaving areas having three entrances required the use of an additional set of signs which requested the use of parking lights. The most minor ramp would be signed for parking lights, while the intermediate volume ramp or roadway would be signed for operation of headlights. The heavy volume multilane entrance, usually a freeway, would be left unsigned. At the counting stations on the exit roadways leading out of the weaving section

had its headlights on; or in the case of three entrances, whether headlights, parking lights, or no lights were on. Whether the weaving area had two entrances or three entrances, it was possible to determine the weaving movements after adjustment was made on each time period for noncompliance vehicles. Driver compliance with the signs ranged from 65 to 98 percent, depending on the length of the roadway available for signing, sight distance, complexity of the location, and type of driving group (commuter or recreation). The overall compliance was approximately 85 percent.

The standard counting period used in the weaving studies was 5 minutes, followed by a 1-min recording period. At the more simple study locations, such as auxiliary lane weaves and successive on-ramps, continuous 5- or 6-min counts were made.

Speeds were obtained by a number of methods largely dependent on the type of site and the available manpower. Entrance and exit speeds were taken by radar or by stopwatch time measurement over a measured distance. Speeds through the weaving section were usually obtained by stopwatch using radio and walkie-talkie communication to describe vehicles being timed. In some studies, supplemental speeds were obtained by recording entrance nose and exit nose stopwatch times for vehicles of a certain type or make, such as Volkswagens of white, black, and red colors. These stopwatch times were later matched to determine average speed through the weaving section. At locations having particularly good vantage points, it was sometimes possible to time vehicles visually through the section.

The studies made in Detroit to determine on-ramp vehicle freeway lane usage downstream from the merge had a slightly different field procedure. The study ramp which usually carried 600-900 vph had "headlights on" signs along its length. The complying ramp vehicles were classified as to lane usage at all ramp-freeway junctions downstream for a distance of one to two miles. Regular counts were also taken by lane. At 1,000-ft intervals between interchanges, "lights on" vehicles only were counted by lane, so that the maximum distance between counting stations for "lights on" vehicles was 1,000 feet. Continuous 5-min counts were made. Speeds were taken by radar at the signed ramp junction with the freeway. Observers' remarks as to quality of traffic operation and estimated speeds were recorded at all interchange points downstream. The adjacent off-ramp upstream from the signed on-ramp was also counted as an aid in determining equations for lane 1 volume at the study on-ramp. A time differential for the different counting stations, assuming a 40-mph average speed for the freeway stream, was used and counting periods adjusted accordingly. The reason for this was to have observers downstream counting approximately the same block of vehicles in each 5-min recording period.

In the studies done by the Urban Weaving Area Capacity Study crew, miscellaneous information was recorded, as available manpower permitted. At some locations, auxiliary lane usage was recorded in some detail; at other locations, such as at successive on-ramps, vehicles entering the freeway from the first ramp had their lane usage recorded at the nose of the second on-ramp.

LANE 1 VOLUME EQUATIONS

There are a number of possible critical sections along any freeway—sections which because of vehicle concentration and/or maneuvering generally can be expected to have traffic breakdown at lower volumes than the normally expected possible capacity of 2,000 vph/lane for urban freeways. These critical sections can be classified generally as: (1) merging areas; (2) diverging areas; (3) weaving areas; (4) lane transition areas, such as where a lane is dropped; and (5) geometrically-marginal areas, such as freeway sections of steep grade or sharp curvature. The emphasis in this report is to provide means of determining the volumes which can be expected at critical areas (1) and (2), given certain freeway volumes, ramp volumes, and distances between ramps. Indirectly, recognition of the expected lane volume distributions (and allowance therefore in design) at critical merging and diverging areas should lead to the alleviation of weaving problems between on- and off-ramps. In other words, if operation can be kept free-flow in the merging and diverging sections of the freeway, simple weaving as found between on- and off-ramps should not be a problem.

The author's previous report (1) presented five equations which could be used to determine freeway lane 1 volume immediately upstream from the on-ramp nose given a certain freeway volume, on-ramp volume, and distances to and volumes of adjacent ramps. The lane 1 volume thus determined is that with which the ramp vehicles must merge. The addition of the lane 1 volume and ramp volume gives an "expected merge" for the ramp-freeway connection. Several formulas were given to estimate whether or not the "expected merge" would be a "free-flow merge." The computed latter volume is dependent on certain volumes, geometrics, traffic makeup, and population factors.

Not brought out, but perhaps much more important, is the fact that the "expected merge" can be checked against predetermined service volume levels, such as 1,200 vph and 1,500 vph. If the service volume level is exceeded by the "expected merge," this many indicate the location at which an extra through lane should be added if the desired level of service is to be maintained. A continuous check of critical sections along the freeway, as well as average lane volumes between interchanges, should enable the designer to produce a balanced freeway, relatively clear of bottlenecks.

This report adds a number of new equations to meet a wider range of conditions. Several of the original equations are updated, using additional data collected in 1963 and 1964. In the interest of continuity, two of the original equations, though not revised, are presented along with the updated and new equations. While merging situations were primarily treated in Highway Research Record No. 27, this report also treats diverging situations by equations to determine lane 1 volume upstream from off-ramps.

The equations for 4-, 6-, and 8-lane freeways contain three separate, dependent variables and nine independent variables. The dependent variables are as follows:

Y = (1) for an on-ramp equation, the lane 1 volume at the on-ramp nose just before the merge takes place.

(2) for an off-ramp equation, the lane 1 volume immediately upstream from the off-ramp nose, before divergence takes place.
(3) for a 2-lane off-ramp or a major fork, the lane 1 volume immediately downstream from the bifurcation.

Y' = (1) for a 2-lane on-ramp, the initial merge volume of lane 1 vehicles and ramp lane A vehicles (i. e., the left ramp lane or lane closest to the freeway lanes).
(2) for a 2-lane off-ramp, the volume in lane 1 before the divergence takes place which splits the volume into lane 1 and ramp lane A.

Y'' = (1) for a major fork at a 6-lane freeway, the volume in the center lane before it splits into lane 1 and lane A of the two forks.

The independent variables used in the different equations are as follows:

X_1 = (1) for an on-ramp equation, the total freeway volume in the one direction at the nose of the on-ramp before the merge takes place.
(2) for an off-ramp equation, the total freeway volume, including prospective off-ramp vehicles, upstream of (approaching) the off-ramp.

X_2 = for an on-ramp equation, the volume using the ramp being considered in the prospective merge.

X_3 = distance in feet measured nose-to-nose from the ramp under consideration to an adjacent upstream off-ramp.

X_4 = volume of an adjacent upstream off-ramp.

X_5 = distance in feet measured nose-to-nose from the ramp under consideration to an adjacent downstream off-ramp. Where an auxiliary lane is added between ramp noses, this distance is identical to the length of the auxiliary lane.

X_6 = volume of an adjacent downstream off-ramp.

X_7 = (1) for an off-ramp equation, the volume exiting at the off-ramp under consideration.
(2) for a major fork, the volume using the right-hand roadway.

[illegible] tion to the adjacent upst[illegible]

X_9 = volume of the adjacent upstream on-ramp.

The volumes are given in vehicles per hour (vph), but if so desired can be hourly rates expanded from short period counts since the equations are derived from 5- or 6-min volumes expanded to hourly rates. The use of short period counts expanded to hourly rates accounts for close to maximum variability in traffic performance from the available data.

The equations yield estimates of lane 1 volumes which are subject to standard errors of estimate. The actual hourly volumes of the independent variables equal or are approximately equal to the average value of the 5- or 6-min volume rates. Had the equations been fitted to the actual hour volumes, the standard errors would have been smaller. Therefore, for use in hour volume calculations, the standard errors tend to be conservative (i. e., higher than they should be).

The equations and the conditions for their use segregated by 4-, 6-, and 8-lane freeways are as follows:

4-Lane Freeways

$$Y = 136 + 0.345X_1 - 0.115X_2 \quad (1)$$

Condition: Used to determine the lane 1 volume at an on-ramp nose before merging takes place. The ramp can be of any single-lane type except cloverleaf loop (see Eqs. 2a, 2b, and 3). An acceleration lane may or may not be present.

The equation replaces Eq. 5 (1, p. 112). The replaced equation, derived from 187 observations, contained the distance to and volume of the adjacent downstream off-ramp as variables although the volume was not statistically significant. The new equation presented herein is derived from 431 observations and contains only the freeway volume, X_1, and the on-ramp volume, X_2, as variables.

$$Y = 166 + 0.280X_1 \quad \text{on-ramp} < 600 \text{ vph} \tag{2a}$$
$$Y = 128 + 0.482X_1 - 0.301X_2 \quad \text{on-ramp } 600 - 1,200 \text{ vph} \tag{2b}$$

Condition: Used to determine the lane 1 volume at an inner loop on-ramp nose at a cloverleaf interchange which lacks an auxiliary lane connecting the loop ramps. An acceleration lane may or may not be present. This design lacking an auxiliary lane is not apt to be found along modern freeways but a number of such interchanges are still in use on older freeways. Eq. 2a, which is for the condition where the on-ramp is less than 600 vph, contains only the freeway volume, X_1, as a variable. Eq. 2b, for the condition where the on-ramp volume is 600-1,200 vph, contains both the freeway volume, X_1, and on-ramp volume, X_2, as variables.

$$Y = 195 + 0.273X_1 - 0.146X_2 + 0.723X_6 \tag{3}$$

Condition: Used to determine the lane 1 volume at an inner loop on-ramp nose at a cloverleaf interchange containing an auxiliary lane between the loops. Assuming all prospective off-ramp vehicles are in lane 1 at the on-ramp nose, it is possible to determine the number of through vehicles in lane 1 by subtracting the off-ramp volume from the computed lane 1 volume. By reference to Figure 19 for auxiliary lane usage, it is possible to determine lane 1 and auxiliary lane volumes at points along the auxiliary lane. These volumes should not exceed the service volume level being used for design.

The variables in the equation are (a) freeway volumes, X_1; (b) on-ramp volumed, X_2; and (c) loop off-ramp volume, X_6. While the distance between loops might well be a significant variable, there was too little variation in the study sites to get a significant coefficient.

$$Y = 123 + 0.376X_1 - 0.142X_2 \tag{4}$$

Condition: Used to determine the lane 1 volume at the on-ramp nose of the second of successive on-ramps when the ramps are within 400 to 2,000 feet of each other. The only significant variables are the freeway volume, X_1, and the volume of the second on-ramp, X_2. The equation is not applicable to situations where the upstream on-ramp volume, X_9, is near its maximum of 1,000 vph, as given in Table 1, and the distance between ramps in feet, X_8, is near its minimum of 400 feet. It should be noted that these variables, X_8 and X_9, while not in the equation, nevertheless, have a required range for accurate use of the equation.

$$Y = 165 + 0.345X_1 + 0.520X_7 \tag{5}$$

Condition: Used to determine the lane 1 volume upstream from an off-ramp just before divergence takes place. The ramp may or may not have a deceleration lane. The equation contains as variables, the freeway volume, X_1 (which includes the off-ramp vehicles), and the off-ramp volume, X_7. If there is an adjacent upstream on-ramp within 3,200 feet, Eq. 6 can be used with more accuracy.

$$Y = 202 + 0.362X_1 + 0.496X_7 - 0.069X_8 + 0.096X_9 \tag{6}$$

Condition: Same as in Eq. 5, except that there is an upstream, adjacent on-ramp within 3,200 feet. In addition to X_1 and X_7, the equation includes X_8, the distance in feet to, and X_9, the volume of, this adjacent upstream on-ramp.

6-Lane Freeways

$$Y = 121 + 0.244X_1 - 0.085X_4 + 640\frac{X_6}{X_5} \tag{7}$$

Condition: Used to determine the lane 1 volume at an on-ramp nose before merging takes place. The ramp can be of any single-lane type, except cloverleaf loop (see Eq. 8). An acceleration lane may or may not be present. In the earlier study this equation is presented as Eq. 1 (1, p. 95). Included as variables are (a) the freeway volume,

X_1; (b) the adjacent upstream off-ramp volume, X_4; (c) the distance in feet to the adjacent downstream off-ramp, X_5; and (d) volume of adjacent downstream off-ramp, X_6. The on-ramp volume, X_2, is not included as a variable at a small loss in accuracy of the equation. The reason for this is to facilitate computation by elimination of the "cut and try" which would be necessary to determine the allowable ramp volume for a given service volume level of merge if the X_2 volume was included in the equation.

$$Y = 87 + 0.225X_1 - 0.140X_2 + 0.500X_6 \tag{8}$$

Condition: Used to determine the lane 1 volume at the inner loop on-ramp nose at a cloverleaf interchange containing an auxiliary lane between the loops. The interchange may or may not have outer connections. The equation does not consider their effects and applies only to inner loop on-ramps. This equation is the same as Eq. 4 in the previous study (1, p. 95).

As explained under Eq. 3, the volumes in lane 1 and the auxiliary lane at selected points can be determined by use of Figure 19.

$$Y = 574 + 0.228X_1 - 0.194X_2 - 0.714X_8 + 0.274X_9 \tag{9}$$

Condition: Used to determine the lane 1 volume at the on-ramp nose of the second of successive on-ramps. The variables used in the equation are (a) the freeway volume, X_1; (b) the second on-ramp volume, X_2; (c) the distance in feet to the adjacent upstream on-ramp, X_8; and (d) the volume of the adjacent upstream on-ramp, X_9.

$$Y = 53 + 0.283X_1 - 0.402X_5 + 0.547X_6 \tag{10}$$

Condition: Used to determine the lane 1 volume at the nose of an on-ramp where there is an auxiliary lane to the adjacent downstream off-ramp. The equation is primarily intended for diamond and similar flat angle entrance ramps, while Eq. 8 is used for cloverleaf loop ramps. The equation is derived from locations having auxiliary lanes 300 to 1,100 feet long. Use of the equation for locations with longer auxiliary lanes may introduce some error in the calculation not completely covered by the

distance to, X_5, and volume, X_6, of the downstream off-ramp. The on-ramp volume, X_2, was not found statistically significant in affecting the lane 1 volume so it is not contained in the equation.

This equation replaces Eqs. 2 and 3 in the earlier study (1, p. 95).

$$Y = 54 + 0.070X_1 + 0.049X_2 \tag{11a}$$

where Y = Lane 1.

$$Y' = -205 + 0.287X_1 + 0.575X_2 \tag{11b}$$

where Y' = Lane 1 + ramp lane A.

Ramp lane B = total ramp - ramp lane A

Condition: Used to determine the initial merge (lane 1 + ramp lane A) and the ramp lane volume distribution at a 2-lane on-ramp, having an acceleration lane at least 800 feet long. There is no extra lane added to the freeway for this condition, so while sometimes used, this design is not generally recommended except under special volume conditions.

Eq. 11a is used to determine the lane 1 volume at the on-ramp nose and uses the freeway volume, X_1, and on-ramp volume, X_2, as variables. After the lane 1 volume is calculated, Eq. 11b is used to calculate the initial merge which consists of (lane 1 + ramp lane A). The ramp lane A volume can be found by subtracting the Eq. 11a calculation of lane 1 from the Eq. 11b calculation of (lane 1 + ramp lane A). Ramp lane B volume is then the total on-ramp volume less ramp lane A volume. Both the (lane 1 + ramp lane A) volume and the ramp lane B should be checked against the service volume level.

The ramp lane B vehicles use the acceleration lane and have to merge with the (lane 1 + ramp lane A) vehicles which have already merged. Some of the latter vehicles will move into other lanes of the freeway before the ramp lane B vehicles merge with the remainder.

$$Y = 139 + 0.242X_1 + 0.408X_7 \qquad (12)$$

Condition: Used to determine lane 1 volume upstream from an off-ramp before divergence takes place. A deceleration lane may or may not be present. The equation includes as variables the freeway volume, X_1 (which includes off-ramp vehicles about to exit), and the off-ramp volume, X_7.

$$Y' = -158 + 0.035X_1 + 0.567X_7 \qquad (13a)$$

where Y' = (lane 1 + ramp lane A).

$$Y = 18 + 0.060X_1 + 0.072X_7 \qquad (13b)$$

where Y = lane 1.

Ramp lane A = Y' - Y
Ramp lane B = total ramp - ramp lane A

Condition: Used to determine (lane 1 + ramp lane A) volume immediately upstream from the divergence and lane 1 volume just past the bifurcation at a 2-lane off-ramp having a deceleration lane at least 700 feet long. No lane is dropped at the off-ramp, so the freeway continues to be three lanes downstream from the ramp. Ramp lane B vehicles use the deceleration lane for at least part of its length, while ramp lane A vehicles diverge directly from the freeway onto the ramp. The importance of an adequate deceleration lane cannot be overemphasized. Without an adequate deceleration lane, the ramp cannot be fed two lanes of traffic from the freeway. To carry volumes over 2,000 vph, lane 1 vehicles should be moving into the deceleration lane for exit in ramp lane B; simultaneously, lane 2 vehicles will be moving into the vacated lane 1 spaces and most of these vehicles will exit in ramp lane A. The minimum length of 700 feet for use of the equations is not necessarily to be considered a design recommendation. Most design standards should exceed this length.

Eq. 13a is used to determine the (lane 1 + ramp lane A) volume, while Eq. 13b is used to calculated lane 1 volume continuing on past the ramp bifurcation. Ramp lane A can be determined by subtracting lane 1 from (lane 1 + ramp lane A). Ramp lane B volume can be calculated by subtracting ramp lane A volume from the total off-ramp volume. The following volumes should be checked against the service volume: ramp lane B and (lane 1 + ramp lane A).

$$Y'' = 64 + 0.285X_1 + 0.141X_7 \qquad (14a)$$

where Y'' = (lane 1 + lane A).

$$Y = 173 + 0.295X_1 + 0.320X_7 \qquad (14b)$$

where Y = lane 1.

Condition: Used to determine lane volumes at a major fork, where three lanes split into two, 2-lane roadways. Using lanes 1 and 2 to denote one roadway and lane A and lane B to denote the other roadway, the ratio of $\frac{\text{lane 1 + lane 2}}{\text{lane A + lane B}}$ volume should fall between $\frac{1}{3}$ and 3 for the equation to be valid.

Eq. 14a is used to calculate the (lane 1 + lane A) volume, which is subject to divergence at the nose of the bifurcation. Eq. 14b is used to determine the lane 1 volume, which continues on past the nose. Knowing the total volumes of each roadway, lane 2 and lane B volumes can be calculated. The following volumes should be checked against the service volume: (lane 1 + lane A), lane 2, and lane B.

8-Lane Freeways

$$Y = -132 + 0.201X_1 + 0.127X_2 \tag{15}$$

Condition: Used to determine lane 1 volume at the on-ramp nose before merging takes place. An acceleration lane should be present. The equation uses as variables the freeway volume, X_1; and the on-ramp volume, X_2.

$$Y = -353 + 0.199X_1 - 0.057X_2 + 0.486X_6 \tag{16}$$

Condition: Used to determine lane 1 volume at the on-ramp nose before merging takes place. An acceleration lane should be present and there should be an adjacent downstream off-ramp 1,500 to 3,000 feet away. The equation uses as variables the freeway volume, X_1; the on-ramp volume, X_2; and the volume of the adjacent downstream off-ramp, X_6. The distance in feet to the adjacent downstream off-ramp, X_5, is a required condition (1,500-3,000 feet), but is not numerically a part of the equation.

$$Y = 584 + 0.180X_1 - 0.203X_2 - 0.487X_5 + 0.204X_6 \tag{17}$$

Condition: Used to determine the lane 1 volume at the nose of an on-ramp where there is an auxiliary lane to the adjacent downstream off-ramp. The equation is primarily intended for diamond and similar flat angle entrance ramps though it can be used in lieu of an equation for a cloverleaf loop ramp on an 8-lane freeway. The equation has as variables the freeway volume, X_1; the on-ramp volume, X_2; the distance feet to the adjacent downstream off-ramp, X_5; and the volume of the adjacent downstream off-ramp, X_6.

TABLES AND NOMOGRAPHS FOR EQUATIONS

The equations and their associated statistical data, including sketches, are shown in Tables 1, 2, 3, and 4. The mean and standard deviation shown for each equation variable are those from the data used to derive the equation by multiple linear re-

[illegible]

able not included in the equation itself, the components should be within the variable range regardless, since conditions outside the stated range could result in a different equation. Also shown as statistical background are each of the net regression coefficients and their associated standard errors; also, the coefficient of determination and the level of significance of the variable. On the right side of the tables, the R^2 value, known as the coefficient of multiple determination, is shown for the equation. An R^2 of 0.90 indicates that the equation accounts for 90 percent of the variability of the dependent variable. Some factors not included in the equations, which could explain some of the unexplained variance, are hard-to-measure items such as: trip lengths, signing effectiveness, and ramp action upstream or downstream from the ramp under consideration, but not adjacent to the ramp. Actually, in some of the equations, such as that for cloverleaf loops on a 6-lane freeway (Eq. 8), the adjacent upstream off-ramp, when present, was not counted in the field work, so could not be included in the equation. If it had been, the standard error most probably would have been lower and the R^2 value higher.

Other statistics shown on the right side of the tables are (a) the standard error of the dependent variable, (b) the number of different locations used to provide data for the multiple regression, and (c) the number of periods or observations used in the analysis. Each of the observations was either a 5- or 6-min count expanded to an hourly rate. Observations were not used if congested operation, consisting usually of stop-and-go accordion action, was present during the period counted. Some very heavy volume periods were used provided the operation was steady, even if speeds were down around 30 mph in some cases. The objective was to provide equations which could handle a wide range of volume conditions representing different levels of service.

There may be some question as to why some variables are found in one equation but not in another. In some cases, a variable was not used because it was not statistically

TABLE 1

FORMULAS OF LANE 1 VOLUME FOR 4-LANE FREEWAYS

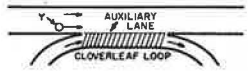
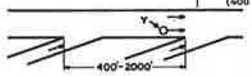
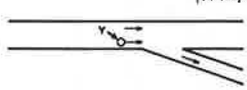
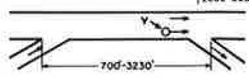
EQUATION NUMBER (Corresponds to Nomograph of Same Figure Number)	VARIABLES — DEPENDENT: Y, Volume in lane 1 v.p.h.	INDEPENDENT: X_1, Freeway volume v.p.h.	X_2, On-Ramp volume v.p.h.	X_5, Dist. to Adjacent Downstrm. Off-Ramp feet	X_6, Vol. of Adjacent Downstrm. Off-Ramp v.p.h.	X_7, Off-Ramp volume v.p.h.	X_8, Dist. to Adjacent Upstrm. On-Ramp feet	X_9, Vol. of Adjacent Upstrm. On-Ramp v.p.h.	R^2	Standard Error of Y	Number of Locations	Number of Observations	REMARKS
1. Standard On-Ramp									.83	113	20	431	Use equation 4 if there is an adjacent upstream On-Ramp within 2000 ft. 
Mean	736	1925	513										
Standard Deviation	262	683	118										
Range of Use of Variables		400-3400	50-1400										
Net Regression Coefficient		+ 0.345	- 0.115										
Std. Error of Net Regression Coefficient		.008	.017										
Partial Determination Coefficient		.82	.09										
Level of Significance		.01	.01										
Equation No. 1	$Y = 136 + 0.345X_1 - 0.115X_2$												
2a. Cloverleaf On-Ramp, No Auxiliary Lane (Ramp < 600 v.p.h.)									.42	102	6	75	
Mean	517	1254	359										
Standard Deviation	132	305	168										
Range of Use of Variables		600-2000	0-599										
Net Regression Coefficient		+ 0.280											
Std. Error of Net Regression Coefficient		.039											
Partial Determination Coefficient		.42											
Level of Significance		.01											
Equation No. 2a	$Y = 166 + 0.280X_1$												
2b. Cloverleaf On-Ramp, No Auxiliary Lane (Ramp 600-1200 v.p.h.)									.82	111	4	51	See sketch above.
Mean	460	1190	805										
Standard Deviation	254	409	166										
Range of Use of Variables		600-2000	600-1200										
Net Regression Coefficient		+ 0.482	- 0.301										
Std. Error of Net Regression Coefficient		.047	.115										
Partial Determination Coefficient		.69	.13										
Level of Significance		.01	.01										
Equation No. 2b	$Y = 128 + 0.482X_1 - 0.301X_2$												
3. Cloverleaf On-Ramp with Auxiliary Lane									.82	96	7	151	
Mean	676	1468	479		208								
Standard Deviation	222	672	379		100								
Range of Use of Variables		600-3600	100-1500	400-750	50-500								
Net Regression Coefficient		+ 0.273	- 0.146		+ 0.723								
Std. Error of Net Regression Coefficient		.012	.023		.087								
Partial Determination Coefficient		.79	.22		.32								
Level of Significance		.01	.01		.01								
Equation No. 3	$Y = 195 + 0.273X_1 - 0.146X_2 + 0.723X_6$												
4. Second of Successive On-Ramps									.93	76	7	148	The equation is not applicable to situations where X_9 is near its maximum (1000 vph) while X_8 is near its minimum (400 feet). 
Mean	829	2075	527				1002	296					
Standard Deviation	285	666	415				497	217					
Range of Use of Variables		800-3600	100-1500				400-2000	100-1000					
Net Regression Coefficient		+ 0.376	- 0.142										
Std. Error of Net Regression Coefficient		.010	.016										
Partial Determination Coefficient		.91	.35										
Level of Significance		.01	.01										
Equation No. 4	$Y = 123 + 0.376X_1 - 0.142X_2$												
5. Off-Ramp (No Nearby Adjacent Ramps)									.90	131	19	518	Use equation 6 if there is an adjacent upstream on-ramp within 3200 ft. 
Mean	1022	1828				434							
Standard Deviation	410	828				307							
Range of Use of Variables		400-4200				50-1500							
Net Regression Coefficient		+ 0.345				+ 0.520							
Std. Error of Net Regression Coefficient		.008				.021							
Partial Determination Coefficient		.79				.73							
Level of Significance		.01				.01							
Equation No. 5	$Y = 165 + 0.345X_1 + 0.520X_7$												
6. Off-Ramp with Adjacent Upstream On-Ramp									.97	83	5	124	Use equation 5 if the adjacent upstream on-ramp is more than 3200 feet away. 
mean	1223	2440				497	1907	236					
Standard Deviation	466	963				325	1034	211					
Range of Use of Variables		700-4200				50-1600	700-3200	50-900					
Net Regression Coefficient		+ 0.362				+ 0.496	- 0.069	+ 0.096					
Std. Error of Net Regression Coefficient		.013				.038	.011	.055					
Partial Determination Coefficient		.87				.59	.25	.02					
Level of Significance		.01				.01	.01	.10					
Equation No. 6	$Y = 202 + 0.362X_1 + 0.496X_7 - 0.069X_8 + 0.096X_9$												

TABLE 2

FORMULAS FOR LA 1 VOLUME FOR 6-LANE FREEWAYS

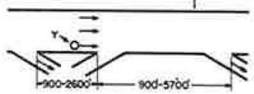
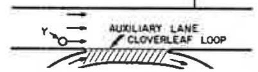
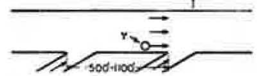
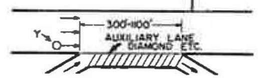
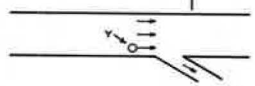
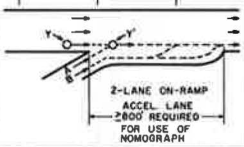
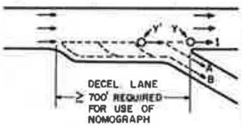
EQUATION NUMBER (Corresponds to Nomograph of Same Figure Number)	V A R I A B L E S											R^2	Standard Error of Y	Number of Locations	Number of Observations	
	DEPENDENT	INDEPENDENT														
	Y	X_1	X_2	X_3	X_4	X_5	X_6	X_6/X_5	X_7	X_8	X_9					
	Volume in Lane 1 v.p.h	Freeway volume v.p.h.	On-Ramp volume v.p.h.	Dist. to Adjacent Upstream Off-Ramp feet	Vol. of Adjacent Upstream Off-Ramp v.p.h.	Dist. to Adjacent Dwnstrm. Off-Ramp feet	Volume of Adjacent Dwnstrm. Off-Ramp v.p.h.		Volume v.p.h.	Dist. to Adjacent Upstream On-Ramp feet	Volume of Adjacent Upstream On-Ramp v.p.h.		v.p.h.			
7. Standard On-Ramp with Adjacent Off-Ramps												.80	140	12	325	See Figure 7 for instructions pertaining to conditions where adjacent ramps fall outside the variable limits. 
Mean	1041	4327	594	1459	465	2482	449									
Standard Deviation	312	943	349	558	251	1404	260									
Range of Use of Variables		2400-6200	100-1700	900-2600	50-1100	5700	50-1300	See X_5&X_6								
Net Regression Coefficient		+ 0.244			- 0.085			+ 640								
Std. Error of Net Regression Coefficient		.008			.035			50								
Partial Determination Coefficient		.72			.02			.34								
Level of Significance		.01			.01			.01								
Equation No. 7	$Y = -121 + 0.244X_1 - 0.085X_4 + 640\ X_6/X_5$															
8. Cloverleaf Inner Loop with Auxiliary Lane												.64	178	7	136	This equation is applicable to cloverleaf inner loops whether or not the interchange has outer connections. 
Mean	878	3792	810				447									
Standard Deviation	293	790	383				282									
Range of Use of Variables		2000-5600	200-1500			850	150-1500									
Net Regression Coefficient		+ 0.225	- 0.140				+ 0.500									
Std. Error of Net Regression Coefficient		.019	.040				.054									
Partial Determination Coefficient		.51	.08				.39									
Level of Significance		.01	.01				.01									
Equation No. 8	$Y = -87 + 0.225X_1 - 0.140X_2 + 0.500X_6$															
9. Second of Successive On-Ramps												.91	142	5	104	
Mean	686	3273	480							891	348					
Standard Deviation	453	1005	422							261	222					
Range of Use of Variables		1800-5400	100-1500							500-1100	100-1400					
Net Regression Coefficient		+ 0.223	- 0.194							- 0.714	+ 0.274					
Std. Error of Net Regression Coefficient		.017	.036							.075	.089					
Partial Determination Coefficient		.64	.22							.48	.09					
Level of Significance		.01	.01							.01	.01					
Equation No. 9	$Y = 574 + 0.223X_1 - 0.194X_2 - 0.714X_8 + 0.274X_9$															
10. Diamond, etc., with Auxiliary Lane												.86	159	6	155	
Mean	1164	4139				627	354									
Standard Deviation	419	1179				222	221									
Range of Use of Variables		1900-5600				-1100	50-1000									
Net Regression Coefficient		+ 0.283				0.402	+ 0.547									
Std. Error of Net Regression Coefficient		.013				.079	.090									
Partial Determination Coefficient		.76				.15	.20									
Level of Significance		.01				.01	.01									
Equation No. 10	$Y = 53 + 0.283X_1 - 0.402X_5 + 0.547X_6$															
12. Upstream from Off-Ramp												.83	133	11	277	
Mean	1475	4590							556							
Standard Deviation	319	1366							365							
Range of Use of Variables		1100-6200							20-1800							
Net Regression Coefficient		+ 0.242							+ 0.408							
Std. Error of Net Regression Coefficient		.007							.025							
Partial Determination Coefficient		.91							.70							
Level of Significance		.01							.01							
Equation No. 12	$Y = 139 + 0.242X_1 + 0.408X_7$															

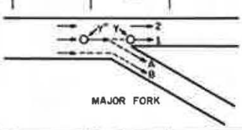
TABLE 3

FORMULAS FOR 2-LANE RAMPS AND MAJOR FORK FOR 6-LANE FREEWAYS

Equation Number (Corresponds to Nomograph Same Figure Number)	VARIABLES: DEPENDENT			INDEPENDENT			R^2	Standard Error of Y, Y', or Y''	Number of Locations	Number of Observations	REMARKS
	Y	Y'	Y''	X_1	X_2	X_7*					
	Volume in Lane 1	Volume in (Lane 1 + Ramp Lane A)	Volume in (Lane 1 + Lane A)	Freeway Volume	On-Ramp Volume	Off-Ramp Volume					
11a. 2-Lane On-Ramp with Acceleration Lane							.23	99	2	115	(1) Ramp Lane A vehicles merge directly with Lane 1 vehicles. (2) Ramp Lane B vehicles use acceleration lane which should be a minimum of 800 feet in length for use of the equation. (3) Some of the merged (Lane 1 + Ramp Lane A) vehicles move to Lane 2 before Ramp Lane B vehicles merge with the remainder.
Lane 1 Calculation											
Mean	270			1625	2091						
Standard Deviation	112			611	453						
Range of Use of Variables				600-3000	1100-3000						
Net Regression Coefficient				+ 0.070	+ 0.049						
Std. Error of Net Regression Coefficient				.016	.021						
Partial Determination Coefficient				.15	.04						
Level of Significance				.01	.02						
Equation No. 11a.	$Y = 54 + 0.070X_1 + 0.049X_2$										
11b. Lane 1 + Ramp Lane A Calculation							.74	212	same as 11a	same as 11a	
Mean		1464		1625	2091			SKETCH -- Same as above.			
Standard Deviation		413		611	453						
Range of Use of Variables				600-3000	1100-3000						
Net Regression Coefficient				+ 0.287	+ 0.575						
Std. Error of Net Regression Coefficient				.034	.046						
Partial Determination Coefficient				.39	.58						
Level of Significance				.01	.01						
Equation No. 11b.	$Y' = -205 + 0.287X_1 + 0.575X_2$										
13a. 2-Lane Off-Ramp with Deceleration Lane							.68	181	1	94	Deceleration lane should be at least 700 feet long for accurate use of equations 13a and 13b.
Lane 1 + Ramp Lane A Calculation											
Mean		1093		3639		1983					
Standard Deviation		315		814		433					
Range of Use of Variables				2100-6000		1100-3000					
Net Regression Coefficient				+ 0.035		0.567					
Std. Error of Net Regression Coefficient				.026		.049					
Partial Determination Coefficient				.02		.59					
Level of Significance				.20		.01					
Equation No. 13a.	$Y' = -158 - 0.035X_1 + 0.567X_7$										
13b. Lane 1 Calculation							.18	150	same as 13a	same as 13a	
Mean	378			3639		1983		SKETCH -- Same as above.			
Standard Deviation	163			814		433					
Range of Use of Variables				2100-6000		1100-3000					
Net Regression Coefficient				+ 0.060		+ 0.072					
Std. Error of Net Regression Coefficient				.022		.041					
Partial Determination Coefficient				.08		.03					
Level of Significance				.01		.10					
Equation No. 13b.	$Y = 18 + 0.060X_1 + 0.072X_7$										
14a. Major Fork (Lane 1 + Lane A Calculation							.91	94	3	84	Equations 14a and 14b should not be used if the volume ratio of $\frac{(\text{Lane 1 + Lane 2})}{(\text{Lane A + Lane B})}$ does not fall between 1/3 and 3.
Mean			1151	3170		1290					
Standard Deviation			314	847		571					
Range of Use of Variables				1200-4500		300-2650					
Net Regression Coefficient				+ 0.285		+ 0.141					
Std. Error of Net Regression Coefficient				.016		.024					
Partial Determination Coefficient				.80		.31					
Level of Significance				.01		.01					
Equation 14a.	$Y'' = 64 + 0.285X_1 + 0.141X_7$										
14b. Lane 1 Calculation							.90	66	same as 14a.	same as 14a.	
Mean	694			3170		1290		SKETCH -- Same as above.			
Standard Deviation	201			847		571					
Range of Use of Variables				1200-4500		300-2650					
Net Regression Coefficient				+ 0.295		- 0.320					
Std. Error of Net Regression Coefficient				.011		.017					
Partial Determination Coefficient				.90		.82					
Level of Significance				.01		.01					
Equation 14b.	$Y = 173 + 0.295X_1 - 0.320X_7$										



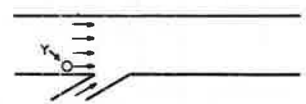
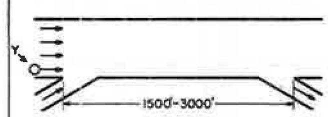
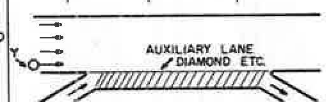




* In Equation 14a, also taken as a leg at a major fork.

TABLE 4

FORMULAS FOR LANE VOLUME FOR 8-LANE FREEWAYS

EQUATION NUMBER (Corresponds to Nomograph of Same Figure Number)	VARIABLES: DEPENDENT: Y, Volume in Lane 1 v.p.h.	INDEPENDENT: X_1, Freeway Volume v.p.h.	X_2, On-Ramp Volume v.p.h.	X_5, Distan to Adj Dwnstr Off-Ra feet	X_6, lume of istrm. -Ramp p.h.	R^2	STANDARD ERROR OF Y (v.p.h.)	NUMBER OF LOCATIONS	NUMBER OF OBSERVATIONS	REMARKS
15. Standard On-Ramp						.68	156	5	104	Use equation 16 if there is an adjacent downstream off-ramp within 3000 feet.
Mean	974	5979	833							
Standard Deviation	270	1072	220							
Range of Use of Variables		3000-7700	300-1300							
Net Regression Coefficient		+ 0.201	+ 0.127							
Std. Error of Net Regression Coefficient		.014	.071							
Partial Determination Coefficient		.660	.030							
Level of Significance		.010	.100							
Equation No. 15	$Y = -312 + 0.201X_1 + 0.127X_2$									
16. On-Ramp with Downstream Off-Ramp						.88	107	2	48	Use equation 15 when the adjacent downstream off-ramp is more than 3000 feet away or when adjacent ramp function differs.
Mean	851	5367	739		368					
Standard Deviation	292	1149	223		160					
Range of Use of Variables		3000-7300	300-1100		0-800					
Net Regression Coefficient		+ 0.199	- 0.057		.486					
Std. Error of Net Regression Coefficient		.016	.074		.116					
Partial Determination Coefficient		.790	.010		.280					
Level of Significance		.010	.250		.010					
Equation No. 16	$Y = -353 + 0.199X_1 - 0.057X_2 + 0.4$									
17. On-Ramp with Auxiliary Lane						.86	107	5	72	
Mean	1237	5897	515	856	557					
Standard Deviation	274	897	255	226	279					
Range of Use of Variables		3670-7500	110-1220	500-1	10-1220					
Net Regression Coefficient		+ 0.180	- 0.203	- 0.48	0.204					
Std. Error of Net Regression Coefficient		.015	.058	.06	.051					
Partial Determination Coefficient		.680	.150	.43	.200					
Level of Significance		.010	.010	.01	.010					
Equation No. 17	$Y = 584 + 0.180X_1 - 0.203X_2 - 0.48$ $0.204X_6$									

significant; in other cases, because the basic data were not collected in the field work. In practically all cases, a variable has the same relative effect (plus or minus coefficient) in all the equations containing it. For instance, freeway volume, X_1, is always a very strong "plus" coefficient, while on-ramp volume, X_2, is a "minus" coefficient, because the higher the ramp volume, the more the possibility that lane 1 vehicles will consider it desirable to move to adjacent lane 2. A downstream off-ramp tends to cause loading of lane 1 upstream; for this reason X_6 is a strong "plus" coefficient. An adjacent off-ramp upstream, on the other hand, would open up gaps in lane 1, so the coefficient is "minus"; i. e., the greater the exiting volume, the less the volume in lane 1 downstream. The distance to an adjacent downstream off-ramp is a "minus" coefficient, because the greater the distance, the less the need for prospective off-ramp vehicles to be in lane 1.

The equations, as presented, are multiple linear regression, except for several, which are simple linear regression. There are several reasons for maintaining linear expressions. One reason is the relative ease of computation and of presenting the equation in nomographic form. Another reason, perhaps more important, was that transformation of variables yielded very little benefit as far as explaining variance and decreasing the standard errors. Most of the variables, such as freeway volume, are strongly linear in their relationship to lane 1 volume.

As a means to faster computation, nomographs have been constructed for each equation. These nomographs, Figures 1 through 17, have the same figure numbers as the corresponding equation numbers. Instructions are given on each nomograph, so confusion in their use should be at a minimum. Sample problems are worked on nomograph Figures 7 and 8. The nomographs should prove very useful in reverse order solution of problems, where the given lane 1 volume is set as a limit and allowable freeway or ramp volumes are desired. For instance, suppose a service volume of 1, 500 vph is specified for the lane 1 volume upstream from an off-ramp on 6-lane freeways and an off-ramp volume of 900 vph is forecast. What freeway volume can be accommodated approaching this off-ramp without exceeding the service volume in lane 1 immediately upstream from the ramp? Using a straightedge on nomograph Figure 12 and laying it across 900 on the X_7 scale and 1, 500 on the solution line, a freeway volume of 4, 100 (4, 108 by computation using Eq. 12) can be read off the X_1 scale. This perhaps illustrates the hazard of designing only by average lane volumes, because the average lane volume upstream from this off-ramp should not exceed $\frac{4,100}{3}$ or 1, 370 vph lane, if the lane 1 volume at the critical section just upstream from the off-ramp is not to exceed 1, 500 vph.

An average freeway lane volume of 1, 500 vph (4, 500 vph total) would result in a lane 1 volume of 1, 600 vph upstream from the off-ramp. On the other hand, if the off-ramp volume was 675 vph or less, there would be no trouble maintaining a lane 1 volume of 1, 500 vph at 4, 500-vph freeway volume. At a lower off-ramp volume of 400 vph and lane 1 service volume limit of 1, 500 vph, the nomograph indicates the freeway volume could be close to 5, 000 vph. The off-ramp section is not the critical factor in this case, but rather, lanes 2 and 3 are now running an average of 1, 750 vph each and the freeway is probably exceeding an average lane volume criterion.

It is rather interesting, turning to nomograph Figure 5 for off-ramps on 4-lane freeways, to compare similar values with the answers just obtained for 6-lane freeways. Using a lane 1 service volume of 1, 500 vph, and an off-ramp volume of 900 vph, a freeway volume of only 2, 500 vph or 1, 250-vph/lane average can be accommodated upstream from the off-ramp. This 1, 250-vph lane average compared to 1, 370-vph lane average for 6-lane freeways points out the lesser efficiency of 4-lane freeways. An average freeway lane volume of 1, 500 vph (3, 000 vph total) would result in a lane 1 volume estimate of 1, 650 vph, as compared to 1, 600 vph for 6-lane freeways.

On the 4-lane freeway, a lane 1 volume of 1, 500 vph at 3, 000-vph freeway volume can be maintained with a ramp volume of 575 vph. This compares with 675 vph for the ramp volume of 6-lane freeways under the same circumstances. However, reducing to a ramp volume of 400 vph, and a lane 1 limit of 1, 500 vph, the 4-lane freeway can handle 3, 250 vph upstream from the off-ramp. While this average lane volume is

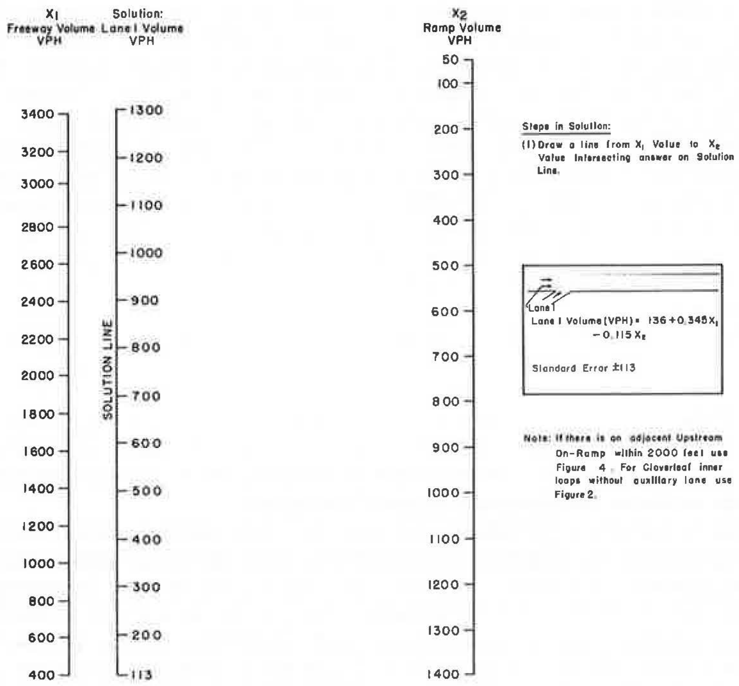


Figure 1. Nomograph for determination of lane 1 volume on 4-lane freeways for all locations except cloverleaf inner loops.

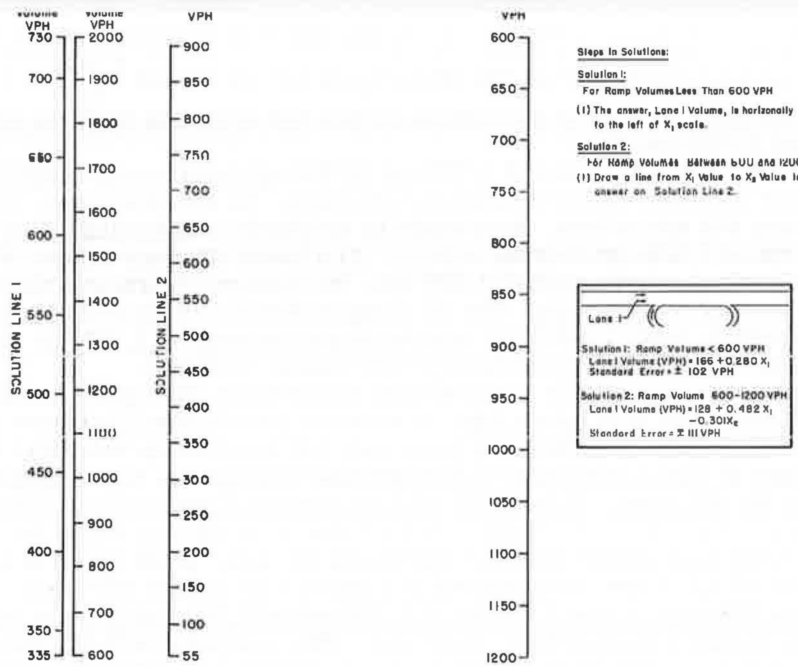


Figure 2. Nomograph for determination of lane 1 volume on 4-lane freeways with cloverleaf inner loops (no auxiliary lane).

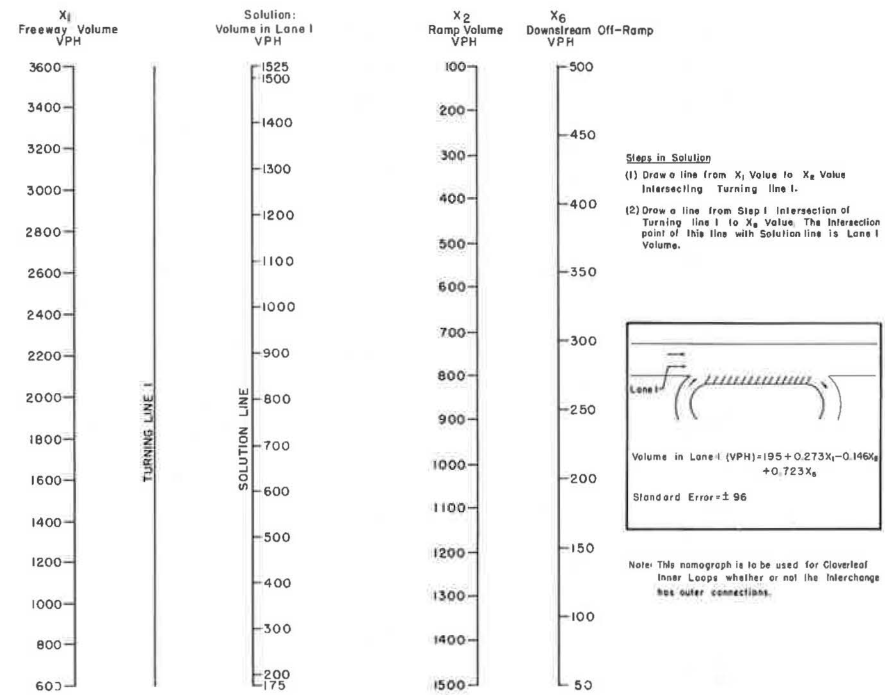


Figure 3. Nomograph for determination of lane 1 volume on 4-lane freeways at cloverleaf inner loops with auxiliary lane.

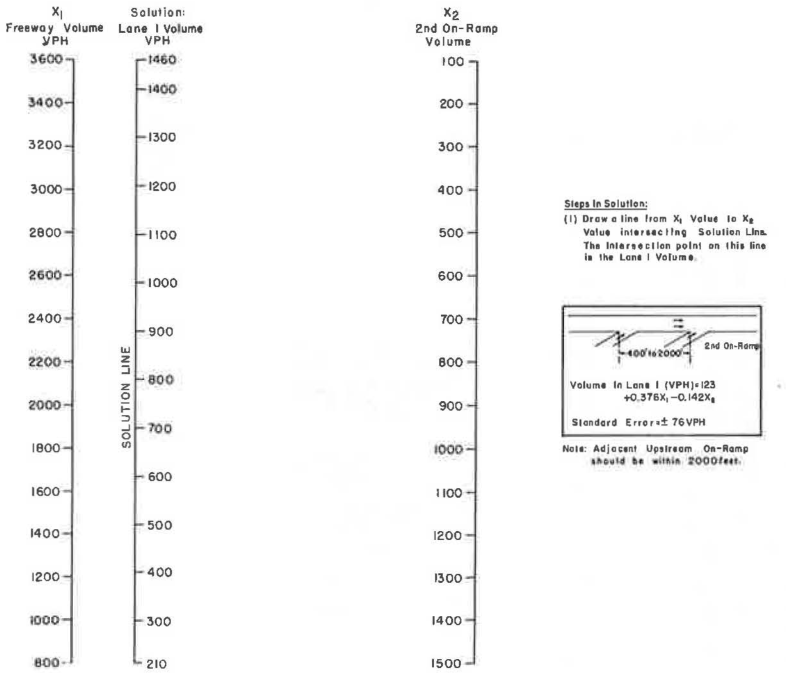


Figure 4. Nomograph for determination of lane 1 volume on 4-lane freeways at second of successive on-ramps.

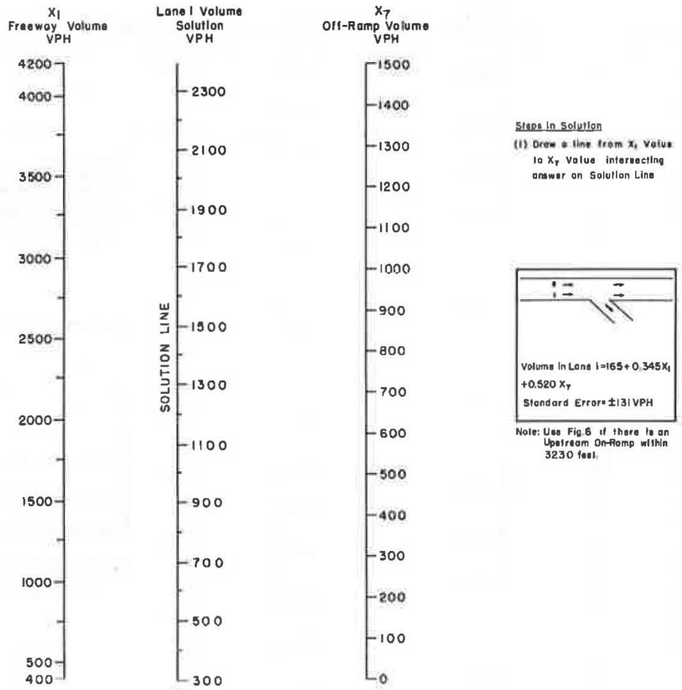


Figure 5. Nomograph for determination of lane 1 volume upstream from off-ramp on 4-lane freeways.

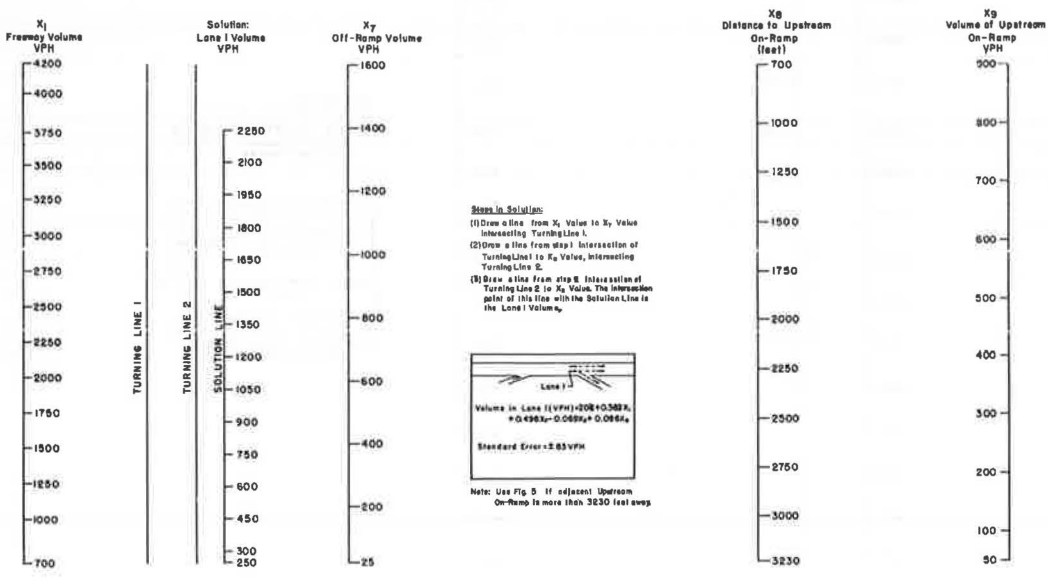


Figure 6. Nomograph for determination of lane 1 volume upstream from off-ramp on 4-lane freeways.

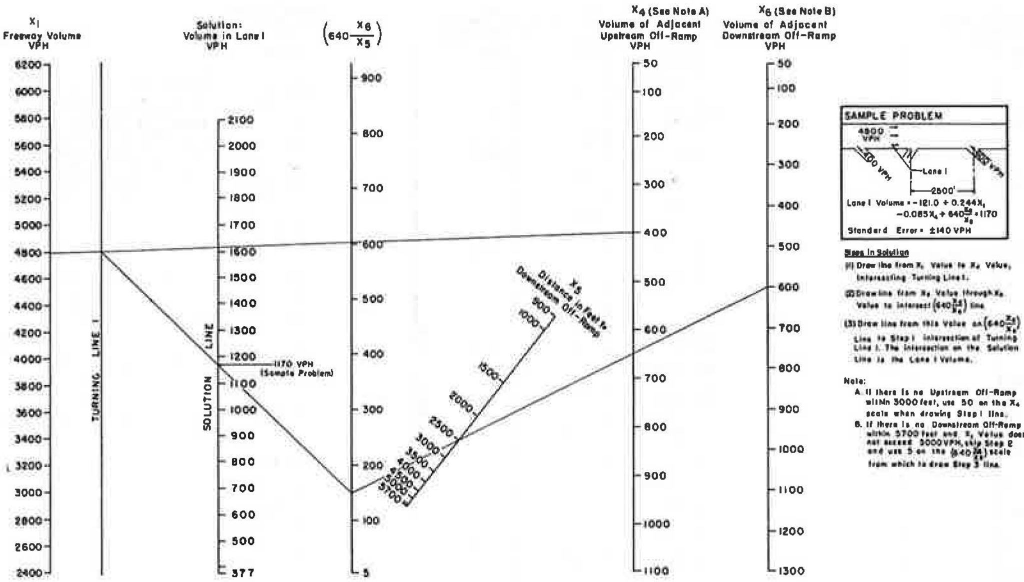


Figure 7. Nomograph for determination of lane 1 volume on 6-lane freeways.

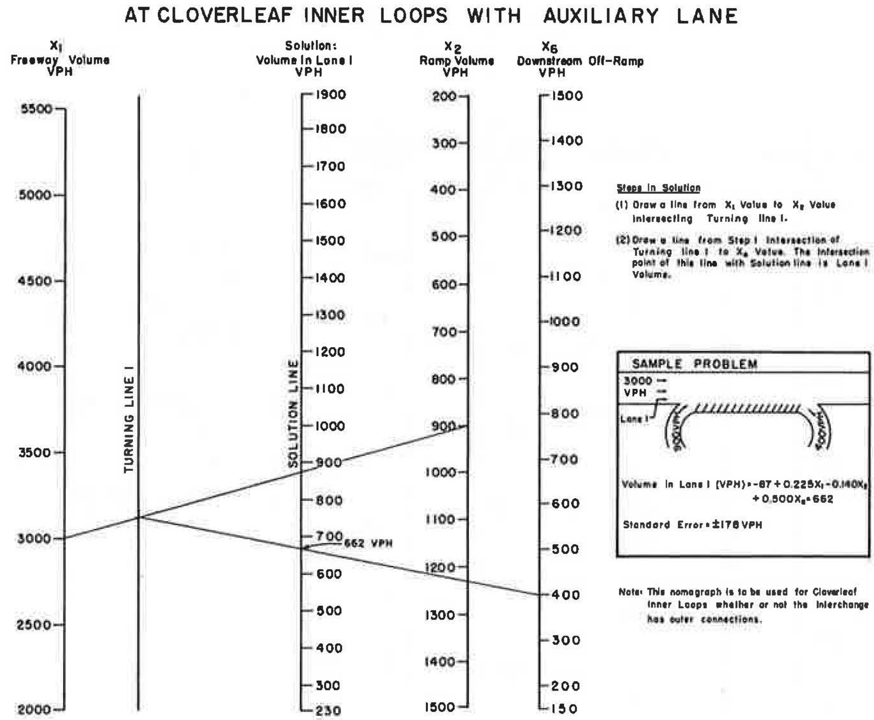


Figure 8. Nomograph for determination of lane 1 volume on 6-lane freeways at cloverleaf inner loops with auxiliary lane.

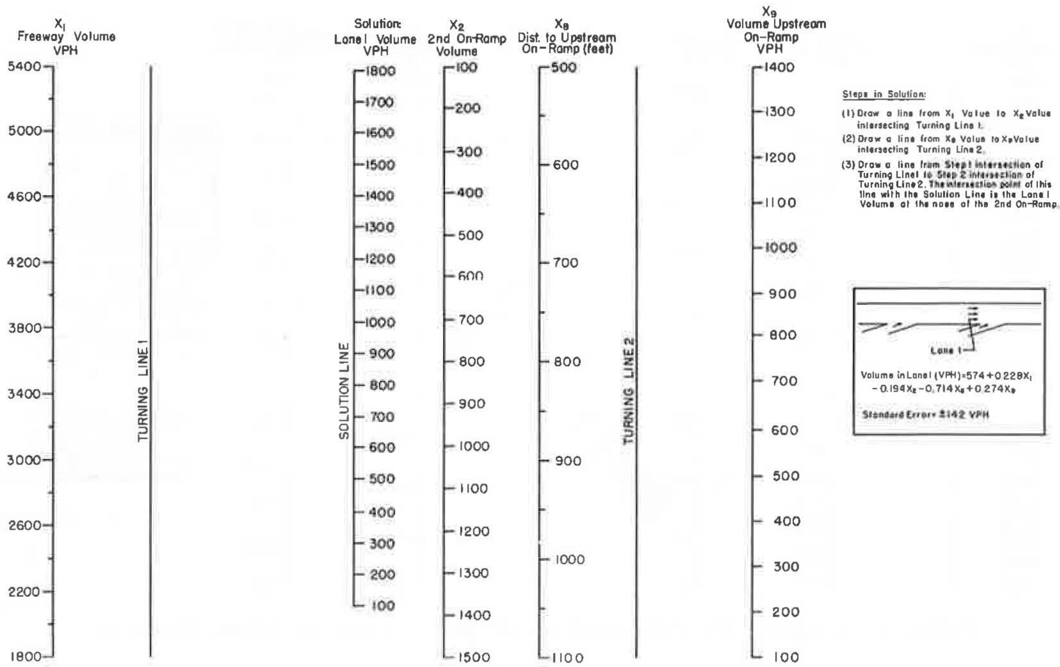


Figure 9. Nomograph for determinaton of lane 1 volume on 6-lane freeways at second of successive on-ramps.

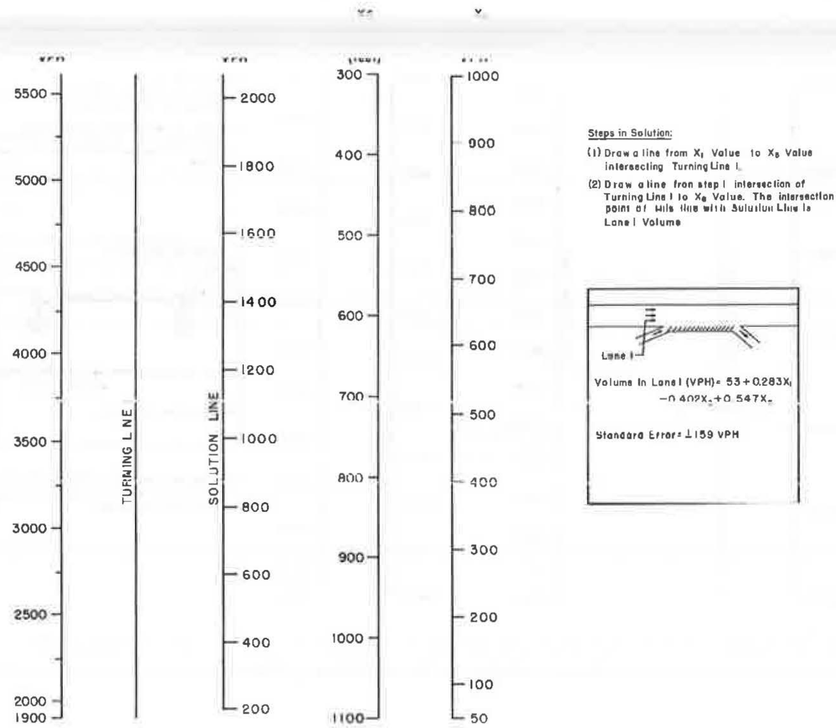


Figure 10. Nomograph for determinaton of lane 1 volume on 6-lane freeways where on-ramp has auxiliary lane.

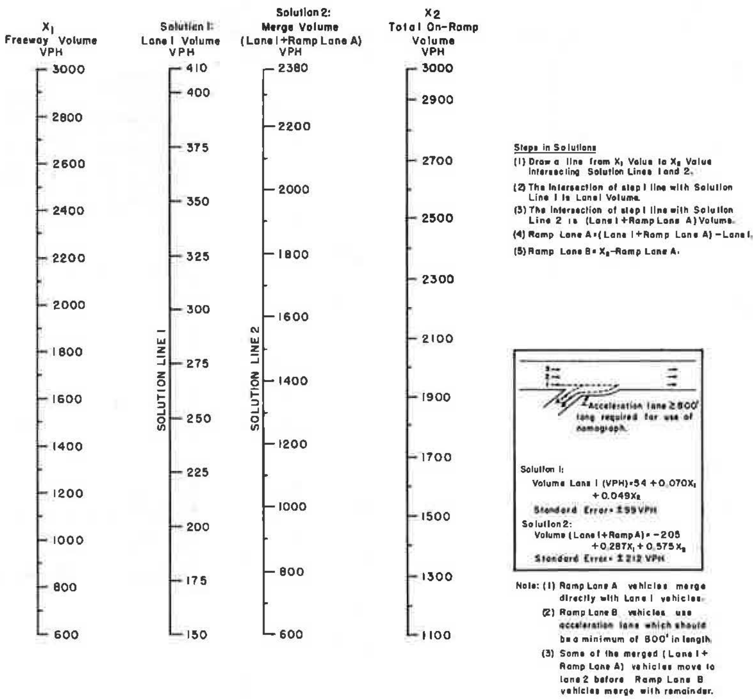


Figure 11. Nomograph for determination of lane distributions and merge on 6-lane freeways at 2-lane on-ramp.

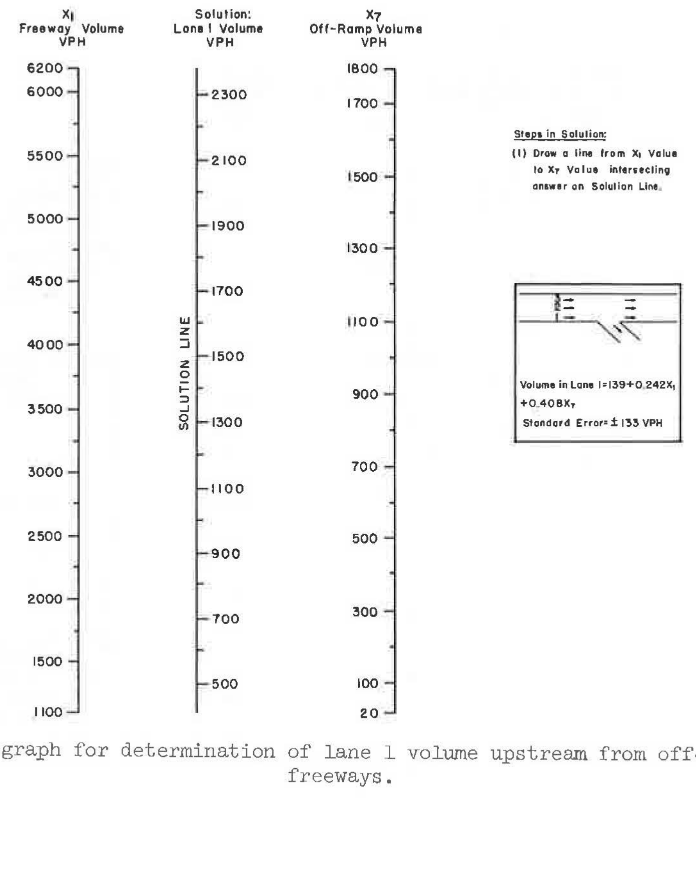


Figure 12. Nomograph for determination of lane 1 volume upstream from off-ramp on 6-lane freeways.

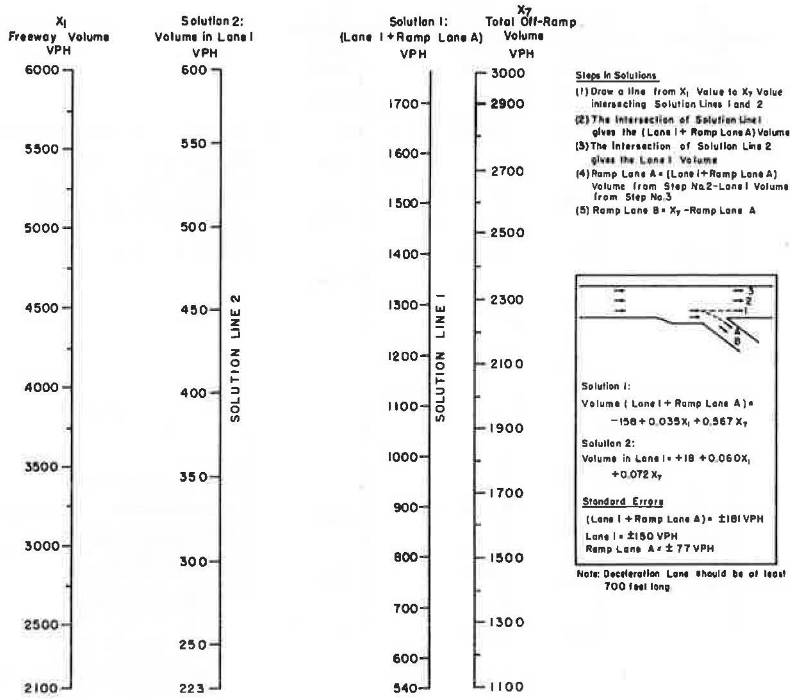


Figure 13. Nomograph for determination of lane volume distributions approaching the nose of a 2-lane off-ramp with deceleration lane.

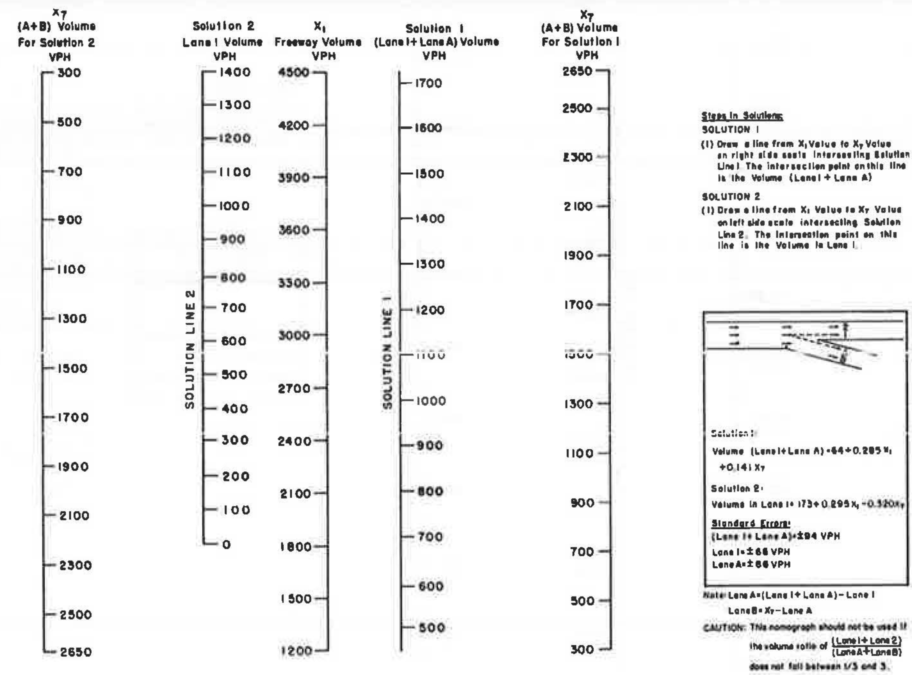


Figure 14. Nomograph for determination of major fork lane volume distribution.

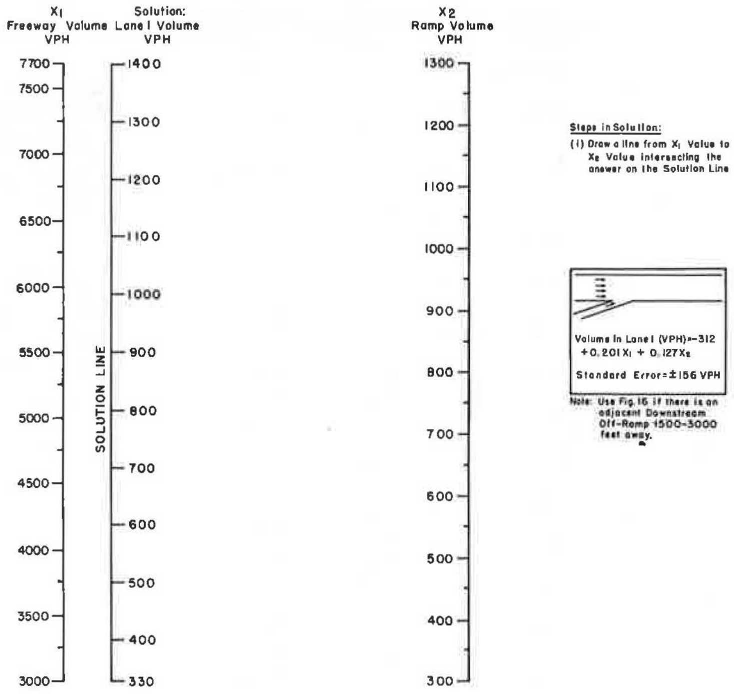


Figure 15. Nomograph for determination of lane 1 volume on 8-lane freeways.

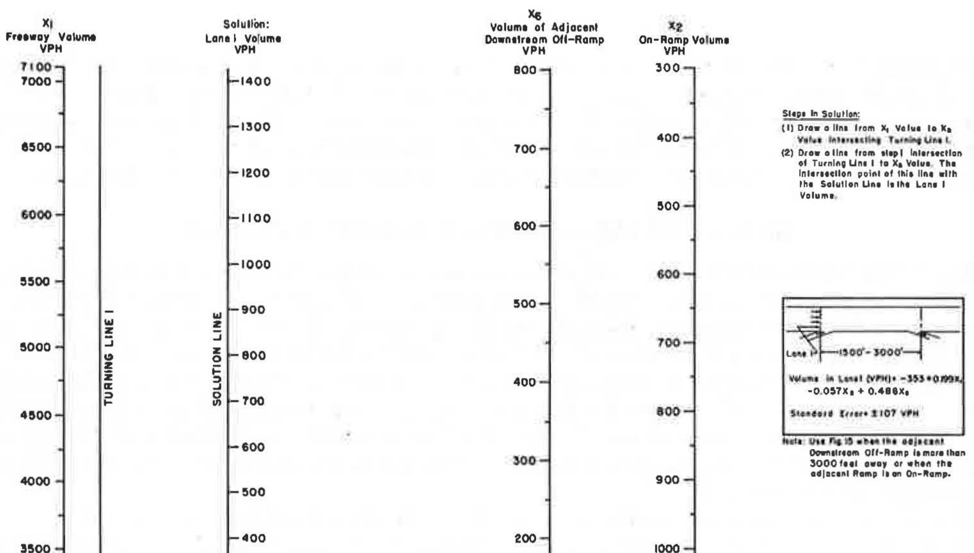


Figure 16. Nomograph for determination of lane 1 volume on 8-lane freeways where there is an adjacent off-ramp 1,500 to 3,000 ft downstream.

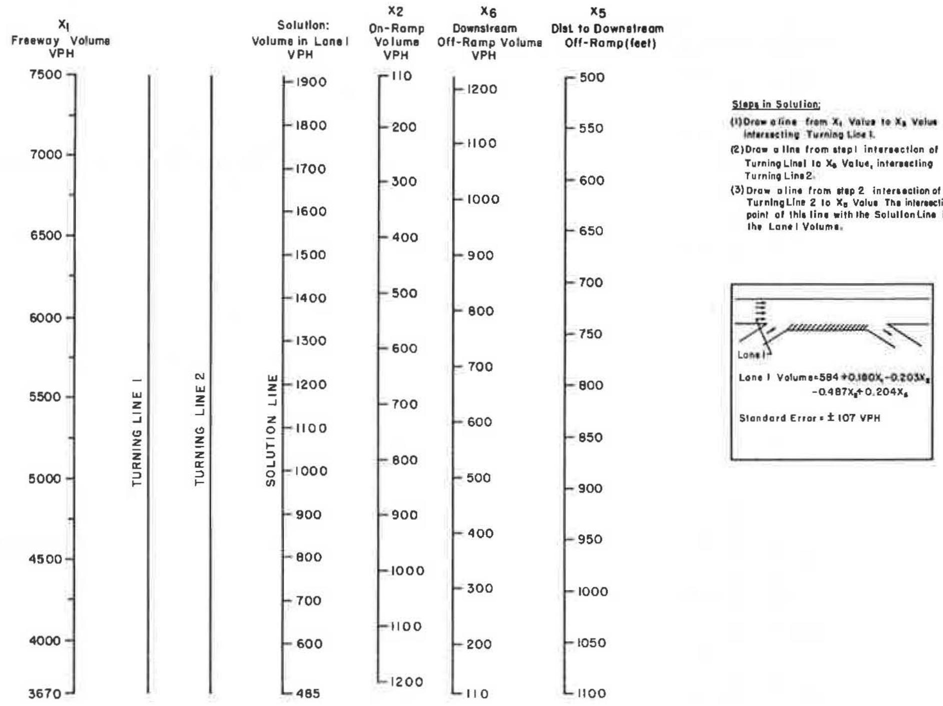


Figure 17. Nomograph for determination of lane 1 volume on 8-lane freeways; on-ramp has auxiliary lane.

lane (lane 2) on the 4-lane freeway is now carrying 1, 750 vph, or the same as the average lane volume of lanes 2 and 3 for a 6-lane freeway. Thus, it appears that as long as off-ramp volumes can be kept low in relation to through volumes, a 4-lane freeway can handle off-ramp movements about as efficiently as a 6-lane freeway.

OFF-RAMP EQUATIONS FOR 4-LANE FREEWAYS

Eq. 5 for calculating lane 1 volume upstream from an off-ramp on 4-lane freeways was presented earlier in this report. This equation, $Y = 165 + 0.345X_1 + 0.520X_7$, comes from 518 observations, taken from 16 locations. These locations include: two from the North Sacramento Freeway; one from I-280 near Santa Clara, Calif.; two from the Tri-State Tollway in Indiana; one from the Muncie Freeway in Kansas City, Kans.; two from the Baltimore-Washington Parkway; one from I-96 near Detroit, Mich.; one from Route 17 at Binghamton, N. Y.; one from Cross County Parkway, Poughkeepsie, N. Y.; two from the Schuylkill Expressway in Philadelphia; and one each from San Antonio and Austin, Tex.

The large number of observations made possible the stratification of the data by percentages of ramp vehicles in the freeway stream. These percentages of ramp vehicles in the freeway stream were taken as follows: 0-9.9 percent, 10.0-19.9 percent, 20.0-29.9 percent, 30.0-39.9 percent, and 40.0-70.0 percent. The last group includes a wider percentage spread of ramp vehicles, as there were too few observations to continue by increments of ten percentage points once the ramp traffic reached 40.0 percent of the freeway volume.

Lane 1 vs freeway volume was plotted for each ramp percentage group, and a simple linear regression line fitted by computer. In addition, multiple regression equations were computed using, in addition to the freeway volume, X_1, the off-ramp volume, X_7,

TABLE 5

FORMULAS FOR LANE 1 VOLUME UPSTREAM FROM OFF-RAMPS ON 4-LANE FREEWAYS CLASSIFIED BY PERCENTAGE OF OFF-RAMP VEHICLES IN FREEWAY STREAM

Ramp Volume / Freeway Volume	Simple Regression Equation	Multiple Regression Equation	Standard Errors (v.p.h.) S.R.E.*	M.R.E.+	r^2 S.R.E.	R^2 M.R.E.	Number of Locations	Number of Observations
0 - 9.9%	$Y = 230 + 0.355X_1$	$Y = 260 + 0.442X_1 - 1.672X_7$	189	170	.70	.76	8	79
10.0 - 19.9%	$Y = 137 + 0.443X_1$	$Y = 130 + 0.240X_1 + 1.346X_7$	163	144	.79	.84	13	124
20.0 - 29.9%	$Y = 166 + 0.474X_1$	$Y = 168 + 0.372X_1 + 0.399X_7$	93	91	.96	.96	16	158
30.0 - 39.9%	$Y = 181 + 0.496X_1$	$Y = 180 + 0.473X_1 + 0.068X_7$	88	88	.96	.96	12	95
40.0 - 70.0%	$Y = 147 + 0.649X_1$	$Y = 120 + 0.242X_1 + 0.797X_7$	154	97	.88	.95	9	62
Equation No. 5 (for 518 observations)		$Y = 165 + 0.345X_1 + 0.520X_7$		131		.94		518

* S.R.E. = Simple Regression Equation.
\+ M.R.E. = Multiple Regression Equation.

as a variable. Table 5 contains the series of equations with their associated standard errors and coefficients of determination. The use of a multiple regression equation results in a considerable improvement over the simple regression in categories containing 0-9.9 percent, 10.0-19.9 percent, and 40.0-70.0 percent ramp vehicles in the freeway stream. The middle range categories, 20.0-29-9 percent and 30.0-39.9 percent, were very slightly improved by the use of the off-ramp as a variable. Of course, inasmuch as the ramp volume is already stratified, its effect as a range within a group is not so discernible. Figure 18 is a graphical representation of lane 1 volume for the stratified ramp volume groups as related to freeway volume upstream from the off-ramp.

Some possible explanations can be offered for the improvement in the multiple regressions of the equations for the lower ramp volume percentages. One reason might be because these categories are made up predominately of freeway vehicles having a free choice of two lanes to use. Any ramp vehicles in the stream would have to be in lane 1, so their inclusion as a variable results in a better fit. On the other extreme, for the high volume ramp category (40.0-70.0 percent in the freeway stream), there is so much difference in ramp volumes between observations that their inclusion as a variable is a necessity for adequate accuracy. Because the freeway volume is a much smaller percentage of the total volume, lane 1 volume is not so precisely determined by using only the freeway volume as a variable as in the simple regression.

It is rather interesting to note, logically enough, that lane 1 volume is more unpredictable at low ramp volumes.

Considering other possible variables, it was hypothesized that an increasing magnitude of the percentage of commercial vehicles in the diverge might have a negative effect on the lane 1 volume. Even though the commercial vehicle percentages for short periods ranged from 0 (on the parkway locations) to 56 percent, there was no discernible effect on lane 1 volume. However, nearly all of the locations were either flat grade or close to flat. Observations of freeway grades which severly reduce truck speeds lead to the conclusion that there is a substantial vacating of lane 1 by passenger cars at such locations.

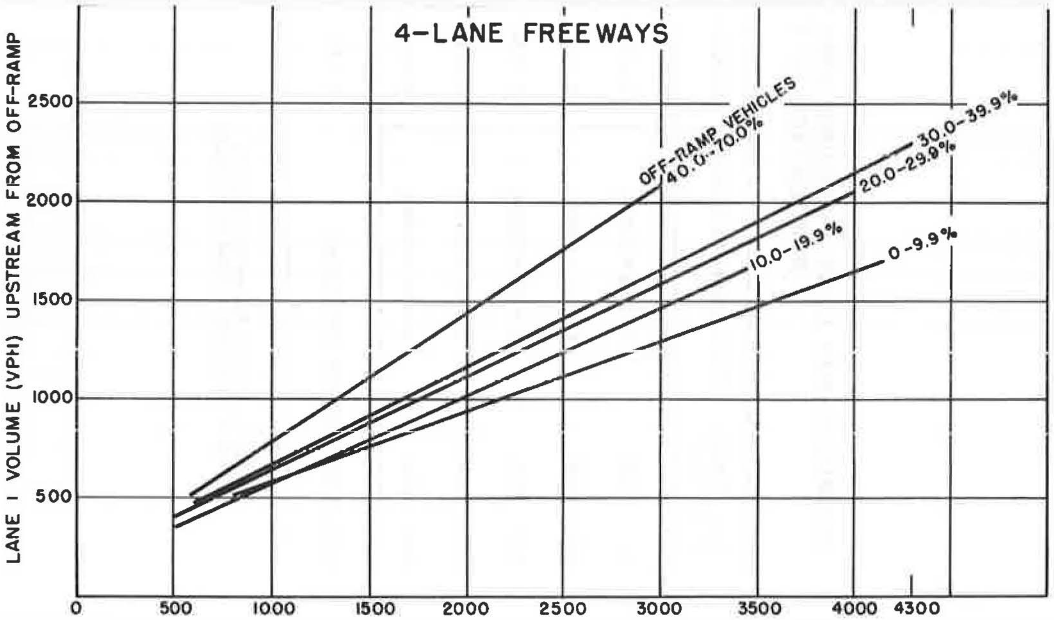


Figure 18. Lane 1 volume upstream from off-ramp related to freeway volume and percentage of off-ramp vehicles in freeway stream upstream from off-ramp.

AUXILIARY LANE USAGE

Eqs. 3, 8, 10, and 17, were developed for on-ramp locations having an auxiliary lane extending to the adjacent downstream off-ramp. The presence of an auxiliary lane changes the computational procedures somewhat from that used in conventional merging and diverging situations. Conventionally, the computed lane 1 volume is added to the ramp volume to get the "expected merge" or in case of an off-ramp, the lane 1 volume upstream from an off-ramp is the "expected diverge." At auxiliary lane locations, however, the opportunity to weave or change lanes between lane 1 and the auxiliary lane makes necessary a computation of volume in each of these lanes at selected points between the ramp noses. The computed lane 1 and auxiliary lane volumes should be checked against the service volume. In making an "average lane volume" check across the lanes, the auxiliary lane should not be counted as a lane and the volume on the auxiliary lane should not be included in the volume used in the numerator of an "average lane volume" calculation.

Figure 19 is the tool used in conjunction with the equations in the analysis of auxiliary lane locations. Its use is intended for auxiliary lanes of 1,100 feet in length or less, as covered by the lane 1 volume equations (Eqs. 3, 8, 10, and 17). Lack of data prevented the development of curves for longer auxiliary lanes. The computational procedure used is as follows:

1. Determine lane 1 volume at the on-ramp nose by the use of the proper equation (Eqs. 3, 8, 10 or 17) or nomograph of the same figure number. This lane 1 volume will consist of lane 1 through vehicles and of off-ramp vehicles intending to exit. For reasons of simplicity in figuring the lane 1 through volume, 100 percent of the "intending to exit" off-ramp vehicles are considered to be in lane 1 at the on-ramp nose. In practice, this is more apt to be approximately 95 percent, as there are always a few "late decision," "blocked off," or "sleeping" prospective off-ramp drivers, who are still in lane 2 at the on-ramp nose.

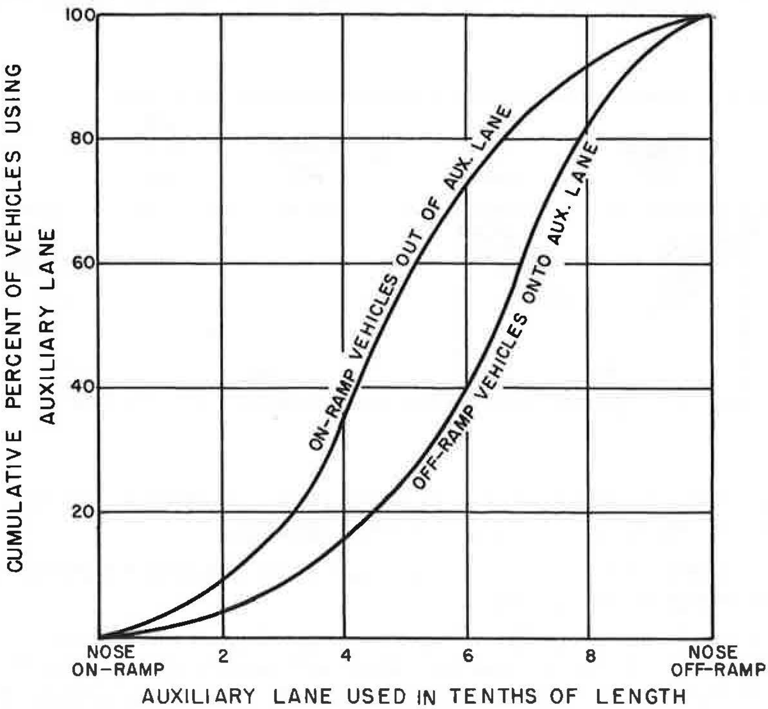


Figure 19. Usage of auxiliary lane by entering and exiting vehicles.

2. Subtract the off-ramp volume from the computed lane 1 volume to get the lane 1 through volume.

3. Make several checks of lane 1 and auxiliary lane volumes at points between the ramps. The volumes will consist of:

Lane 1 volume = lane 1 through + on-ramp vehicles out of auxiliary lane (Fig. 19, upper curve) + off-ramp vehicles still in lane 1 (Fig. 19)

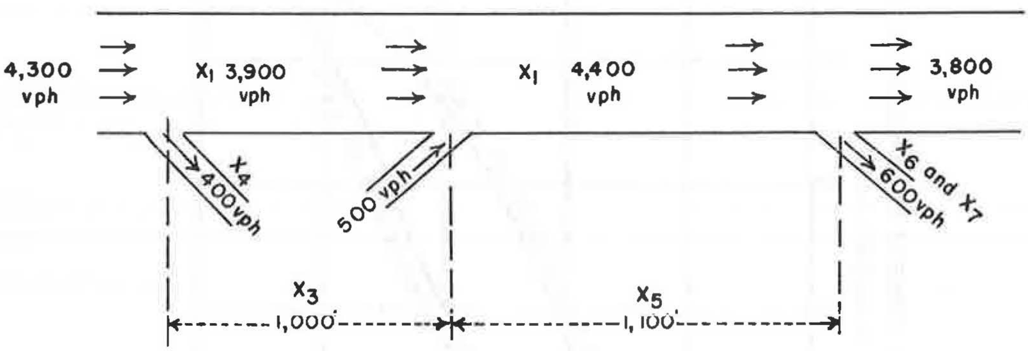
Auxiliary lane volume = on-ramp vehicles still in auxiliary lane (Fig. 19) + off-ramp vehicles who have moved onto the auxiliary lane (Fig. 19, lower curve).

Because lane 1 carries through vehicles as well as ramp vehicles, it seems obvious that it will usually be the critical lane for overloading as compared to the auxiliary lane which carries ramp vehicles only. The most critical checkpoint between the ramps can usually be ascertained by noting the relative ramp volumes and the shape of the curves in Figure 19. An examination of the upper curve in Figure 19 discloses there is increased movement by on-ramp vehicles from the auxiliary lane to lane 1 between the 0.3 and 0.6 distance points. Also, off-ramp vehicles tend to stay in lane 1 until the 0.5 point after which increased movement onto the auxiliary lane takes place up to the 0.8 distance point.

The foregoing suggests that the most heavily trafficked portion of lane 1 is the section from 0.5 to 0.6 of the distance along the auxiliary lane. Usually a volume check at the 0.5 point will suffice unless ramp volumes are quite high. Should the off-ramp volume be comparatively high, the lane 1 section just downstream from the on-ramp nose should be checked for overloading, say at the 0.2 point. On the other hand, if the on-ramp volume is comparatively high, a volume check of lane 1 should be made just upstream from the off-ramp, at possibly the 0.8 point.

SAMPLE PROBLEM

Given: A proposed design for a 6-lane freeway with the following traffic volumes:



The chosen service volumes not to be exceeded are 1,500-vph/lane average across the freeway lanes; 1,500-vph merge; and 1,600-vph diverge.

Find: If the design is adequate for average lane volume, merge volume at the on-ramp, and diverge volumes at the off-ramps.

Solution: The equations applicable to this layout are Eqs. 7 and 12, or, if nomographic solutions are desired, Figures 7 and 12. It is obvious by inspection that the average lane volume is less than 1,500 vph/lane at between interchange sections. The most critical diverge would be that at the second off-ramp, because its volume is 600 vph, as compared to 400 vph at the first off-ramp. Also, freeway volume is 4,400 vph at the second off-ramp as compared to 4,300 vph at the first off-ramp.

Using Eq. 12 to check the diverge:

$$Y = 139 + 0.242X_1 + 0.408X_7$$
$$= 139 + 0.242(4400) + 0.408(600)$$
$$= 1,449 \text{ vph in lane 1 upstream from the second off-ramp}$$

1, 449 vph < 1, 600 vph, so the diverge meets service volume specifications.

Using Eq. 7 to check the merge:

$$Y = -121 + 0.244X_1 - 0.085X_4 + 640 \frac{X_6}{X_5}$$
$$= -121 + 0.244(3900) - 0.0085(400) + 640 \left(\frac{600}{1,100}\right)$$
$$= 1,146 \text{ vph in lane 1 at the on-ramp nose}$$

Expected merge = 1, 146 vph (lane 1) + 500 vph (ramp) = 1, 646 vph

1, 646 vph > 1, 500 vph, so the merge exceeds the service volume specification and requires an alteration in design.

To meet service volume specifications at the on-ramp merge, the distance to the downstream off-ramp could be increased or an auxiliary lane could be added between the ramp noses. If consideration is given to increased distance between ramps as a remedy, using Eq. 7 and solving for the X_5 distance value needed for a Y value of 1, 500 vph, a distance of 1, 892 feet is computed. This means an additional 792 feet between ramps (1, 892 - 1, 100 = 792) is needed to reduce the expected merge from 1, 646 vph to 1, 500 vph. Assuming, as will often be the case, that this method of solution is not feasible or most feasible, an auxiliary lane could be added between the ramps as a means of increasing capacity so that the service volume specification can be met. Because there is a somewhat changed lane volume distribution at auxiliary lane locations, Eq. 10, or Figure 10, would be used, instead of Eq. 7, to determine the lane 1 volume at the on-ramp nose.

$$Y = 53 + 0.283X_1 - 0.402X_5 + 0.547X_6$$
$$= 53 + 0.283(3900) - 0.402(1100) + 0.547(600)$$
$$= 1,043 \text{ vph in lane 1 at the on-ramp nose}$$

At first inspection, it would appear that this 1, 043 vph would be added to the 500-vph ramp volume and the expected merge would still exceed 1, 500 vph. However, with the addition of an auxiliary lane, this merge no longer takes place. Rather, there is a weaving action on lane 1 and the auxiliary lane between the ramp noses. This action involves the following volumes for lane 1: (1) through lane 1 vehicles, (2) on-ramp vehicles which have moved off the auxiliary lane, and (3) off-ramp vehicles which have not yet moved onto the auxiliary lane prior to exiting. For the auxiliary lane, the volumes include: (1) on-ramp vehicles which have not yet moved into lane 1, and (2) off-ramp vehicles which have moved off lane 1 onto the auxiliary lane, prior to exiting.

Figure 19 is used to determine the lane 1 and auxiliary lane volumes at selected points between the ramp noses. Because ramp volumes are not high in this problem, it will be sufficient to check volumes at $^5/_{10}$ of the distance along the auxiliary lane. Additional checks should be made at the 0. 2 and 0. 8 points between the ramps if the combined volume of lane 1 (computed by Eq. 10) and the on-ramp exceeds 150 percent of the service volume level for one lane.

Lane 1 Volume Calculation:

Lane 1 through vehicles = 1, 043 (lane 1) - 600 vph (off-ramp vehicles)
= 443 vph (all off-ramp vehicles are assumed to be in lane 1 at the on-ramp nose)

On-ramp vehicles in lane 1 at 0. 5 point = 0. 58 × 500 vph = 290 vph

Off-ramp vehicles in lane 1 at 0. 5 point = (1. 00 - 0. 25) × 600 vph = 450 vph

Total vph in lane 1 at 0.5 point = 443 through + 290 on-ramp + 450 off-ramp
= 1,183 vph
1,183 < 1,500, so this meets the service volume specifications.

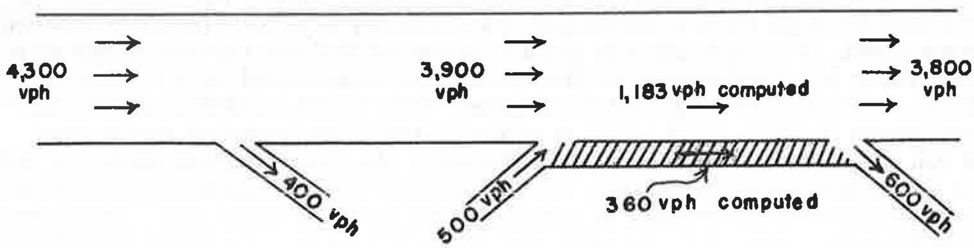
Auxiliary Lane Volume Calculation:

On-ramp vehicles in auxiliary lane at 0.5 point = (1.00 - 0.58) × 500 vph = 210 vph
Off-ramp vehicles in auxiliary lane at 0.5 point = 0.25 × 600 vph = 150 vph
Total vph in auxiliary lane at 0.5 point = 210 on-ramp + 150 off-ramp = 360 vph
360 < 1,500, so this meets the service volume specification.

Average Lane Volume Calculation:

At 0.5 point, volume on the three through freeway lanes will be
3,900 + 500 - 360 (on aux. lane) = 4,040 vph
4,040/3 = 1,347 vph/lane which is satisfactory

Our final design diagram is now satisfactory and looks like this:



given distance, as a design criterion. If so, this factor should also be checked.

2-LANE ON-RAMPS

The use of 2-lane ramps is sometimes necessary at high volume connections, such as might be found at a freeway-to-freeway interchange. At present, specific warrants for the use of 2-lane ramps are lacking. In the case of a right-hand, 2-lane on-ramp, it is the usual recommendation that one ramp, usually ramp lane B (by this nomenclature), be continued on as an added lane to the freeway. Ramp lane A vehicles merge directly with lane 1 vehicles of the freeway. An alternative to this design is to have ramp lane A add a lane to the freeway and have ramp lane B merge into ramp lane A, now lane 1 of the freeway. Unfortunately, field data are lacking for both these designs, so comparative evaluations cannot be made at this time.

The 2-lane, on-ramp data herein were taken from 6-lane freeway locations on Long Island. In contrast to the designs mentioned above, there was no freeway lane added at these locations. While this type of design is still employed occasionally, it is not generally recommended unless the freeway volume is low-to-moderate and forecasts are that the freeway volume will remain at this lower volume level. For this design, ramp lane A vehicles merge directly into lane 1, while ramp lane B vehicles use an acceleration lane before merging with the already-merge (lane 1 + ramp lane A) vehicles. Some of the initially-merged vehicles move over into lane 2, while still adjacent to the acceleration lane, thus opening up some gaps for the ramp lane B vehicles to utilize. Eqs. 11a and 11b and nomograph Figure 11 were developed from data collected in five studies made at the two locations on Long Island—the Southern Parkway (North Conduit Avenue, service road) ramp to Van Wyck Expressway northbound and the Northern State Parkway ramp to the Long Island Expressway westbound. The length of acceleration lane (810 feet) is hardly considered adequate by present standards, but the equations should give a fairly accurate representation of lane volume distribution, even for more liberal acceleration lane designs.

The important consideration in this type of design is that there are, in effect, two merges, not easily distinguishable to an observer of the scene, but there, nevertheless. The first merge, that of ramp lane A with lane 1, can be most easily quantified and is the likeliest candidate for a service volume control. Because these ramp vehicles (and the merge is made up mainly of ramp vehicles) will be merging directly or with slight usage of the tapered apex, this merge volume should not exceed 1, 500 vph and, perhaps, should be less if relatively smooth operation is desired. The second merge is that of ramp lane B vehicles which have utilized (partially or fully) the acceleration lane before merging with the already-merged (lane 1 + ramp lane A) vehicles. This merge cannot be easily quantified, because a number of the already-merged vehicles move over into lane 2, while still parallel to the acceleration lane. This "moved out" volume is likely to be higher when freeway volumes are low and/or ramp volumes are high, because there will be more gaps in lane 2 and/or more pressure exerted by ramp vehicles to move out.

Using a straightedge on nomograph Figure 11, if a 1, 500-vph service volume is set for the merge of lane 1 and ramp lane A and a ramp volume of 1, 500 vph is forecast, the freeway volume approaching the merge should not exceed 2, 940 vph. The downstream (after merge) volume is 4, 440 vph (2, 940 + 1, 500). At progressively higher ramp volumes and correspondingly lower freeway volumes, a markedly lower downstream (after merge) freeway volume would result. For instance, at a ramp volume of 2, 200 vph, only 1, 520 vph freeway volume could be present if the 1, 500-vph merge criterion of lane 1 and ramp lane A is set as a desirable service volume limit. Thus, the downstream freeway volume could only be 3, 720 vph (1, 520 + 2, 200). There would be 270 lane 1 vehicles and 1, 230 ramp lane A vehicles making up the initial merge, leaving 970 ramp lane B vehicles to merge subsequently along the length of the acceleration lane. This does not appear to be a desirable operation, because, in effect, 970 vehicles would have to vacate lane 1 along the length of the acceleration lane to maintain a 1, 500-vph merge. This merge would include the ramp lane B vehicles, and the leftover lane 1 and ramp lane A vehicles.

If the acceleration lane were increased to, say, 1, 500-1, 600 feet in length, this possibly would attract more vehicles to ramp lane B, reducing ramp lane A and the initial merge. The capacity, therefore, conceivably could be increased somewhat. Obviously, an even better way to increase capacity would be to continue ramp lane B on as an added freeway lane. In this case, ramp lane A would carry little traffic until ramp lane B volume was at least up to practical capacity.

Eqs. 11a and 11b were derived from one hundred and fifteen 5-min hourly rates, or a total of almost 10 hours of operation spread over five separate studies, conducted over a several months period. As an illustration of why the equation standard errors might be larger than they need be to apply accurately to full-hour volumes, Table 6 is shown with the four peak hours for the N. Y. -36 studies and two 1-hr volumes for the N. Y. -23 study. All computed volumes, using as independent variables the actual freeway and ramp volumes, fall within one standard error of the actual component volumes. Except for some congestion in N. Y. -36O and N. Y. -36D studies, the traffic operation could generally be considered free-flowing.

A final important note. There were no weaves of any consequence for at least several miles downstream from these ramps. Consequently, there was little need for freeway vehicles to be in lane 1 at the ramp nose. Also, because of the intense pressure of 2-lane on-ramps, the average driver shies away from lane 1. However, 2-lane ramps are likely to be found leading into major weaving areas. If so, lane 1 could be loaded up with prospective weave vehicles and distributions altered greatly. This situation should be treated as a weaving problem and Eqs. 11a and 11b would not be applicable.

TWO-LANE OFF-RAMPS

Eqs. 13a and 13b and nomograph Figure 13 apply to a 2-lane off-ramp on a 6-lane freeway. Though only one location was used in the collection of data, two morning and

TABLE 6

COMPUTED HOUR VOLUMES VS ACTUAL [illegible]JR VOLUMES FOR 2-LANE ON-RAMPS ON 6-LANE FREEWAYS USI[illegible] FORMULAS 11a AND 11b

Study Number	Time of Study	Actual Fwy. Vol. v.p.h.	Actual Ramp Vol. v.p.h.	Actual Lane 1	Computed Lane 1	[illegible]	Actual Ramp Lane A	Computed Ramp Lane A	Error Ramp Lane A	Actual Merge (Ramp Lane A + Lane 1)	Computed Merge (Ramp Lane A + Lane 1	Error Merge (Ramp Lane A + Lane 1)	Actual Ramp Lane B	Computed Ramp Lane B	Error Ramp Lane B
N.Y.-36A	A.M. PK	1526	2040	183	242	[illegible]	1279	1161	- 118	1462	1403	- 59	761	879	+ 118
N.Y.-36B	P.M. PK	964	2185	168	201	[illegible]	957	1126	+ 169	1145	1327	+ 182	1228	1059	- 169
N.Y.-36C	A.M. PK	2735	2303	295	337	[illegible]	1495	1568	+ 73	1790	1905	+ 115	808	735	- 73
N.Y.-36D	P.M. PK	1949	2265	272	277	[illegible]	1460	1380	- 80	1732	1657	- 75	805	885	+ 80
N.Y.-23	6:50-7:50 A.M.	1267	2303	244	256	[illegible]	1207	1226	+ 19	1451	1482	+ 31	1096	1077	- 19
N.Y.-23	7:50-8:50 A.M.	1685	2483	336	294	[illegible]	1336	1412	+ 76	1743	1706	- 37	1147	1071	- 76

Notes: The "Error" is (Computed - Actual) for each c[illegible]ry.
N.Y.-36 Studies are at the Northern State Par[illegible] ramp to Long Island Expressway, Westbound.
N.Y.-23 Study is at the Southern Parkway ramp [illegible]e Van Wyck Expressway, Northbound.

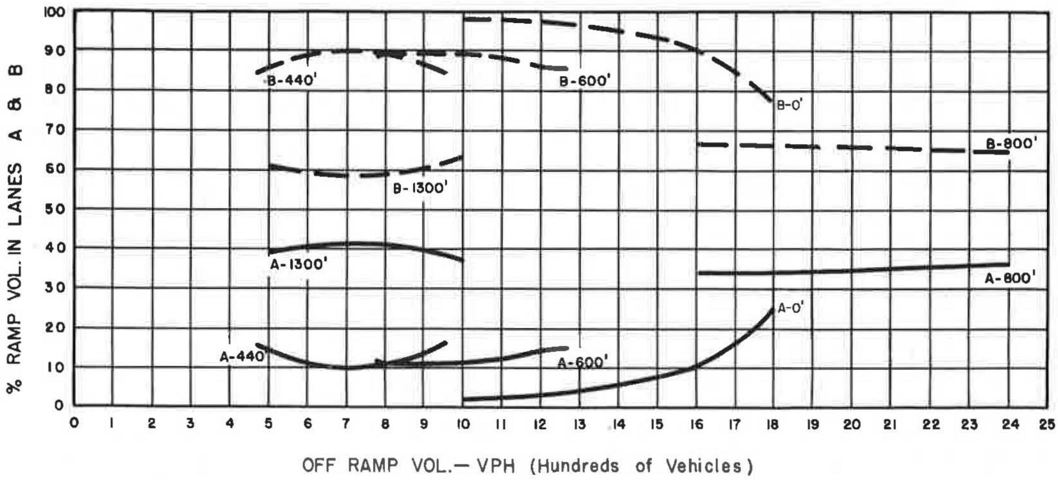


Figure 20. Volume distribution on the ramp for 2-lane off-ramps segregated by length of deceleration lane.

two afternoon studies were made over a wide range of volumes. The ramp studied was the Long Island Expressway eastbound ramp to Northern State Parkway eastbound. This ramp has an 800-ft long parallel deceleration lane. It is paramount that an adequate deceleration lane be provided if the ramp's capacity is to be utilized. Without a deceleration lane, drivers will use ramp lane B almost exclusively or make hazardous maneuvers to get to ramp lane A. Under these circumstances, a driver using ramp lane A must be apprehensive about being hit by through vehicles. He cannot distinguish which vehicles behind him will be using the ramp or continuing on through on the freeway. Moreover, at high ramp volumes, lane 1 is jammed with prospective ramp lane B vehicles. The temptation is for "blocked off" or impatient drivers to try to exit on ramp lane A from lane 2 of the freeway. Several of the 2-lane off-ramps studied nationally were of this type. There is no logic in developing equations for their lane distributions, as this would appear to be a tacit approval of their design. Suffice to say that their operation was poor and that nearly all ramp vehicles used ramp lane B. Approximately 5 percent of the vehicles used ramp lane A, up to the point where ramp lane B exceeded practical capacity. The use of ramp lane A necessarily increased at higher volumes, with resultant hazardous operation. Figure 20 shows some ramp lane volume distributions segregated by length of deceleration lane.

With the layout shown in nomograph Figure 13, the diverge volume of lane 1 and ramp lane A can be used as the service volume control. A check can also be made of ramp lane B volume, if so desired.

MAJOR FORKS

Much needs to be known about major fork operation and Eqs. 14a and 14b, and corresponding nomograph Figure 14, are only starters. The quality of operation of a major fork is very much dependent on the bifurcation treatment, the lane striping, the visibility (both day and night), and the signing employed. This is an area of study which has been neglected researchwise, probably because of the difficulty in gathering traffic movement data over such a wide expanse of roadway.

The layout shown in nomograph Figure 14 is simply a 3-lane (one-way) roadway, splitting into two, 2-lane roadways. All four lanes can be checked for volumes. Lane 1 and lane B will carry the bulk of the respective movements. The (lane 1 + lane A) volume, which must diverge, is the most likely service volume control.

The equations are not applicable if the leg volumes differ greatly—specifically, one roadway should not carry more than three times as much traffic as the other.

RAMP VEHICLE LANE USAGE DOWNSTREAM AFTER MERGING

Much of the preceding discussion in this report has elaborated on lane volume distributions at critical freeway sections for various ramp types and sequences, freeway and ramp volumes, and distances between ramps. The philosophy is that if the layout is kept at or below a specified service volume for critical sections, the overall design and/or operation will be balanced and relatively free of bottlenecks. The forecasting of traffic volumes has not yet reached a desirable level of accuracy and a ramp or freeway section can be overloaded shortly after the ribbons are cut. Allowance should be made, if feasible, for possible forecasting errors or unforeseen traffic generators, especially at critical sections (for example, a diverging major fork). The possible consequences should be considered of a slavish "dropping of lanes" to adhere to lane volume forecasts.

Eventually, it should be possible to consider the freeway as a series of input-output components, which can be "added up" algebraically to give a picture of the operational volumes at any section along the freeway. Key elements needed to makes this procedure possible and currently missing or under development are (1) lane usage of on-ramp vehicles downstream from their merge, to the point where their distribution reaches a state of equilibrium, and (2) lane usage of prospective off-ramp vehicles from the time the drivers first begin to consider their impending divergence from the freeway stream.

Both of these elements could be very complex. The operation curves are almost certainly dependent on more than mere distance from the entrance or exit point. For example, what is the effect on the curves of volume or density, speed, the location of the freeway, whether downtown or outlying; the type of freeway, whether radial or circumferential, etc. This information would not be easy to obtain, particularly for off-ramp vehicle paths. Aerial vehicle-tracing techniques are still too costly and cumbersome to obtain the volume of information needed to determine accurately vehicle paths.

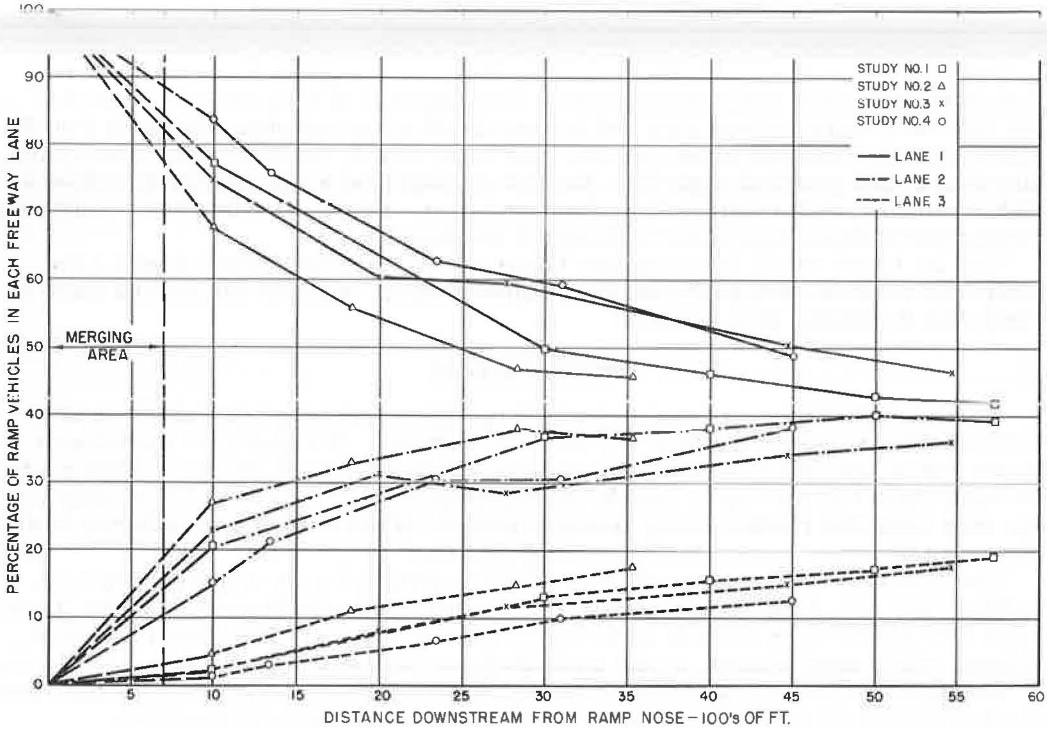


Figure 21. On-ramp vehicles usage of freeway lanes related to distance traversed downstream from ramp nose.

A start has been made, however. Believed to be the first extensive field work of this type, the Bureau's Urban Weaving Area Capacity Study crew, assisted by City of Detroit and State of Michigan personnel, conducted four studies of on-ramp vehicle freeway lane usage in Detroit in September 1963. Four freeway sections having different distance between ramps were studied on the eastern end of the Edsel Ford Expressway. This stretch of 6-lane freeway, opened in 1958-59, is two to seven miles east of the Ford-Lodge interchange, and is quite typical of a good depressed freeway facility carrying high volumes. The studies were primarily aimed at freeway lane usage information, but they were also expected to provide input data for the lane 1 equations already discussed in this report.

Briefly, the methodology was to erect the "lights on" weaving study signs on certain study ramps and then count these "lights on" ramp vehicles by lanes at points 1,000 feet or less apart downstream from the on-ramp nose. At ramp-freeway junctions, full traffic counts were made, broken down into "lights off" and "lights on." More detail on the study procedure can be ascertained by referring to the section on "Field Procedure." Manpower limitations and sign messages limited the study section lengths to from 4,500 to 6,000 feet. Actually, the zero or beginning point was at the upstream off-ramp adjacent to the signed study ramp, so 6,000-8,000 feet more accurately describe the lengths of the study sections.

Figure 21 shows the overall lane usage results for each of the $2\frac{1}{2}$-hr studies. With each signed ramp carrying between 1,450 and 1,950 complying ramp vehicles, a total of more than 6,000 ramp vehicles were traced through the four freeway sections. Approximately 15 percent of the ramp users did not comply with the "lights on" signs. Five-minute volumes expanded to hourly rates varied from 1,200-vph/lane average across the freeway lanes to 2,200-vph/lane. Most of the volumes were in the practical-to-possible capacity range. A brief description of each freeway section, giving distances between ramp noses, percent of ramp vehicles exiting at downstream off-ramps within the study section, and quality of traffic flow, is as follows:

Study No. 1—Morning Inbound.—Operation mostly 40-50 mph free-flow with several short-lived stoppages.

Chalmers on-ramp (Signed)
5,730 feet
Conner off-ramp (5.1 percent of Chalmers vehicles exited)

Study No. 2—Afternoon Outbound.—Operation steady throughout, but speeds were mostly in the 20-30-mph range.

Chene on-ramp (Signed)
1,830 feet
Mt. Elliott off-ramp (2.6 percent of Chene vehicles exited)
1,700 feet
Mt. Elliot on-ramp
3,450 feet
Van Dyke off-ramp (9.2 percent of Chene vehicles exited)

Study No. 3—Morning Inbound.—Operation rather severely congested, with much stop-and-go accordion action, as contrasted to Study No. 2, where speeds were also low, but more steady.

Gratiot on-ramp (Signed)
2,770 feet
Van Dyke off-ramp (1.2 percent of Gratiot vehicles exited)
1,700 feet
Van Dyke on-ramp

Study No. 4—Afternoon outbound.—Operation all free-flow, with speeds in the 40-50-mph range and traffic volumes high.

Gratiot on-ramp (Signed)
1,340 feet
French off-ramp (0.6 percent of Gratiot vehicles exited)
1,760 feet
French on-ramp

1,400 feet
Conner off-ramp (4.2 percent of Gratiot vehicles exited)

Referring to Figure 21, it appears that the lane usage curves are flattening out at possibly 6,000 feet downstream from the study ramp noses, with approximately 40 percent of the ramp vehicles in lane 1, 40 percent in lane 2, and 20 percent in lane 3. It can be hypothesized that lane 3 will be carrying the highest percentage of long trip vehicles and, while carrying a lesser percentage of each ramp volume, there would be a wider range of origins contained in this lane. Lane 2 would probably contain the widest range of trip lengths and lane 1 would be predominately short trips with some intermediate trip lengths.

Extensive analysis is just getting underway to develop the relationships between the lane usage of the study ramp vehicles and the total freeway volume, the merge volume, the ramp volume itself, the lane 2 (adjacent lane) volume, speeds, and extent of congestion. Preliminary results indicate that there is a statistically significant difference in percentages of ramp vehicles in lane 1, based on freeway volume rates. Stratifying into three freeway volumes rates—less than 4,500 vph, 4,500-5,400 vph, and more than 5,400 vph, there is increased movement out of lane 1 by ramp vehicles as freeway volume rates increase. While movement out of lane 1 would appear to be a more difficult maneuver as volumes increase, it also appears to become more desirable if improved position in the traffic stream is a driver's motivation. However, volume rates are not always an accurate indicator of quality of flow and more attention will be given to other factors as the analysis continues.

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Freeway Fatal Accidents: 1961 and 1962

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Accident and fatality rates for 1961 and 1962 for freeways are compared to those of conventional highways and streets in California. In 1962 there were 426 fewer people killed in California traffic accidents than there would have been if all travel had taken place on conventional highways and streets. For a given amount of travel, there are a little over one-half as many people killed on freeways as on other roads and streets.

The freeway fatal accident rate rose from 2.29 in 1961 to 2.71 in 1962, and the fatality rate rose from 2.70 to 3.35. Ten county routes, amounting to 175 mi, accounted for 90 percent of the statewide increase in fatal accidents, but the accident rate, including nonfatal and property damage accidents on these sections, rose only slightly.

•IF ALL THE TRAVEL on California freeways since Jan. 1, 1949, had been obliged to use conventional roads and streets, there would have been over 2,000 more deaths than there were during that period (Fig. 1). Because of the lower accident rate on freeways, 406 lives were saved in 1961 and 426 in 1962. Calculations showing the effect of the lower accident rates in terms of lives saved are given in Table 1. During this 2-yr period, there were 37,384 accidents on freeways and 26,112 million vehicle-miles (MVM) traveled on freeways. The accident rate was 1.43/MVM. Non-freeways had an accident rate of 4.21.

For 1961 and 1962, there were a total of 660 fatal accidents on California freeways resulting in death to 802 persons and injury to 823 others. There were 3.07 fatalities per 100 MVM for the 2-yr period. Non-freeways had 5.77 fatalities per 100 MVM. Freeways carry approximately 16 percent of all the traffic in the state, but have only 8 percent of the fatalities with a fatality rate on freeways of about one-half that on conventional roads and streets.

TYPES OF FATAL ACCIDENTS

The number and percentage of each type of freeway fatal accident are given in Table 2. The percentages of each of the four major classifications remain relatively constant from year to year. The installation of median barriers (begun in 1959) has lowered the percentage of cross-median fatal accidents from 19 percent in 1958 to 10.9 percent in 1960. The percentage for 1961 and 1962 combined was 10.3 percent, but in 1962 alone it decreased to 8.7 percent. However, although the percentage of cross-median accidents has gone down, the fatality rate, including all types of accidents, has increased. All California freeways with a volume of 60,000 veh/day or more now have median barriers installed or under construction. A study of median barriers is currently under way to determine the effect of this installation on all types of accidents.

Fixed Object Accidents

Single vehicles which hit fixed objects account for 31 percent of freeway fatal accidents, which is more than any other single classification. Fixed objects are found on

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TABLE 1

CALCULATION OF LIVES SAVED BY FREEWAYS

Type of Facility	Year	Freeway			Conventional Highways or Street, Fatalities/ 100 MVM	Estimated Fatalities if Travel Took Place on Non-Freeways	Lives Saved
		100 MVM	Fatalities	Fatalities/ 100 MVM			
Rural	1961	36.51	142	3.89	9.24[a]	337[b]	195[c]
	1962	45.66	223	4.88	9.10[a]	415	192
Urban	1961	80.69	174	2.16	4.77	385	211
	1962	95.92	259	2.70	5.15	493	234
Total	1961	—	—	—	—	—	406
	1962	—	—	—	—	—	426

[a]Rural state highways other than freeways.
[b]Product of the freeway travel (in 100 MVM) and the conventional fatality rate per 100 MVM.
[c]Estimated fatalities on non-freeways minus actual number of fatalities on freeways.

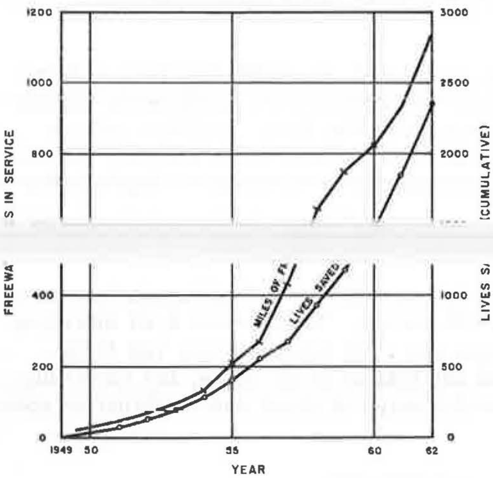

Figure 1. Estimated number of lives saved by California freeways in operation since Jan. 1, 1949.

all highways, but some are more characteristic of freeways than conventional roads. For example, there are rarely any abutments or piers at the edges of conventional roads. These objects are portions of overcrossing structures which are designed to separate cross traffic from main line traffic. Similarly, if the crossroad goes

ture is an object which would not normally be found on a highway with grade intersections.

The number of freeway fatal accidents associated with fixed objects characteristic of freeways and of all roads is indicated in Table 3.

A study of all freeway accidents (fatal and non-fatal) revealed that approximately 25 percent involved fixed objects which indicates that fixed-object accidents have a much higher fatality incidence than other accidents.

The Division of Highways Bridge Department has recently been able to eliminate columns from the right-hand side of the traveled way on most open abutment bridges with a very nominal increase in cost. The elimination of some curbs, dikes, light wells and side opening drop inlets has been accomplished. These changes cannot be easily made on existing freeways; therefore, the real effect of such changes will not be apparent for several years. It is felt that there is a need for further study and improvement in the reduction and placement of freeway fixed objects.

Rear-End Accidents

Rear-end accidents account for approximately 60 percent of all freeway accidents but only 18 percent of freeway fatal accidents. Of the 123 rear-end fatal accidents, 61 (50 percent) involved trucks, even though trucks account for only 6 to 8 percent of the vehicle-miles on freeways (Table 4).

TABLE 2

NUMBER AND TYPE OF FATAL ACCIDENTS ON CALIFORNIA FREEWAYS

Type of Accident	Fatalities per Fatal Accident (1961 and 1962)	No. (1961 and 1962)	Percent (1961 and 1962)	Percent (1960)
Single vehicle:		333	50.5	52.3
Hit fixed object	1.21	204	30.9	31.0
Did not hit fixed object	1.07	129	19.6	21.3
Pedestrian:		84	12.7	14.4
Walkers	1.02	57	8.6	10.5
Dismounted vehicle occupants	1.04	27	4.1	3.9
Head-on collision:		104	15.8	16.7
Driving on wrong side of median	1.54	36	5.5	5.8
Crossed median	1.68	68	10.3	10.9
Overtaking and sideswipe:		139	21.0	16.6
Rear-end	1.36	123	18.6	
Sideswipe	1.14	16	2.4	
Total	1.25	660	100.0	100.0

TABLE 3

FREEWAY FATAL ACCIDENTS ASSOCIATED WITH FIXED OBJECTS

Fixed Object	No. of Fatal Accidents Involved	Fatal Fixed Object Accidents (%)	All Freeway Fatal Accidents (%)
(a) Freeways			
Undercrossing rails	38	19	5.9
Overcrossing columns	35	17	5.2
Abutments	35	17	5.2
Ramp noses	18	9	2.8
Light poles	18	9	2.8
Sign poles	12	6	1.9
Median barriers	9	5	1.5
Total	165	82	25.3
(b) All Roads			
Guardrails	11	6	1.9
Bridge rails (water crossings)	9	5	1.5
Culvert headwalls	8	4	1.3
Trees	3	2	0.6
Walls	2	1	0.3
Total	33	18	5.6

Ten percent of the rear-end fatal accidents occur under congested conditions, even though freeways are congested much less than 10 percent of the time. However, the major problem is not congestion but slow-moving and stopped vehicles.

The speed limit for trucks on freeways is 50 mph as compared to 65 mph for cars and buses. There were 37 fatal accidents in which a car ran into the back of a truck. In 29 of these, the truck was traveling considerably below the speed limit. Raising this limit would have had no effect on the speed of the truck. In the other 8 fatal accidents, the severity might have been lessened had the truck been traveling faster. There were 22 fatal accidents in which a truck ran into the back of a car. Raising the truck speed limit would only tend to increase this type of fatal accident. Therefore, it appears that raising the truck speed limit would not be desirable from the standpoint of freeway fatal accidents.

Rear-end accidents are the only type which are equally prevalent (for a given amount of travel) on freeways and on conventional rural roads between intersections. One might expect to find more rear-end accidents on freeways because the volumes are generally higher, cars drive with less headway, and occasionally freeways reach capacity and experience "stop and go" situations.

TABLE 4

FATAL REAR-END ACCIDENTS

Conditions	No. of Accidents				
	Involving Trucks			Not Involving Trucks	Total
	Car Ran Into Truck	Truck Ran Into Car	Truck Ran Into Truck		
Freeway congested (stop and go)	1	1	0	10	12
Stopped vehicle (no congestion)	9	10	1	15	35
Slow vehicle (no congestion)	19	7	1	13	40
Other (sleep, speeding, etc.)	8	4	0	23	35
Total	37	22	2	61	122

TABLE 5

FATAL PEDESTRIAN ACCIDENTS

Location of Pedestrians When Struck	No. of Pedestrian Accidents	Percent
Traveled way	64	76
[illegible]	[illegible]	[illegible]
Ramp	6	7
Ramp shoulder	1	1
Unknown	1	1
Total	84	100

Pedestrian Accidents

There were 84 fatal pedestrian accidents on California freeways in 1961 and 1962. This was 13 percent of all freeway fatal accidents. In about two-thirds (57) of these accidents, pedestrians walked onto the freeway, although this is prohibited by law and most ramps are posted to in- [illegible] remaining one-third of the pedestrians were dismounted vehicle occupants. One of the reasons for fencing freeways is to keep pedestrians out. However, this also makes it difficult for dismounted vehicle occupants to get off the freeway. Table 5 shows where the pedestrians were when struck.

A study of all freeway pedestrian accidents has been undertaken to determine what measures the Division of Highways can take to reduce these accidents.

Ramp Accidents

Eleven and one-half percent of freeway fatal accidents and 18 percent of non-fatal freeway accidents occurred on a ramp or involved a ramp maneuver. The reason for the relatively low fatal ratio is probably that speeds are lower on ramps than on the freeway through lanes. The ratio of off-ramp fatal accidents to on-ramp fatal accidents was 1.7 to 1, approximately the same as for non-fatal ramp accidents.

Maneuvers in the vicinity of ramps are often thought to be a prime contribution to ramp accidents. The fact is that 75 percent of all fatal accidents involving a ramp maneuver or which occur on a ramp are single-vehicle accidents. There are less pedestrian, rear-end and sideswipe accidents proportionately on ramps than on the freeway through lanes.

Alignment standards (vertical and horizontal) generally are somewhat lower on ramps and quite often are very low. Ramps have many fixed objects (sign poles, light standards, curbs, dikes, and guardrails) which contribute significantly to ramp fatalities. In 50 percent of the off-ramp fatal accidents, a fixed object was hit.

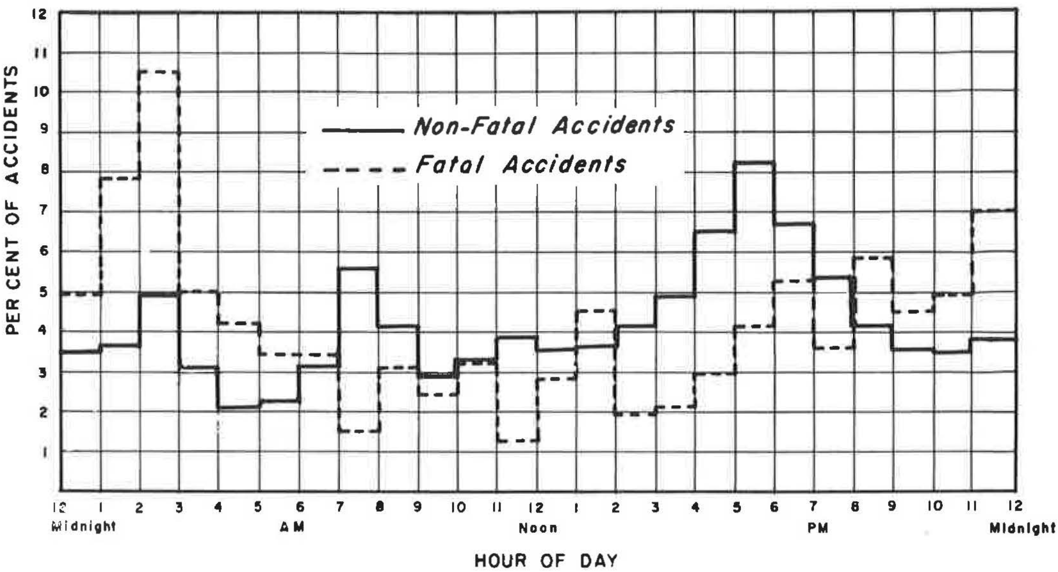

Figure 2. Hour of occurrence, fatal and non-fatal accidents, California freeways, 1961 and 1962.

TABLE 6

LIGHTING CONDITION AT TIME OF ACCIDENT

Condition	Total Accidents (%)	
	Fatal	Non-fatal
Daylight	34.9	52.2
Dusk or dawn	2.5	2.5
Dark:	62.6	45.3
Without highway illumination	35.0	25.4
With highway illumination	27.6	19.9
Total	100.0	100.0

A freeway ramp accident study is currently under way.

ENVIRONMENTAL FACTORS ASSOCIATED WITH FREEWAY ACCIDENTS

Hour of the Day

The distribution of freeway fatal accidents by hour of day does not change much from year to year. Thirty percent of the fatal accidents occur between 11 PM and 3 AM; approximately 5 percent of the travel occurs during these hours. Of all freeway accidents, only 16 percent occur between 11 PM and 3 AM.

Previous studies have indicated that 35 to 40 percent of the freeway fatal accidents occur between midnight and 5 AM. This has decreased to 32 percent in 1961 and 1962. The percentage of accidents by hour of day, as shown in Figure 2, indicates that the severity of accidents is considerably higher during hours of darkness.

Lighting

Table 6 presents light conditions for fatal and non-fatal accidents on freeways. Sixty-five of the fatal accidents occurred during hours of darkness (including dusk and dawn), whereas 48 percent of non-fatal accidents occurred during the same hours. Since the ratio of fatal accidents to total accidents varies during hours of darkness, it seems that darkness alone does not account for the increased severity at night.

Considering fatal nighttime accidents only, 27.6/62.6 or 44 percent occurred in areas where there is highway illumination. Since only portions of the freeways are

TABLE 7

WEATHER CONDITION AT TIME OF ACCIDENT

Weather	Total Freeway Accidents (%)	
	Fatal	Non-fatal
Clear or cloudy	90.0	90.4
Raining	3.5	6.5
Snowing	0.7	0.3
Fog	5.8	2.8

TABLE 8

AGE OF FREEWAY DRIVERS

Age	Fatal Accidents (%)	Total Accidents (%)	Licensed Drivers (%)
14	0.0	0.1	0.0
14-18	5.5	6.8	4.4
19-23	19.8	10.6	7.9
24-28	15.1	15.3	11.3
29-33	8.9	14.1	12.7
34-38	11.4	13.3	13.5
49-58	12.2	12.1	15.1
59-68	5.2	5.7	8.6
69	2.9	1.7	3.9

lighted, the value of illumination could be questioned. However, illumination is placed at points of potential conflict and places which have greater numbers of fixed objects, such as interchanges. Therefore, no conclusions can be drawn concerning the effect of illumination on freeway fatal accidents. It is also pertinent, but inconclusive, that 2.8 percent of all freeway fatal accidents involve light poles.

Weather

Nine out of every ten freeway fatal accidents occur during good weather. The remaining 10 percent occur during rain, snow or fog conditions, with fog accounting for over one-half of the remainder (Table 7).

An analysis of 1961 fog accidents indicated that the severity of fog accidents was practically the same as that of non-fog accidents for all state highways. Two and one-half percent of the fog accidents were fatals, whereas 2.7 percent of the total accidents were fatals. Analysis of freeway accidents revealed that 2.9 percent of the fog accidents were fatals and only 1.6 percent of the total accidents were fatals.

Age of Driver

In making this study, only the age of the driver deemed to have caused the accident was recorded. In all single vehicle accidents, the driver caused the accident. In two or more vehicle accidents it is almost always clear which driver caused the accident from the data on the accident report.

Drivers between 19 and 23 yr of age contribute disproportionately to both fatal accidents and total accidents (Table 8). Drivers in this age group caused 20 percent of the freeway fatal accidents, although they constitute only 8 percent of all licensed drivers.

It is generally believed that older drivers (69 and over) drive less than younger drivers. They also have fewer accidents and fewer fatal accidents on freeways.

Condition of Driver

From reports on 660 freeway fatal accidents, it is apparent that driver errors and physical shortcomings play an important role in such accidents. Drivers get behind the wheel when they are physically incapable of operating a motor vehicle safely. They make irrational errors and they use vehicles which should not be on the road. Fatal accidents involve actions which are irrational to an extent not observed in ordinary accidents. Two or three examples of this type of behavior are included in the Appendix, in which several fatal accidents are abstracted.

Of the drivers who caused freeway fatal accidents, 36 percent had been drinking, 17 percent had a physical shortcoming (sleep, fatigue, ill, poor eyesight, etc.), and 7 percent were driving defective vehicles. (Four percent of the drivers had more than one shortcoming; therefore, the total is 56 percent rather than 60 percent.) In addition, some drivers are emotionally upset; even though the degree of emotional disturbance is

TABLE 9

VEHICLES INVOLVED IN FATAL ACCIDENTS

Model Year	Total No. of Veh. Involved	Veh. Involved (%)	Registered Veh.[a] (%)	Veh. From Which One or More Persons Were Ejected	
				No.	Percent
1949 & Before	45	5.0	4.6	15	33
1950	33	3.7	3.5	9	27
1951	28	3.1	3.7	8	29
1952	30	3.3	3.3	9	30
1953	52	5.8	6.5	18	35
1954	38	4.2	6.2	11	29
1955	96	10.7	10.0	34	35
1956	70	7.8	9.3	21	30
1957	81	9.0	9.6	23	28
1958	62	6.9	7.1	14	23
1959	106	11.9	9.5	31	29
1960	105	11.8	10.1	26	25
1961	99	11.0	9.1	36	36
1962	52	5.8	7.5	15	29
Total	897	100.0	100.0	270	30

[a]Data derived from Automotive News, Almanac Issue, p. 42, 1963.

not readily measurable, it is considered a factor in fatal accidents. An emotionally upset driver probably will take chances and make errors he would otherwise not make.

When all freeway accidents (fatal and non-fatal) were analyzed it was found that of the drivers who caused these accidents, 14 percent had been drinking, 7 percent had a physical shortcoming and 10 percent were driving a defective vehicle, for a total of 31 percent. Drunkeness and other driver shortcomings are a major cause of, or contributing factor to, freeway fatal accidents. In the counties of Marin, San Francisco and San Mateo, 56 percent of the drivers at fault in freeway fatal accidents had been drinking.

The Vehicle

There were 897 vehicles (excluding motorcycles) studied and one or more persons were ejected from 270 of these (Table 9). All motorcycle drivers were ejected.

The percentage of vehicles from which one or more persons were ejected is not related to the year model of the vehicle. The proportion of occupants ejected from new cars was the same as from cars 10 or more years old, although it would seem that the chances of being ejected from a 10-year-old car should be greater than a brand new one because of safety features such as door latches and seat belts.

Of all persons killed in the freeway fatal accidents, 42 percent were ejected. Of all persons injured, 20 percent were ejected.

The 1959 through 1961 model vehicles were involved in slightly more fatal accidents than would be expected after looking at the percent of registered vehicles. This is not surprising since it is known that newer cars are driven more miles per year than older cars and probably faster. Exactly how much more they are driven, particularly on freeways, is not known.

The 1950 and earlier model vehicles were also involved in more fatal accidents than would be expected. This is significant because these vehicles are driven fewer miles per year on the average than newer vehicles. These older vehicles may be involved in more than their share of the freeway fatal accidents because they are in poor mechanical condition or the drivers of these older vehicles may differ from other freeway drivers in some way that would involve them in more fatal accidents.

The figures for the 1962 model vehicles are not significant because during part of the study period, there were no 1962 model vehicles.

INCREASE IN FATAL ACCIDENTS FROM 1961 TO 1962

In 1962 there was an increase over 1961 of 121 fatal accidents (269 vs 390) and an increase of 158 fatalities (322 vs 480). The fatal accident rate per hundred MVM rose from 2.29 to 2.71. As indicated in Table 10, 44 of the fatal accidents occurred on new freeways opened to traffic during 1961 and 1962, and were offset by increases in vehicle-miles of travel. On the 811 mi of freeway open to traffic during all of both years, there was an increase in 1962 over 1961 of 77 fatal accidents. In an attempt to explain this increase, an analysis was made by locations, and it was found that 10 sections of freeway totaling 175 mi accounted for 90 percent of the increase; i.e., if it were not for the fact that in 1962 70 more fatal accidents occurred than in 1961 on these ten sections, there would have been no increase in the statewide rate. Data for these sections are tabulated in Table 11. The rate of all accidents including non-fatal and property damage rose from 1.18 to 1.36 on these ten sections. In spite of this rise, the rate on these 10 sections was below the rate for the statewide freeway system in both 1961 and 1962.

It will be noted that the rate of fatal accidents on the remainder of the freeway system decreased from 2.45/100 MVM in 1961 to 2.36/100 MVM in 1962.

TABLE 10

INCREASE IN FATAL ACCIDENTS

Year	No. of Fatal Accidents on Freeways Opened to Traffic		Total	
	Before	After	Fatal	
1961	264	5	269	322
1962	341	49	390	480
Increase	77	44	121	158

Analysis of the Ten Sections

Table 11 also indicates that 4 of the ten sections (Mrn-1, SD-2, SD-12 and

statewide average in 1961 and approximately equal to the statewide average in 1962. The increases in these sections

TABLE 11

FREEWAY SECTIONS SHOWING LARGE INCREASE IN FATAL ACCIDENTS, 1962 OVER 1961

County Route[a]	Fatal Accidents		Increase		Length (mi)	MVM		Fatal Accidents per 100 MVM		Total Accidents		Accidents/ MVM	
	1961	1962	No.	Percent		1961	1962	1961	1962	1961	1962	1961	1962
Mrn-1	5	9	4	80	12.3	230	235	2.2	3.8	373	436	1.62	1.85
SD-2	2	8	6	300	23.1	239	235	0.8	3.4	413	394	1.73	1.68
SB-2	1	10	9	900	16.9	92	112	1.1	8.9	62	59	0.67	0.53
Fre-4	3	8	5	167	12.3	88	87	3.4	9.2	132	123	1.50	1.41
Ker 4	3	0	6	200	13.4	73	69	4.1	13.1	114	110	1.56	1.59
Mad-4	1	5	4	400	10.7	50	54	2.0	9.3	37	56	0.74	1.04
SJ-4	3	8	5	167	23.7	115	121	2.6	6.6	91	122	0.79	1.01
SD-12	1	8	7	700	11.5	224	239	0.4	3.3	223	259	1.00	1.08
SBd-26	6	22	16	267	24.6	328	344	1.8	6.4	387	537	1.18	1.56
SM-68	12	20	8	67	26.1	827	884	1.5	2.3	844	1,135	1.02	1.28
Total, 10 sections	37	107	70	189	174.6	2,266	2,380	1.63	4.49	2,676	3,231	1.18	1.36
Statewide total	269	390	121	45	1,148.0	11,720	14,392	2.29	2.71	16,563	20,821	1.41	1.45
Total remainder	232	283	51	22	973.4	9,454	12,012	2.45	2.36	13,887	17,590	1.47	1.46

[a]No other county route increased by more than three fatal accidents.

TABLE 12

COMPARISON OF FATAL AND TOTAL ACCIDENTS

Section	MVM Travel		Total Accidents		Total/ 100 MVM		Fatal Accidents		Fatal/ 100 MVM		Fatal/ Total Ratio	
	1961	1962	1961	1962	1961	1962	1961	1962	1961	1962	1961	1962
US 99, Kern, Fresno, Madera, & San Joaquin Co.	326	331	374	411	151	124	10	30	3.1	9.1	0.027	0.073
S. B. Freeway, S. B. Co.	328	344	387	537	118	156	6	22	1.8	6.4	0.015	0.041
US 101, S. B. Co.	92	112	62	59	67	53	1	10	1.1	8.9	0.016	0.168
Total	746	787	823	1,007	110	128	17	62	2.28	7.88	0.021	0.062
Statewide total	11,720	14,392	16,563	20,821	141	145	270	390	2.29	2.71	0.016	0.019

TABLE 13

TYPES OF FATAL ACCIDENTS ON FREEWAY SECTIONS SHOWING LARGE INCREASE IN FATAL ACCIDENTS, 1962 OVER 1961

Type of Accident	Statewide Total		No. of Fatal Accidents							
			US 99, Kern, Fresno, Madera & San Joaquin Co.		S. B. Freeway, S. B. Co.		US 101, S. B. Co.		Total (3 sections)	
	No.	%	1961	1962	1961	1962	1961	1962	1961	1962
Single vehicle:										
Hit fixed object	204	30.9	2	7	2	7	0	0	4	14
Did not hit fixed object	129	19.6	3	2	2	3	0	4	5	9
Pedestrian:										
Walkers	57	8.6	1	6	0	1	1	3	2	10
Dismounted vehicle occupants	27	4.1	1	1	0	0	0	1	1	2
Head-on collision										
Wrong-way	36	5.5	0	2	1	2	0	2	1	6
Crossed median	68	10.3	1	1	0	4	0	0	1	5
Overtaking:										
Rear-end	123	18.6	2	11	1	4	0	0	3	15
Sideswipe	16	2.4	0	0	0	1	0	0	0	1
Total	660	100.0	10	30	6	22	1	10	17	62

can be considered as "coming up to the average" through chance, rather than as an alarming increase.

Of the remaining six sections, four were US 99 in the San Joaquin Valley (Fre-4, Ker-4, Mad-4, SJ-4). Although individually these four sections show small numerical increases (5, 6, 4 and 5, respectively), when combined the total increase is significant.

The ratio of fatal to non-fatal freeway accidents for US 99 in the San Joaquin Valley and the two remaining sections (San Bernardino Freeway in San Bernardino County and US 101 in Santa Barbara County) is given in Table 12. It will be noted that the overall accident rate on these roads increased only 16 percent (from 1.10 to 1.28/MVM), whereas the fatal accident rate increased 246 percent (from 2.28 to 7.88/100 MVM).

The ratio of fatal to total accidents increased tremendously in 1962 in all three sections even though the statewide ratio did not increase very much. There is a very fine dividing line between a serious injury accident and a fatal accident. Since the total accident rates did not change nearly as much as the fatal accident rates in the three sections, it appears that these sections were "extremely unlucky" in 1962. Several other attempts were made to explain the increase in fatal accidents in the three sections, and even though they were unsuccessful, the data are presented in Tables 13 and 14.

In short, there was an increase of 121 fatal accidents between 1961 and 1962 causing the rate to rise from 2.29 to 2.71/100 MVM. This is accounted for as follows:

TABLE 14
FACTORS AFFECTING ACCIDENTS ON FREEWAY SECTIONS SHOWING LARGE INCREASE IN FATAL ACCIDENTS, 1962 OVER 1961

Factor	Statewide Total		No. of Fatal Accidents							
			US 99, Kern, Fresno, Madera & San Joaquin Co.		S. B. Freeway, S. B. Co.		US 101, S. B. Co.		Total (3 sections)	
	1961	1962	1961	1962	1961	1962	1961	1962	1961	1962
Sobriety										
Had been drinking	68	129	2	3	2	6	0	4	4	13
Had not been drinking	146	200	7	24	3	15	1	5	11	44
Other & unknown	56	61	1	3	1	1	0	1	2	5
Total	270	390	10	30	6	22	1	10	17	62
Lighting:										
Daylight	90	138	6	12	2	5	0	1	8	18
Dusk or dawn	4	12	0	1	0	0	0	0	0	1
Dark—no lights	90	139	3	12	2	10	0	8	5	30
Dark—lights	83	94	1	5	2	7	1	1	4	13
Unknown	3	7	-	-	-	-	-	-	-	-
Total	270	390	10	30	6	22	1	10	17	62
Weather										
Clear or cloudy	243	334	8	21	5	18	1	7	14	46
Raining	5	18	1	1	0	3	0	1	1	5
Snowing	1	3	0	0	0	0	0	0	0	0
Fog	14	24	1	8	0	1	0	1	1	10
Other or unknown	7	11	0	0	1	0	0	1	1	1
Total	270	390	10	30	6	22	1	10	17	62

1. Forty-four owing to travel on new freeways (Table 10);
2. Twenty-five owing to sections in which the fatal accident rates were unusually
3. Forty-five owing to an increase in ratio of fatal accidents to non-fatal accidents on portions of three routes, comprising 102 mi of freeway, apparently due primarily to sheer chance (Table 12); and
4. Seven owing to increased travel on existing freeways.

Appendix

FULL FREEWAY FATAL ACCIDENT SUMMARY SHEETS
(Cross-Reference)

Sheet 1

Number of Lanes 8

Time of Day 1050 (Daylight) ______ (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2	#3	1
No. of People Killed In:	1			1
No. of People Injured In:	0			0
Number of Pedestrians Killed:				

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo-Fixed object bridge railings on both sides of the overcrossing

Primary Driver Condition or Error: Drunk (blood alcohol 0.20)

Description: Driving at a very high rate of speed, approximately 80 miles per hour according to two witnesses. Vehicle #1 went up the off-ramp which is a diamond with a hook at the upper end. The vehicle was going about 65 miles per hour after it passed the 25 mph off-ramp speed sign. Approximately 150 feet before it got to the terminal of the off-ramp, the driver applied the brakes, skidded 60 feet, and struck the curb on the left. It then scraped along the curb for a distance of 84 feet, rolled over the eastbound half of the Overcrossing, struck the curb median, and then slid upside down for 66 feet on the other side of the median where it landed upside down against the bridge rail. The driver was ejected from the car. The car went about 150 feet upside down after it turned over. It turned over when it hit the end of the curb nose where the ramp joins the cross street.

Sheet 2

Number of Lanes 8 lanes

Time of Day ____________ (Daylight) 2030 (Dark or Dusk)

	#1	#2	#3	Total No.
Type of Vehicle:	Car	Truck		2
No. of People Killed In:	2	0		2
No. of People Injured In:	1	0		1
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo-Fixed Object (truck parked on shoulder)

Primary Driver Condition or Error: Drunk

Description: Aiming for an off-ramp, the driver went onto the shoulder before getting to the ramp and hit a truck parked there. (This is a lighted freeway according to the Police report.) The driver climbed out of the car and began running around yelling "shoot me, shoot me" and tried to pull an officer into #4 traffic lane. The driver had to be handcuffed and physically restrained. Two passengers killed, including child having same address as driver but different name.

Number of Lanes 4

Time of Day ____________ (Daylight) 0515 (Dark or Dusk)

	#1	#2	#3	Total No.
Type of Vehicle:	Car			1
No. of People Killed In:	1			1
No. of People Injured In:	0			1
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo - Lamp Standard

Primary Driver Condition or Error: Driving very fast in rain.

Description: The driver was very tired, according to statement. Vehicle #1 was being driven about 80 MPH in the rain. The driver lost control and cannot explain why. The vehicle ran off the road and struck the light standard on the right hand side after going under an Overcrossing. The car then fishtailed for 400' before coming to a stop. A passenger in the rear seat was killed.

Sheet 4

Number of Lanes 4

Time of Day 1345 (Daylight) ______ (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2	#3	1
No. of People Killed In:	2			2
No. of People Injured In:	0			0
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo Bridge Abutment

Primary Driver Condition or Error: Possibly Asleep

Description: In moderate to heavy traffic in the afternoon the car suddenly began driving erratically, speeding up and slowing down in front of other vehicles. The vehicle went off the roadway and went off the shoulder then back onto the roadway. Then it went off onto the shoulder again as if to stop but instead of stopping it went ahead and hit the bridge abutment on the right without attempting to stop.

Sheet 5

Number of Lanes 6 lanes

Time of Day 0855 (Daylight) ______ (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2	#3	1
No. of People Killed In:	1			1
No. of People Injured In:	4			4
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo, fixed object, light pole

Primary Driver Condition or Error: Front end of car not in very good shape.

Description: Vehicle #1 on the freeway just before reaching the on-ramp, lost control due to the front end bouncing on an out-of-balance tire. The driver applied the brakes on wet pavement and skidded into the curb which threw the car out of control, and spun it around. This curb was on the right hand side of the freeway. After the car skidded 150' along the shoulder, it struck a pole in the gore of the on-ramp. The driver was killed.

Sheet 6

Number of Lanes 4

Time of Day 0550 (Daylight) ______ (Dark or Dusk)

	#1	#2	#3	Total No.
Type of Vehicle:	2 passenger Car	Car		2
No. of People Killed In:	1	0		1
No. of People Injured In:	3	0		3
Number of Pedestrians Killed:				

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo - bridge rail

Primary Driver Condition or Error: Had been drinking

Description: The vehicle #1 (2 passenger) loaded with four passengers, one of them sitting on the floor, caught up with a slower vehicle and lost control going into the median. The vehicle traveled 225' out of control in the median, then went into a 200' sideways skid across both lanes into the guardrail approaching the OP and then skidded along the guardrail for 110' and into the end post of the bridge; there is where the fatality occurred. All four passengers in the vehicle were thrown out of the car. The car that was being passed also went into a skid when the driver saw that the vehicle was about to be hit and drove into the guardrail approach on the left side of the highway. However, nobody in this vehicle was injured.

Sheet 7

Number of Lanes 4

Time of Day ______ (Daylight) 0115 (Dark or Dusk)

	#1	#2	#3	Total No.
Type of Vehicle:	Car			1
No. of People Killed In:	1			1
No. of People Injured In:	0			0
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo - Off Road

Primary Driver Condition or Error: Extreme high speed

Description: Vehicle #1 and another car were apparently racing as stated by three witnesses who they had passed going approximately 85 to 90 MPH. When they got to the curve in the main roadway, which curves to the left, vehicle #1 did not make it. It went off the road on the left first, which was the inside of the curve. It then skidded along the left hand shoulder 120', across the turning roadway and then the car finally wound up over 300' from where it left the turning roadway down on the off-ramp. In the middle of the trip 300' off the roadway the driver was ejected and killed.

Sheet 8

Number of Lanes 4

Time of Day ______ (Daylight) 0035 ______ (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2	#3	1
No. of People Killed In:	1			1
No. of People Injured In:	0			0
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo-Off road on right side

Primary Driver Condition or Error: Asleep

Description: Just prior to the accident vehicle #1 had been observed with the driver leaning against the window of the vehicle and the car was weaving all over the road. It ran into the earth median, first knocking down several sight posts, then turned sharply to the right across the freeway and struck the roadside shoulder and went on over the fill and also the off-ramp. Vehicle #1 turned over in the off-ramp and finally wound up in a drainage canal outside of the right of way. Note: This was not an off-ramp accident.

Sheet 9

Number of Lanes 4-Divided

Time of Day ______ (Daylight) 1855 ______ (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2	#3	1
No. of People Killed In:	0			0
No. of People Injured In:	0			0
Number of Pedestrians Killed:				1

Primary Vehicle: Vehicle #1

Primary Type of Accident: Pedestrian

Primary Driver Condition or Error: None

Description: The pedestrian was walking across the freeway on a rainy night dressed in dark clothing and having a very strong alcoholic odor, in fact the pedestrian had a blood alcohol content of 0.16. The driver of the car saw the pedestrian about 60 feet before impact. The driver applied the brakes and went into a skid on the wet pavement.

Sheet 10

Number of Lanes 6

Time of Day ____________ (Daylight) 0415 ________ (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2	#3	1
No. of People Killed In:	1			1
No. of People Injured In:	0			0
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo - Off Road

Primary Driver Condition or Error: Extreme wild driving and drunk. BA : 0.23

Description: Vehicle #1 traveling in the vicinity of an undercrossing at about 80 miles per hour passed another vehicle and then fishtailed and skidded all over the road. The car went over the right hand edge of the shoulder and was airborne for 35 feet before landing on the ramp below. It then continued across the ramp and into the dirt where it wound up 210 feet from the place where it first left the road. The car rolled when it first left the freeway, ejecting the driver who was killed.

Sheet 11

Number of Lanes 6

Time of Day ____________ (Daylight) 1740 ________ (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2	#3	1
No. of People Killed In:	0			0
No. of People Injured In:	0			0
Number of Pedestrians Killed:				1

Primary Vehicle: Vehicle #1

Primary Type of Accident: Pedestrian

Primary Driver Condition or Error: None

Description: Pedestrian, who lived in neighborhood, was crawling on hands and knees across the freeway in the dark. The driver of Vehicle #1 saw the pedestrian just before impact.

Sheet 12

Number of Lanes 6

Time of Day ______ (Daylight) 0145 (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2	#3	1
No. of People Killed In:	1			1
No. of People Injured In:	0			0
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Solo - Off Road

Primary Driver Condition or Error: Excessive Speed

Description: Vehicle #1 going about 100 (?) MPH (witness guess) in fog, failed to negotiate a main line curve (2,500' radius) flew off of freeway and landed on a private parking lot behind an apartment house.

Sheet 13

Number of Lanes 4

Time of Day ______ (Daylight) 1710 (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2 Car	#3 Car	3
No. of People Killed In:	0	0	3	3
No. of People Injured In:	2	2	1	5
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Head-on X median

Primary Driver Condition or Error: Drunk, 80 MPH

Description: Vehicle #1 was traveling at approximately 80 MPH (judged by witness going approximately 65) struck a barricade in the median then went into broadside skid. It slid 70 feet on traveled way, then 80 feet in median, then across both opposing lanes where it was struck by Vehicle #3 in far right lane. Then it was struck by vehicle #2 in near (median) opposing lane. Fatalities were all in vehicle #3. Vehicle #1 was probably stolen; it was full of shady type individuals, some of whom had "done time" together, and who had been drinking beer and wine all day.

Sheet 14

Number of Lanes 6

Time of Day ______ (Daylight) 0148 (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2	#3	1
No. of People Killed In:	0			0
No. of People Injured In:	0			1
Number of Pedestrians Killed:				1

Primary Vehicle: Pedestrian(directing traffic at the scene of a prior accident)

Primary Type of Accident: Pedestrian

Primary Driver Condition or Error: Had Been Drinking

Description: Driver of Vehicle #1 failed to notice a string of flares and flashing lights and etc., at a prior accident. Also failed to notice the pedestrian standing in the road directing travel. Vehicle #1 went into a skid and struck the pedestrian. Vehicle #1 then went clear off of the freeway to the right over the outer separation and onto the on-ramp which enters the freeway near this location and continued on fleeing the scene of the accident. Drive of Vehicle #1 is believed to have been drinking as well as being inattentive and fleeing the scene of the accident. The pedestrian was killed.

Sheet 15

Number of Lanes 6

Time of Day 10:09 (Daylight) ______ (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2 Car	#3 Car	3
No. of People Killed In:	0	2	0	2
No. of People Injured In:	0	0	0	2
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Head-on X median

Primary Driver Condition or Error: speed

Description: Vehicle #1 which was fixed up like a hot-road, jumped the median and struck vehicle #2 and Vehicle 3. The two occupants of vehicle #2 were ejected and killed. The driver of the hot-rod that jumped the median was young, had been chased by another patrolman but had slowed down before being arrested.

Sheet 16

Number of Lanes 4

Time of Day ____________ (Daylight) 2330 ________ (Dark or Dusk)

	#1	#2	#3	Total No.
Type of Vehicle:	Car	Car		2
No. of People Killed In:	1	0		1
No. of People Injured In:	0	0		0
Number of Pedestrians Killed:				

Primary Vehicle: Vehicle #2

Primary Type of Accident: Head-on, wrong-way driver

Primary Driver Condition or Error: Strong odor of alcohol, driving without headlights and on wrong side of freeway

Description: Vehicle #2 came onto the freeway from an undetermined location and drove the wrong way with the headlights off. The driver then suddenly turned the lights on when approximately 100 feet from car going the right way and they crashed (note: this driver must have been insane or very seriously troubled in some way and this would be a good case for an investigating team to investigate the driver's background). Vehicle #1 driver was killed.

Sheet 17

Number of Lanes 4

Time of Day 1610 ________ (Daylight) ____________ (Dark or Dusk)

	#1	#2	#3	Total No.
Type of Vehicle:	Car	Truck & Trailer		2
No. of People Killed In:	2	0		2
No. of People Injured In:	2	0		2
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #2

Primary Type of Accident: Rear-end

Primary Driver Condition or Error: Driver was afraid to put on brakes for fear of spilling logs.

Description: Vehicle #1 traveling about 60 MPH had a flat tire in the rear and lost control and was wobbling on the pavement. Vehicle #2 which was a few hundred feet behind Vehicle #1 was afraid to put on the brakes because it would spill the load. Two people in Vehicle #1 were killed.

Sheet 18

Number of Lanes 8

Time of Day 1320 (Daylight) ____ (Dark or Dusk)

	#1	#2	#3	Total No.
Type of Vehicle:	2 passenger car	car		2
No. of People Killed In:	1	0		1
No. of People Injured In:	1	0		1
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Rear-end followed by off-road

Primary Driver Condition or Error: Extreme speed

Description: Vehicle #1 was driving in what the police call lane 4, the lane next to the shoulder, and changed lanes into lane 3 where it struck the left rear corner of vehicle #2. Vehicle #1 apparently was on its way towards the median lane. After striking the vehicle #2, vehicle #1 went into a broadside skid, skidding for 269 feet in a gradual arc and finally leaving the roadway on the righthand side and rolling down the bank. A passenger in vehicle #1 was killed. The officers using a coefficient of 80% computed the speed of vehicle #1 when it started to skid as 82 MPH.

Number of Lanes 4

Time of Day ____ (Daylight) 2300 (Dark or Dusk)

	#1	#2	#3	Total No.
Type of Vehicle:	car	car		2
No. of People Killed In:	0	1		1
No. of People Injured In:	1	0		1
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Rear-end

Primary Driver Condition or Error: Extremely belligerent driving

Description: Vehicle #1 was traveling on the freeway at slower than average speed. When vehicle #1 noticed somebody overtaking the driver would straddle the lane line and would jockey with the other drivers apparently trying to race. This was reported by two independent witnesses not involved in the accident. Finally vehicle #2 passed vehicle #1 and pulled back into the right lane. Vehicle #1 then accelerated and clipped vehicle #2 in the left rear, causing vehicle #2 to over turn killing the driver of vehicle #2.

Sheet 20

Number of Lanes 4

Time of Day ______ (Daylight) 0225 (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2 Car	#3	2
No. of People Killed In:	1	0		1
No. of People Injured In:	0	1		1
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #2

Primary Type of Accident: Rear-end

Primary Driver Condition or Error: Inexperience

Description: Driver of Vehicle #2 came upon a scene consisting of a car on fire and a row of flares which had been set out and vehicle #1 which was slowing down with its orange stop lights on because of the flares and so forth. Vehicle #2 plowed right into the rear-end of vehicle #1 without slowing down. A witness behind the vehicle #2 also saw the flares, etc., and slowed down and saw the whole thing happen. The only driver who didn't slow down was the driver of vehicle #2.

Sheet 21

Number of Lanes 4

Time of Day ______ (Daylight) 2300 (Dark or Dusk)

				Total No.
Type of Vehicle:	#1 Car	#2 Car	#3 Truck	3
No. of People Killed In:	2	0	0	2
No. of People Injured In:	0	0	0	2
Number of Pedestrians Killed:				0

Primary Vehicle: Vehicle #1

Primary Type of Accident: Rear-end

Primary Driver Condition or Error: Stopped in traveled way

Description: The driver of vehicle #1 missed the turn-off in the very dense fog and stopped in the middle of the southbound lanes, straddling the lane line. The car was then struck by vehicle #2 and vehicle #3. Driver and passenger of vehicle #1 were killed.

Effect of Traffic Volumes and Number of Lanes On Freeway Accident Rates

RICHARD A. LUNDY, California Division of Highways

Three years of experience on 659 mi of four-, six-, and eight-lane freeways have revealed that the accident rates for each classification will normally increase with an increasing ADT. The rate of increase per 10,000-veh increase in ADT is four-lane, 0.240 accidents/MVM; six-lane, 0.094 accidents/MVM; and eight-lane, 0.078 accidents/MVM. For any given ADT, the four-lane freeways have a higher accident rate than the six-lane, and six-lane freeways have a higher rate than the eight-lane. Therefore, as the ADT increases, the difference in rates between the three classifications becomes greater. This relationship introduces the possiblity of significantly reducing the total number of freeway accidents by increasing the number of traffic lanes, even though the increase is not required by traffic volumes.

•A RECENT REPORT (1) by the California Division of Highways indicated that the accident rates on freeways increase as the traffic volumes increase, and for a given

The present study analyzes in more detail the accident rate vs volume relationship reported in the comparative freeway study report. It is based on a 3-yr observation (1960-1962) of 659 mi of freeway on which 26,152 million vehicle miles (MVM) were traveled and 35,675 accidents occurred.

All freeway sections in existence during the study period were classified into three categories: four, six, and eight lanes. Sections having more lanes in one direction of travel than in the other and sections with less than 30 MVM/yr of travel were eliminated. The requirement of a minimum amount of travel of 30 MVM was imposed since it was felt that with this amount of travel, the element of chance variation in the accident rate would be significantly reduced and fairly stable accident rates would result. The number of sections meeting these requirements are indicated in Table 1.

TABLE 1

FREEWAY SECTIONS MEETING REQUIREMENTS FOR STUDY

Year	No. of Sections			
	4-Lane	6-Lane	8-Lane	Total
1960	24	21	10	55
1961	35	30	17	82
1962	42	35	20	97
Total	101	86	47	234

For each lane classification, the curve best representing the accident rate vs ADT relationship was calculated by the

Paper sponsored by Committee on Highway Safety.

TABLE 2

1960 ADT AND ACCIDENT RATES, GROUP AVERAGES

GROUP NO.	ADT CLASS INTERVAL	4 LANES					6 LANES					8 LANES				
		Miles	No of Acc.	MVM	Acc./MVM	Avg. ADT for Group*	Miles	No. of Acc.	MVM	Acc./MVM	Avg. ADT for Group*	Miles	No. of Acc.	MVM	Acc./MVM	Avg. ADT for Group*
I	Less than 7,000															
II	7,000 to 9,999															
III	10,000 to 14,999	82.79	327	379	0.86	12.5										
IV	15,000 to 21,499	40.03	210	246	0.85	18.5										
V	21,500 to 31,599	103.96	1268	982	1.29	26.2	6.69	58	65	0.89	26.8					
VI	31,600 to 46,499	12.47	283	165	1.72	37.0	28.32	322	361	0.89	36.5					
VII	46,500 to 67,999	4.17	172	95	1.81	62.6	43.59	980	881	1.11	56.9	17.78	218	231	0.94	50.7
VIII	68,000 to 99,999						47.30	2005	1372	1.46	82.6	5.51	358	174	2.06	82.4
IX	100,000 to 120,000						18.69	940	706	1.33	107.1	12.72	621	469	1.32	110.4
X	Over 120,000						1.75	108	88	1.23	137.9	8.72	1011	525	1.93	160.8
	TOTALS	243.42	2260	1867	1.21	24.4	146.34	4413	3473	1.27	66.3	44.73	2208	1399	1.58	102.0

*ADT in thousands

BLE 3

1961 ADT AND ACCID T RATES, GROUP AVERAGES

GROUP NO.	ADT CLASS INTERVAL	4 LANES					6 LANES					8 LANES				
		Miles	No. of Acc.	MVM	Acc./MVM	Avg. AI for Grou	Miles	No. of Acc	MVM	Acc./MVM	Avg. ADT for Group*	Miles	No. of Acc.	MVM	Acc./MVM	Avg. ADT for Group*
I	Less than 7,000															
II	7,000 to 9,999															
III	10,000 to 14,999	99.93	384	431	0.78	13.3										
IV	15,000 to 21,499	52.66	302	324	0.93	18.4										
V	21,500 to 31,599	69.70	685	623	1.10	24.5	12.88	114	121	0.94	27.3					
VI	31,600 to 46,499	62.75	1172	879	1.33	38.7	32.37	443	478	0.93	41.4	3.27	25	38	0.66	35.3
VII	46,500 to 67,999	8.43	327	169	1.94	55.0	29.25	663	582	1.14	54.7	22.35	378	478	0.79	60.8
VIII	68,000 to 99,999						46.97	1532	1350	1.13	83.7	11.00	508	355	1.43	87.8
IX	100,000 to 120,000						30.75	2263	1194	1.90	110.4	23.38	1254	952	1.32	109.0
X	Over 120,000											15.52	1479	864	1.71	146.3
	TOTALS	293.47	2870	2496	1.15	27.4	152.22	5015	3725	1.35	66.8	75.52	3644	2687	1.36	94.2

*ADT in thousands

TABLE 4

1962 ADT AND ACCIDENT RATES, GROUP AVERAGES

GROUP NO.	ADT CLASS INTERVAL	4 LANES					6 LANES					8 LANES				
		Miles	No. of Acc.	MVM	Acc./MVM	Avg. ADT for Group*	Miles	No. of Acc.	MVM	Acc./MVM	Avg. ADT for Group*	Miles	No. of Acc.	MVM	Acc./MVM	Avg. ADT for Group*
I	Less than 7,000	24.14	57	55	1.04	6.2										
II	7,000 to 9,999	31.58	146	97	1.51	8.9										
III	10,000 to 14,999	94.62	374	399	0.94	13.8										
IV	15,000 to 21,499	69.41	412	400	1.03	17.2	5.81	45	44	1.02	20.8					
V	21,500 to 31,599	75.65	767	701	1.09	25.3	18.22	165	152	1.09	25.4					
VI	31,600 to 46,499	75.44	1510	1008	1.50	38.4	30.79	526	453	1.16	40.3	12.03	178	158	1.13	45.6
VII	46,500 to 67,999	12.17	496	226	2.19	50.6	46.07	1172	878	1.33	53.5	10.24	164	170	0.96	59.9
VIII	68,000 to 99,999	4.27	263	107	2.46	68.4	56.56	2696	1702	1.58	83.8	20.37	641	626	1.02	85.9
IX	100,000 to 120,000						14.75	1074	609	1.76	110.4	11.53	562	444	1.27	106.3
X	Over 120,000						10.61	987	490	2.01	127.5	35.13	3030	1796	1.69	144.7
	TOTALS	387.28	4025	2993	1.34	27.5	182.81	6665	4328	1.54	68.2	89.30	4575	3194	1.43	98.7

*ADT in thousands

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1960 TO 1962 ADT AND A DENT RATES, GROUP AVERAGES

GROUP NO.	ADT CLASS INTERVAL	4 LANES					6 LANES					8 LANES				
		Miles	No. of Acc.	MVM	Acc./MVM	Avg. AD for Grou	Miles	No. of Acc.	MVM	Acc./MVM	Avg. ADT for Group*	Miles	No. of Acc.	MVM	Acc./MVM	Avg. ADT for Group*
I	Less than 7,000	24.14	57	55	1.04	6.2										
II	7,000 to 9,999	31.58	146	97	1.51	8.9										
III	10,000 to 14,999	277.34	1085	1269	0.86	13.2										
IV	15,000 to 21,499	162.10	924	970	0.95	17.9	5.81	45	44	1.02	20.8					
V	21,500 to 31,599	249.31	2720	2306	1.18	25.4	37.79	337	338	1.00	26.5					
VI	31,600 to 46,499	150.66	2965	2051	1.45	38.4	91.48	1291	1292	1.00	39.7	15.30	203	196	1.04	42.1
VII	46,500 to 67,999	24.77	995	491	2.03	54.1	18.91	2815	2341	1.20	54.8	50.37	760	879	0.86	57.8
VIII	68,000 to 99,999	4.27	263	107	2.46	68.4	50.83	6233	4424	1.41	83.5	36.88	1507	1155	1.30	85.9
IX	100,000 to 120,000						54.19	4277	2509	1.70	109.6	47.63	2437	1865	1.31	108.3
X	Over 20,000						12.36	1095	578	1.89	130.1	59.37	5520	3185	1.73	148.9
	TOTALS	924.17	9155	7346	1.25	26.7	81.37	16,093	11,526	1.40	67.2	209.55	10,427	7280	1.43	97.8

* ADT in thousands

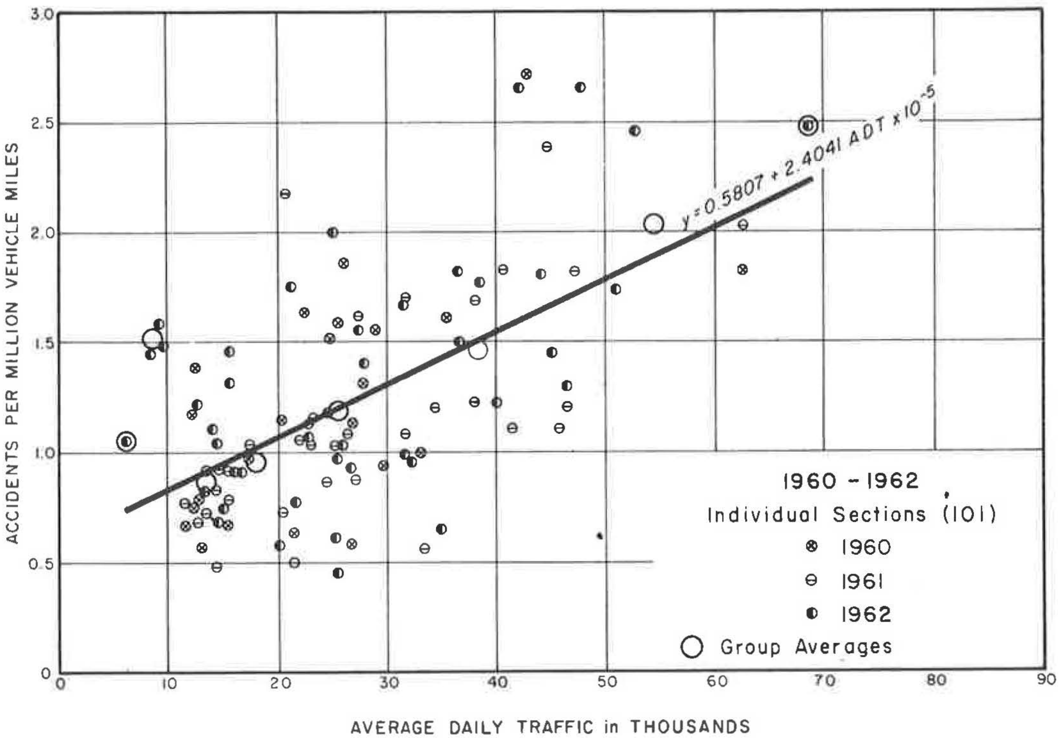


Figure 1. 1960 to 1962 four-lane accident rate vs ADT (9,155 accidents, 7,346 MVM).

TABLE 6

STANDARD ERROR OF ESTIMATE FOR ACCIDENT RATE VS ADT CURVES (Least Squares Program)

YEAR	NUMBER of LANES	TYPE of CURVE				
		Linear $y = a + bx$	Exponential $y = a(b)^x$	Semi Log 1 $\log_e y = a + bx$	Semi Log 2 $y = a + b \log_e x$	Log Log $y = a(x)^b$
1960	4 Lanes	0.397	0.410	0.422	0.395	0.398
1961	4 Lanes	0.358	0.360	0.364	0.363	0.364
1962	4 Lanes	0.463	0.447	0.464	0.510	0.519
1960-1962	4 Lanes	0.419	0.415	0.422	0.446	0.447
1960	6 Lanes	0.495	0.481	0.498	0.463	0.473
1961	6 Lanes	0.359	0.353	0.357	0.378	0.374
1962	6 Lanes	0.412	0.412	0.415	0.420	0.421
1960-1962	6 Lanes	0.424	0.426	0.430	0.430	0.431
1960	8 Lanes	0.455	0.479	0.487	0.480	0.489
1961	8 Lanes	0.391	0.395	0.412	0.392	0.403
1962	8 Lanes	0.401	0.395	0.401	0.415	0.414
1960-1962	8 Lanes	0.438	0.436	0.446	0.447	0.451

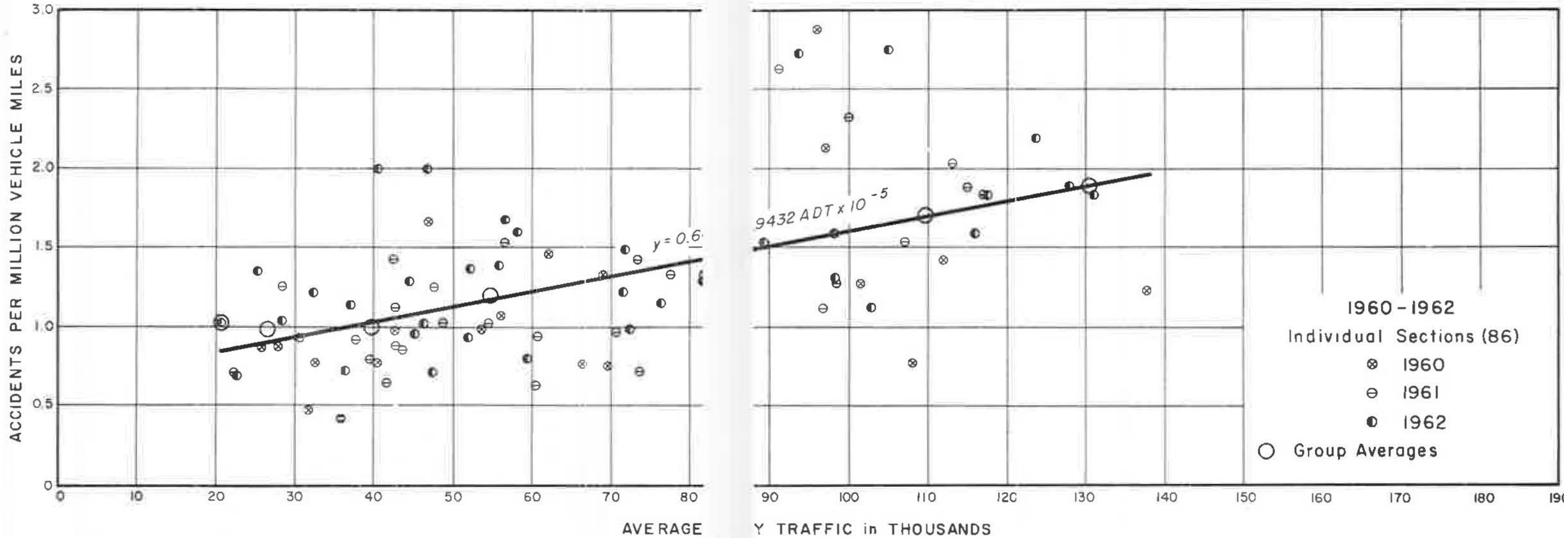


Figure 2. 1960 to 1962 six-lane ac ent rate vs ADT (16,093 accidents, 11,526 MVM).

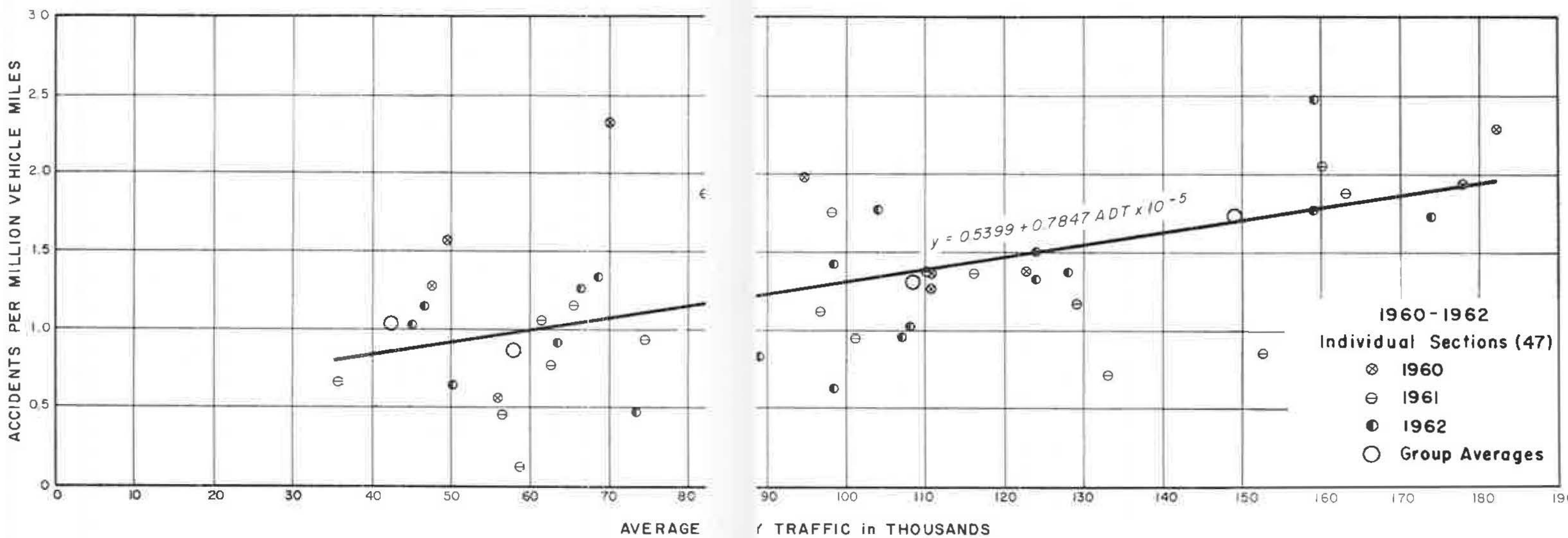


Figure 3. 1960 to 1962 eight-lane dent rate vs ADT (10,427 accidents, 7,280 MVM).

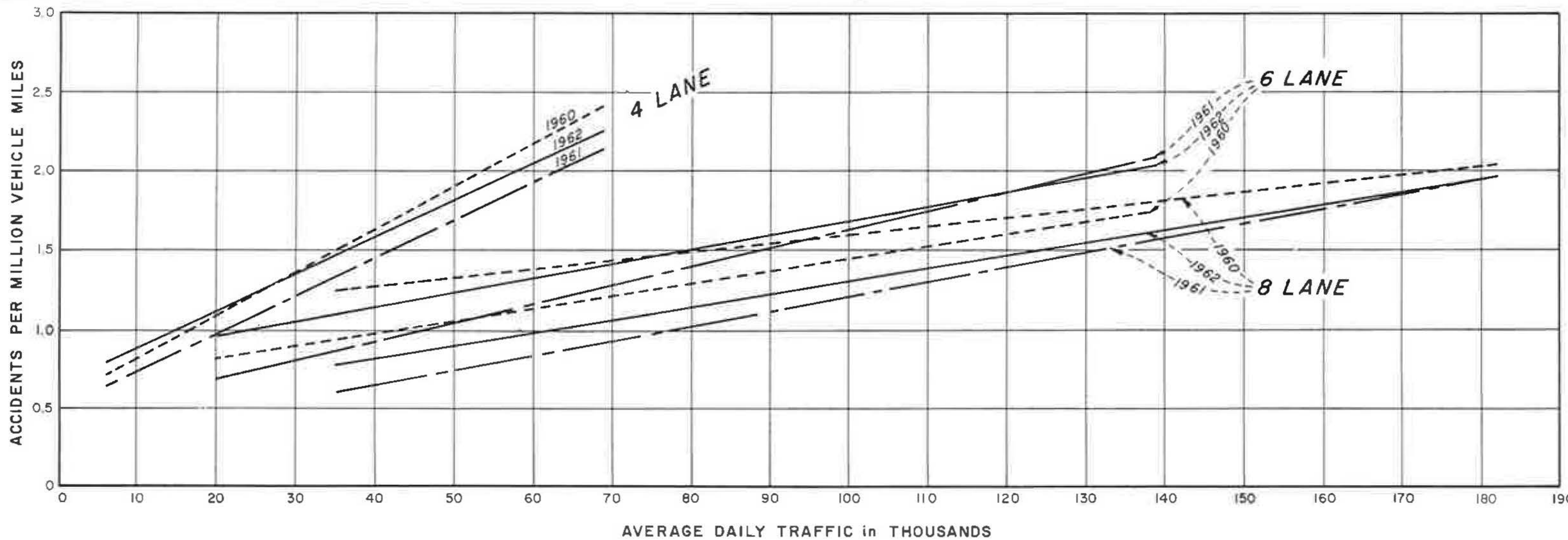


Figure 4. Four-, six-, and eight-lane linear fits for 1960, 1961, and 1962.

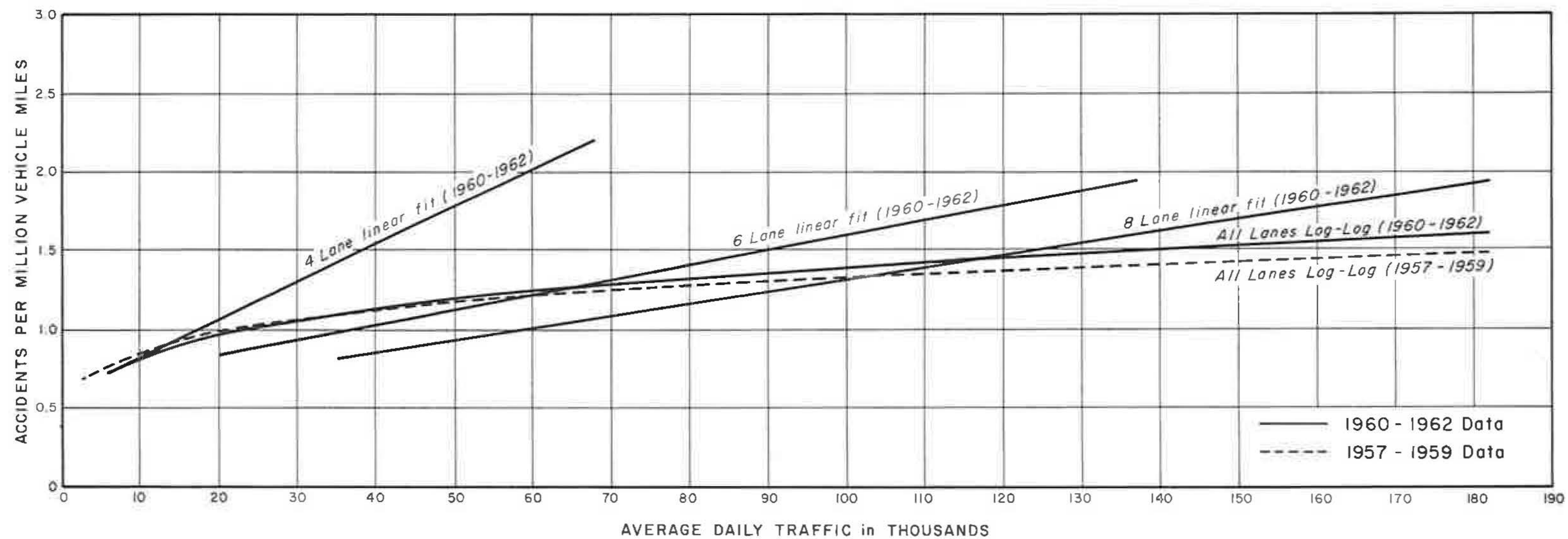


Figure 5. Linear fits (1960-1962) for all four, six, and eight lanes and log-log fits for all lanes (1960-1962 and 1957-1959).

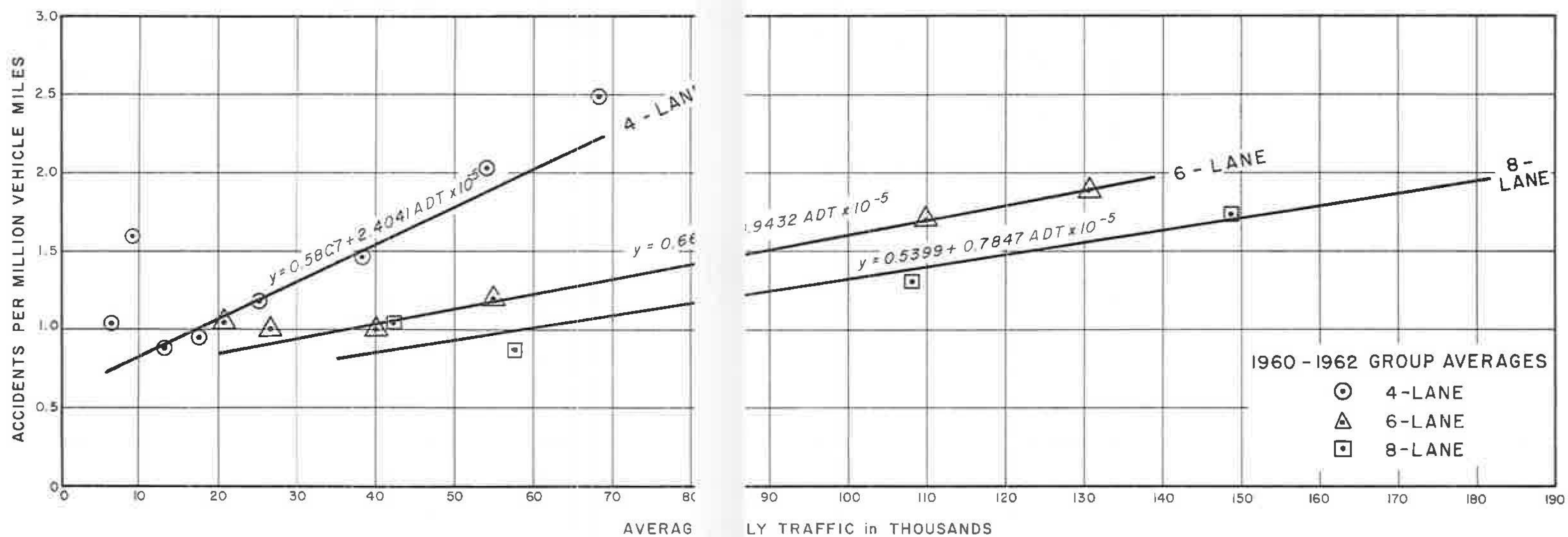


Figure 6. Linear fits and group averages (1960-1962) for four-, six-, and eight-lane freeways.

method of least squares, using each individual freeway segment as a pair of coordinates (y = accident rate, x = ADT). Group average accident rates for several ADT groups (or class intervals of ADT) were superimposed on these curves for purposes of inspection.

Tables 2 through 4 are group average accident rates for each volume class and number of lanes. Each year is tabulated separately and then the 3 yr are combined in Table 5. The ADT class intervals are the same as those used in "Accidents on Freeways in California" (2). The original selection of the class intervals was such as to present six uniform intervals on the logarithmic scale between traffic volumes of 10,000 and 100,000 veh/day.

Figures 1 through 4 show the linear fits obtained through the results of a computer program. The program used also provided fits for four other types of curves along with the standard error of estimate for each type. Table 6 indicates the curve types and the standard errors. For all practical purposes, the linear fit proved as good as any other type and is by far the easiest to understand and work with.

The original curve (2) showing accident rate vs ADT was a log-log equation ($y = ax^b$) that combined the four-, six-, and eight-lane data (1957-1959) and provided the single curve shown in Figure 5. The (1960-1962) log-log equation is also plotted on Figure 5 along with the 1960-1962 four-, six-, and eight-lane linear fits. The lines tilt up when sorted by number of lanes, yet as a group they still approximate the original log-log form.

Figure 6 is a summary of all 3 yr of experience. The two most significant points of interest are apparent:

1. All other things being equal, the accident rate for a four-, six-, or eight-lane freeway will normally increase with an increasing ADT.
2. For any given ADT, a four-lane freeway would be expected to have a higher accident rate than a six-lane freeway and a six-lane a higher rate than an eight-lane freeway.

This phenomenon is apparently characteristic of total accident rates, but not of fatal accident rates. The curves or relationships between fatal accident rates and traffic volumes are not yet available. However, the trends seem to show a slight decrease in the fatal accident rate as the ADT increases.

If the future relationship remains consistent with the past, a sizable reduction in total accidents could be realized by providing more lanes. For example, if a proposed freeway has an estimated future ADT of 60,000 veh/day, it would generate about 22 MVM/mi/yr. Figure 6 shows a probable accident rate of 2.00 accidents per MVM for a four-lane freeway at this ADT. In raw numbers, this reduces to $22 \times 2 = 44$ accidents per mile per year. The six-lane accident rate at 60,000 would be about 1.23, or $1.23 \times 22 = 27$ accidents per mile per year. In other words, 17 accidents per mile per year could be prevented by building a six-lane freeway instead of a four-lane freeway. At this rate, the extra lanes would prevent $20 \times 17 = 340$ accidents per mile in 20 yr.

In this study alone, there were 92 mi of four-lane freeway in the 31,600 to 68,500 ADT range by 1962. If these freeways had been six-lane and if Figure 6 may be used to predict the probable accident rate, there would have been 1,425 accidents in 1962 as compared to the 2,269 accidents which actually occurred. If this rate were to continue for 20 yr (it is more likely to increase since the ADT is constantly increasing), there may be a possiblity of preventing 16,880 accidents in the 20-yr period by adding two lanes to the 92 mi of four-lane freeways.

The number of traffic lanes of a freeway is only one of many factors affecting the accident rate on the freeway. However, increasing the number of lanes, although not necessarily initially required by traffic volume demands, does reduce significantly the number of accidents on the freeway during its lifetime. Therefore, the possibility of providing the ultimate number of lanes on freeways should always be considered for initial construction whenever stage-type construction is contemplated.

REFERENCES

1. The Comparative Freeway Study. Calif. Div. of Highways, April 1964.
2. Accidents on Freeways in California. Theme IV. World Traffic Eng. Conf., 1961.

Discussion

JOHN VERSACE, Ford Motor Co.—The results of this study imply that freeway accidents can be reduced by increasing the number of lanes, even if the traffic volume does not call for such an increase. The accident rate on the larger freeways was less than might be expected because the traffic volume per lane was not reduced that much. However, these results may reflect the peculiarities of the sample as much as anything, so we should be conservative about the conclusions until there is independent evidence.

Although the data have been classified according to whether the freeway had four, six, or eight lanes, we must not rule out the possibility that other factors which covary with the size of freeway might be major causes of the results given. Is it certain that the freeways compared are similar in all respects other than the number of lanes? Is there any way in which the traffic is consistently different for the three types of freeway, as might result from urban vs suburban differences? Were the speeds comparable, on the average, for all the roads? Was the distribution of cars and trucks uniform? And most importantly, was the number of accesses per mile comparable?

It may be useful to scrutinize the data more completely by selecting matched triplets of four-, six-, and eight-lane segments, with the matching based on factors such as these. Or, if all else can be construed as remaining essentially equal, we could compare the before-and-after accident experience of roads that have been widened.

However, the results as they stand do invite some commentary. Why was there [illegible] be one or more factors in addition to the mere change in traffic volume per lane. Traffic volume is a carrier, as it were, of traffic accidents; the more volume, the more opportunities for the conflicts and frictions which lead to accidents. But traffic friction can be caused by other things as well, and previous evidence indicates that the number of accidents increases where there are more conflicts or friction points.

The addition of the third lane produces a qualitiative change in driving behavior by increasing the driver's flexibility. However, additional lanes also increase the opportunities for friction by providing more opportunities for lane crossovers. Furthermore, brief spells of mental confusion and disorientation are more likely to occur to drivers who find themselves embedded in a sea of cars without the fixation point, or focus of orientation, provided by a nearby road edge.

The further reduction in accident rate on the eight-lane freeways was very small and statistically unreliable. The six-lane freeway may be the optimum size. Improvements in both volume and accident reduction might be better realized by multiplying the number of six-lane freeways, even if they are parallel and contiguous but not interacting, rather than by adding more lanes. Channeled traffic, which restricts passing and merging, is an important consideration and should be studied in terms of speed fluctuations.

Finally, this was a cross-sectional study which compared the accident rates on existing freeways. The implication that increasing the number of lanes on existing freeways will reduce accidents is an interpretation. The results of the study are perhaps necessary for such a conclusion, but they are not sufficient. Among other things, a widened freeway encourages greater use, thereby at least partially defeating itself.

STANLEY R. BYINGTON, U. S. Bureau of Public Roads. —In 1906, Sir Oliver Lodge noted in Easy Mathematics, "An equation is the most serious and important thing in mathematics." This review shows how important equations are in understanding relationships such as those described by Mr. Lundy. Treated here is an extension of Mr. Lundy's analysis including the effect on the accident-ADT relationships when a third parameter is introduced.

Recent research on a National Cooperative Highway Research Project (3) hypothesized and confirmed that the MVM rate does not adequately portray the risks associated with traveling along differently designed two-lane highways. The MVM rate does not eliminate the effect of mileage on crude accident data for conventional (uncontrolled-access) two-lane highways. In fact, it was shown that omission of the consideration of study segment length within ADT groupings does distort the MVM rate vs ADT relationship. Reported findings of the research showed that for segments of constant length, the true relationship is a slight decrease of rate with increased ADT, whereas the apparent relationship showed the reverse, i. e., an increase of the rate with increased ADT.

Because regression analysis, which excludes a segment length parameter, can influence and distort the MVM rate vs ADT relationship, an analysis similar to that conducted in the NCHRP study has been applied to Mr. Lundy's data. Regression equations were developed from already prepared computer programs to define a surface in log-log-log space that would pass through the data. The form of the regression equations utilized is actually an extension of the log-log equation ($y = ax^b$) plotted in Figure 5 of Mr. Lundy's report. The statistical relationship of segment length and traffic volume on the number of accidents was determined from the logarithmic transformation of the equation:

$$y = a x_1^{b_1} x_2^{b_2} \qquad (1)$$

where

x_1 = segment length (mi);
x_2 = ADT (thousands of veh/day);
y = number of accidents per year per segment; and
a, b_1, b_2 = constants determined from data by method of least squares.

The regression calculations gave the estimate of the coefficients, a, b_1, and b_2 indicated in Table 7. The length elasticity coefficient b_1, is nearly the same and is less than unity, regardless of the number of freeway lanes. What this means is that for every 1 percent difference in segment length, there is a smaller percent difference in the number of accidents. For example, on 5-mi road segments, there are less than 5 times the number of accidents as on 1-mi road segments. This finding implies that inclusion of segment length in the denominator of an accident rate does not actually remove the effect of length. This is true unless segment lengths and their distribution are nearly the same for all ADT groupings. A question arises concerning what effect the coefficients given in Table 7 have on Mr. Lundy's statement that "... accident rates on freeways increase as the traffic volumes increase, and for a given traffic volume, the accident rates decrease as the number of lanes increase." The summary of the effects of the elasticity coefficients on the MVM rate is best shown pictorially.

TABLE 7

VALUES OF LOGARITHMIC EQUATION COEFFICIENTS[a]

No. of Lanes	a	b_1	b_2
4	-0.544	0.816	1.231
6	-1.139	0.856	1.483
8	-2.074	0.878	1.891

[a]Lundy's data.

Figure 7 illustrates that when the segment length is held constant and ADT increases, the MVM rate also increases.

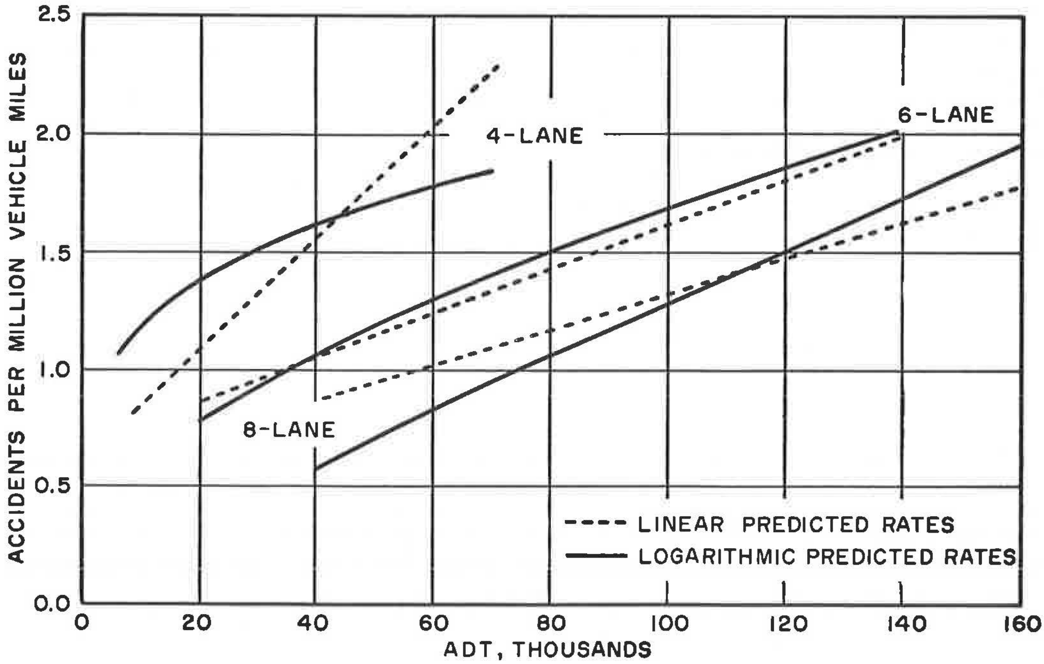


Figure 7. Accidents per MVM vs ADT, Lundy's data (2-mi study segments).

TABLE 8

AVERAGE SEGMENT LENGTHS BY TRAFFIC VOLUME AND NUMBER OF LANES[a]

Traffic Vol. (1,000)	Mean Segment Length (mi)	Frequency of Section	Traffic Vol. (1,000)	Mean Segment Length (mi)	Frequency of Section	Traffic Vol. (1,000)	Mean Segment Length (mi)	Frequency of Section
6-13.9	15.7	16	20-39.9	5.1	16	45- 66.9	4.4	11
14-17.9	14.7	16	40-52.9	5.3	17	67- 98.9	3.1	12
18-24.9	6.8	17	53-72.9	7.7	17	99-127.9	5.0	12
25-29.9	8.7	17	73-98.9	5.7	17	128+	4.7	10
30-40.9	7.6	16	99+	5.0	15			
41+	3.6	16						

[a]Lundy's data.

TABLE 9

LOGARITHMIC PREDICTED ACCIDENT RATES PER MVM, SIX- AND EIGHT-LANE FREEWAYS[a]

Segment Length (mi)	Accidents per MVM[b]							
	Six Lanes				Eight Lanes			
	25 ADT	50 ADT	75 ADT	100 ADT	60 ADT	90 ADT	120 ADT	150 ADT
2	85	119	145	167	81	116	150	184
4	77	108	132	150	75	107	138	179
6	72	101	124	142	71	102	131	161
8	70	98	119	137	68	98	127	155

[a]Lundy's data. [b]For ADT groups in thousands.

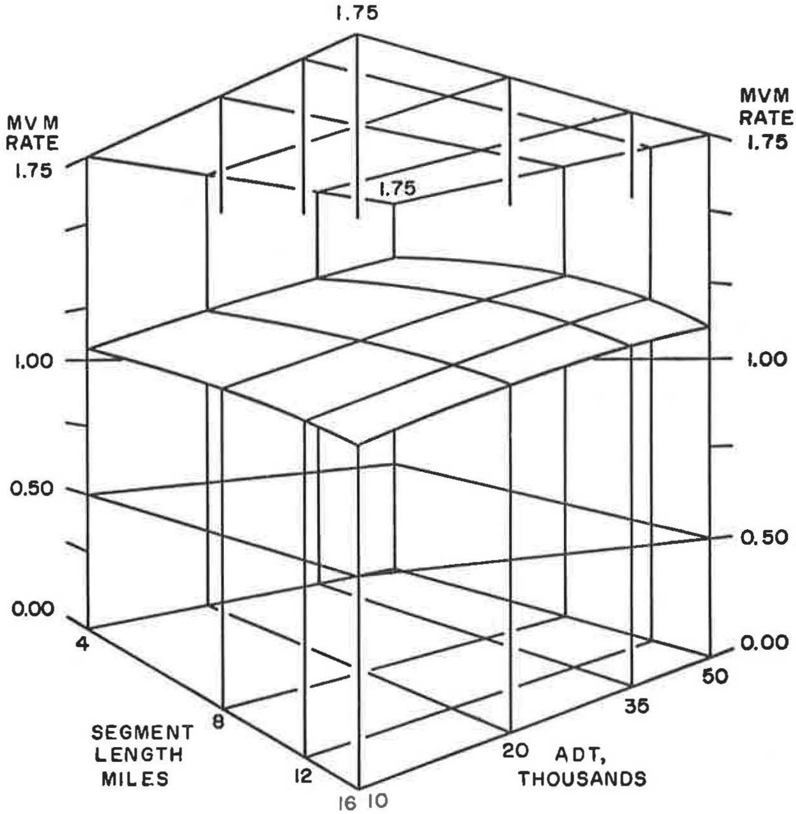


Figure 8. Logarithmic predicted accident rate per MVM, four-lane freeways, Lundy's data.

This figure also shows that for the same segment length and for a given traffic volume, the accident rate decreases as the number of lanes increase. Using Mr. Lundy's example of a proposed freeway with an estimated future ADT of 60,000 veh, Figure 7 indicates that for four- and six-lane highways the logarithmic regression curves predict probable accident rates of 1.77 and 1.30 accidents per MVM, respectively. The accident savings would then be 10 accidents per mile per year [(1.77 - 1.30) accid./ MVM × 22 MVM] rather than the 17 found by Mr. Lundy. As the logarithmic curves in Figure 7 are for constant segment lengths of 2 mi, there is the problem of how the accident savings vary when other segment lengths are used. For 4-, 8-, and 12-mi segments, the computed savings amounted to 8, 7 and 6 accidents per mile per year, respectively. Resultant differences in accident savings as predicted by the linear and logarithmic equations decrease for ADT values less than that used in this example. Thus, Mr. Lundy's statements on the effect of the number of lanes and ADT on accident rates are sufficiently accurate for those volumes below which additional lanes are not normally justified by the traffic volume demand, i.e., 50,000 ADT for four lanes and 75,000 for six lanes.

Two additional observations can be derived from Figure 7. First, regardless of the length segment, the accident savings predicted by the logarithmic equations remain nearly constant for all ADT values. Second, this figure shows the effect of unequal segment lengths within different ADT groupings: whereas the logarithmic curves closely approximate the linear curves for the six- and eight-lane freeways, there is an evident difference between the four-lane freeway curves. This is explained by the data given in Table 8, the ADT groupings of which were selected to yield as equal a frequency of study segments as possible. There is a distinct decreasing of average segment lengths for the four-lane freeway segments studied by Mr. Lundy, whereas the segment lengths are nearly the same for all ADT groups of six- and eight-lane freeway segments studied.

Because Figure 7 offers a limited view of the combined effect of segment length and ADT on the MVM rate, Figure 8 is presented to show for four-lane freeways the MVM rate as a surface above an ADT-length base. The height of the surface above a set of segment length-ADT coordinates indicates the accident rate; the higher the surface, the higher is the rate. This figure reveals that the accident rate decreases with increased segment length and increases with ADT. Table 9 indicates that figures for six- and eight-lane freeways would have produced similar surfaces.

Reference

3. Kihlberg, J. A., Campbell, B. J., and Tharp, K. J. Analysis of Motor Vehicle Accident Data as Related to Highway Classes and Design Elements. Cornell Aeronautical Lab., Aug. 1964.

JOSEPH S. CHAMPAGNE, Port of New York Authority. —Many factors contribute to accidents on our highways. Generally these are alignment, cross-section, access control, the driver, the weather, and highway capacity. Mr. Lundy has pointed out that accident rates increase with increased traffic volume and that accidents are proportional to traffic volumes. The signficant difference between a four-, six-, or eight-lane freeway is the rate at which the accident rates increase. Four lanes had a rate of increase of 0.24 accidents per MVM, six lanes of 0.094 accidents per MVM, and eight lanes of 0.078 accidents per MVM.

[illegible]

This is due to the number of traffic lanes and the other factors previously mentioned. Also, passing opportunities are more frequent on six- and eight-lane highways than on the four-lane highway, which in part contribute to lower accident rates on the six- and eight-lane highways. The additional lanes reduce the motorist's need to take unnecessary risks.

Going one step further, I compared accident rate and ADT to highway capacity (Figs. 9 and 10, Table 10). The basic data were obtained from Figure 6.

In Mr. Lundy's example of a freeway with an estimated future ADT of 60,000 veh/day, the four-lane facility was estimated at 100 percent capacity and the six- and eight-lane facilities were estimated at 65 percent and 50 percent of capacity, respectively. It is assumed that 10 percent of ADT volume is equivalent to the peak hour volume and the practical capacity is 1,500 veh/lane/hr (4). Tables 11, 12 and 13 give accident numbers and rates when all the facilities have reached 100 percent capacity and when the four-lane facilities are at 100, 65 and 50 percent of capacity.

The results of the example, given in Table 11, indicate fewer accidents on the six- and eight-lane facilities than on the four-lane. But the six- and eight-lane facilities are only operating at 65 and 55 percent of capacity. Table 12 indicates that when facilities are operating at 100 percent capacity, there is little difference in the number of accidents per mile per year. Table 13 gives the effect on accidents when the four-lane facilities are operating at and below capacity. The accident rate is virtually the same for the six- and eight-lane facilities when they are operating at the same percent of capacity.

Mr. Lundy's paper emphasizes the fact that we should not wait for a facility to become a parking lot before we decide to expand. We should continually monitor our facilities whether they be two, four, six, eight or ten lanes and be prepared to act before the accident rate increases above the 100 percent capacity rate. As his example

TABLE 10

DATA ON FOUR-, SIX-, AND EIGHT-LANE HIGHWAYS[a]

ADT (1,000)	Four Lanes		Six Lanes		Eight Lanes	
	Capacity (%)	Accident Rate per MVM	Capacity (%)	Accident Rate per MVM	Capacity (%)	Accident Rate per MVM
10	16.6	0.82	11.1	-	8.3	-
20	33.20	1.08	22.2	0.86	16.6	-
30	49.80	1.31	33.3	0.94	24.9	-
40	66.40	1.55	44.4	1.04	33.2	0.85
50	83	1.80	55.5	1.13	41.5	0.94
60	99.60	2.03	66.6	1.23	49.8	1.02
70	116.20	2.27	77.7	1.32	57.5	1.09
80	132.80	-	88.8	1.41	66.4	1.16
90	149.40	-	99.9	1.50	74.7	1.24
100	166	-	111.0	1.60	83	1.31
110	182.60	-	122.1	1.70	91.3	1.40
120	199.20	-	133.2	1.80	99.6	1.48
130	215.80	-	144	1.88	108	1.56
140	232	-	156	1.98	116.2	1.63
150	249.0	-	167	-	124.5	1.71
160	265.6	-	178	-	132.8	1.78
170	282.2	-	189	-	142	1.87
180	298	-	200	-	150	1.94

[a]ADT and accident rate taken from Figure 6 of Lundy's report.

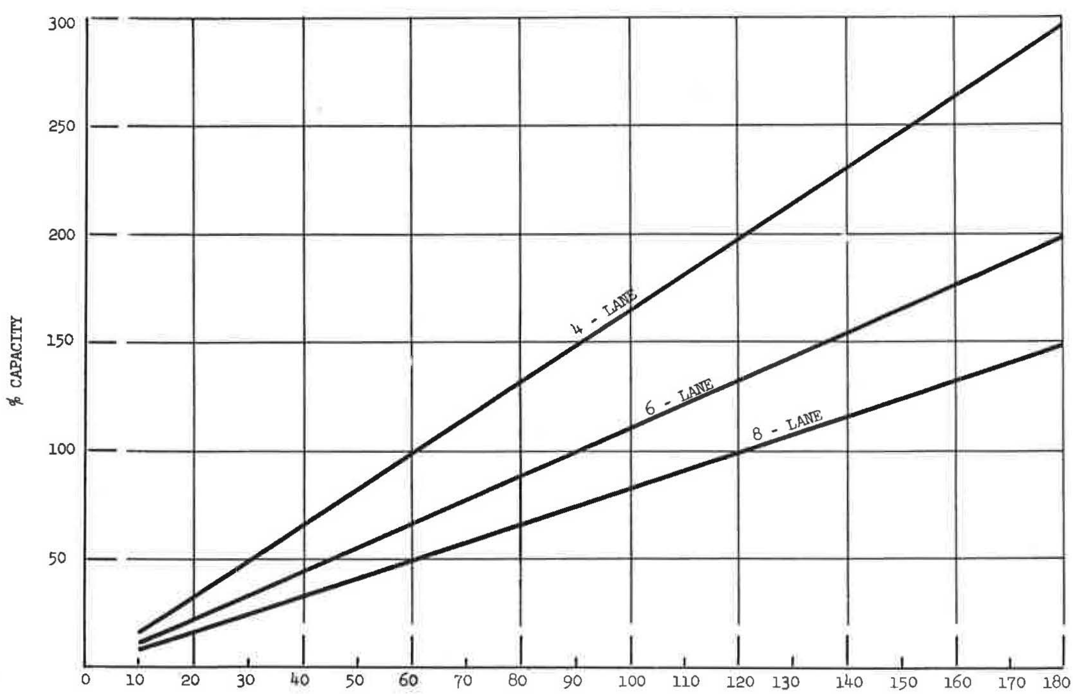


Figure 9. Percent capacity vs ADT for four-, six-, and eight-lane freeways.

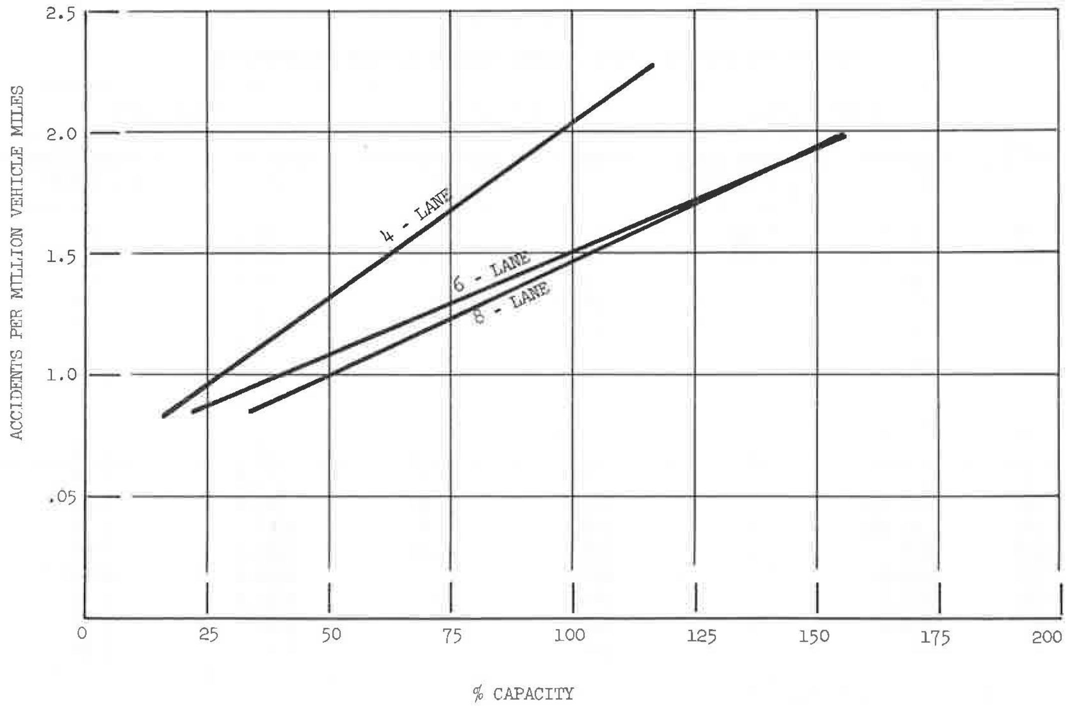


Figure 10. Accidents per MVM vs capacity for four-, six-, and eight-lane freeways.

No. of Lanes	Capacity (%)	Accident Rate	No. Accidents/Mi/Yr
4	100	2.2	44
6	65	1.2	14
8	50	1.15	13

[a]ADT = 60,000 veh, 22 veh-mi/mi/yr.

No. of Lanes	ADT (1,000)	Accident Rate	MVM/Mi/Yr	No. Accidents/Mi/Yr
4	60	2.2	22	44
6	90	1.5	32	48
8	120	1.45	43	62

TABLE 13

FOUR-LANE HIGHWAY AT VARIOUS CAPACITIES

Capacity (%)	ADT (1,000)	Rate	MVM/Mi/Yr	No. Accidents/Mi/Yr
100	60	2.2	22	44
65	40	1.5	14.6	21
50	30	1.0	11	11

shows, we should not permit highways to exceed capacity because of the increase in the number of accidents and also because of the increase in delays and the inability of the facility to handle unusual increases in peak hour volumes.

Reference

4. Geometric Highway Design. Table II-9, p. 86. AASHO.

R. A. LUNDY, Closure—Mr. Versace, Mr. Byington, and Mr. Champagne have presented interesting and valuable discussions of this report. These ideas will serve as useful reference in future research of this nature.

One of the questions raised by Mr. Versace concerns the basic data used in the study. It is granted, as Mr. Versace explained, that the freeways studied were not exactly equal in every respect other than in the number of lanes; however, from a practical standpoint, it would be difficult to say that they are significantly different. Every effort was made to exclude abnormalities from the data.

It was suggested that matched triplets of four-, six-, and eight-lane segments be studied in lieu of massing data which was the method of this study. These triplets, matched in design, volume content, weather conditions, driver traits, etc., are extremely difficult to find and in the final analysis it would probably be necessary to build models to specifications, which, of course, is impractical. If the models did exist, they would be equal in ADT and hence no ADT vs rate relationship would be obtained.

Mr. Versace surmised that the six-lane freeway may be the optimum size. This interpretation cannot be drawn from the California report; the evidence points to exceptionally low accident rates on eight-lane freeways and there is no evidence that drivers become "lost in a sea of cars" when operating on an eight-lane facility. On the contrary, eight-lane freeways provide far more passing opportunities (hence, less lane blockage) and greater areas for emergency maneuvers. If these lanes were channelized with curbing or some other physical barrier, this advantage would be lost and accident concentrations would no doubt occur at those locations where cross-over is permitted.

The National Cooperative Highway Research Project (3), which formed the backbone of Mr. Byington's discussion, utilized 1961-1962 California (conventional two-lane) data in the development of the accident rate vs segment length relationship. These data were obtained from routine annual tabulations (TS-5.0 tables) produced by the California Division of Highways. In October 1964, the division was asked to comment on the project and those comments pertinent to the accident rate vs segment length relationship were as follows:

> Each highway segment shown in the TS-5.0 tabulations is homogeneous in respect to number of lanes, access control, and whether divided or not; but it is not necessarily homogeneous in respect to traffic volume; i.e., the range of traffic volume within a section may be very large. Since one of the primary factors that the Laboratory, and we independently, have found to affect accident rates or numbers is the magnitude of traffic volumes, the Laboratory findings in this respect may have large errors. The magnitude or direction of these probable errors are unknown and a great deal of analysis would be required to determine them.
>
> The Laboratory further found that the accident rate and accident numbers are related to the segment length. This is undoubtedly true and as the Laboratory pointed out, it is probably due to the fact that the shorter homogeneous highway sections usually contain those roadside and highway geometry features which are normally associated with accident causation. In using the relationships that have been established for the prediction of accidents for a particular section of highway, one must remember to enter the report graphs or tables with the maximum highway segment length which is homogeneous in all respects and which includes the piece of highway under consideration. Otherwise, if the length of the piece under consideration only is used, the curves or tables will be entered at an accident rate portion of the data which is too high.

These comments were, of course, based on the study containing only two-lane conventional highway data. Mr. Byington's application utilized four-, six- and eight-lane California freeway data and the ADT is quite stable within these segment lengths; however, the segment lengths were created for a number of reasons besides ADT, including legislative boundaries which certainly do not affect the accident rate. Many of the sections do, however, terminate at some distinctly different type of facility or the break is introduced at a point of major volume change. These end conditions could, and probably do, affect the accident rate on shorter sections. However, the California data must be reworked and broken into nonbiased sections before accurate distance coefficients can be calculated.

Mr. Champagne's application of the study data is interesting and informative. The significance of his comment, "Table 12 indicates that when all facilities are operating at 100 percent capacity, there is little difference in the number of accidents per mile per year," is not clear. The statement is correct; however, it should be pointed out that the six-lane freeway is allowing 50 percent (and the eight-lane, 100 percent) more vehicles to use the freeway without accident.

Comparative Freeway Study

JOHN VOSTREZ and RICHARD A. LUNDY, California Division of Highways

Thirty-three sections (200 mi) of freeways with widely divergent accident rates were analyzed. A total of 11,384 accidents were included. These accidents occurred during the course of 9,198 million vehicle-miles (MVM) of travel. The average accident rate for the 33 sections is 1.24 accidents per MVM. The accident rates for the individual sections range from 4.52 to 0.60 accidents per MVM.

The primary purpose of the first part of the study was to obtain information concerning freeway design characteristics as related to accident frequency. The emphasis is on the relative safety value of the various design features. Human factors were considered homogeneous throughout the sections unless they obviously played an unusual role in a section's accident history. The second part attempted to analyze each section and explain why that section had an unusually good or poor accident history.

•FOR STUDY purposes, freeways are broken down into various lengths called sections. These sections are either geographically separate or section breaks are established by virtue of a change in traffic volume, accident rate, design characteristics, or year built.

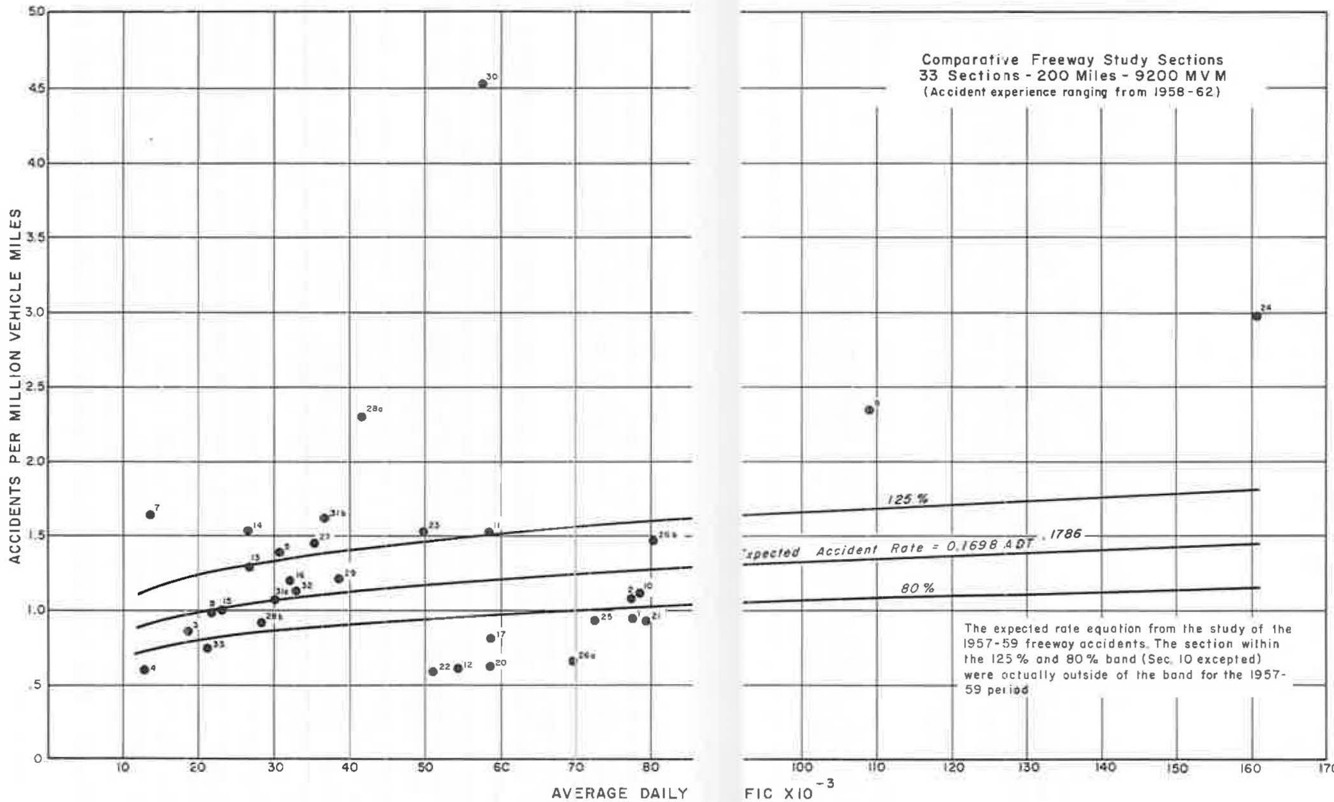
Many sections of freeway have accident rates that vary significantly from the expected rate. This was discovered by a freeway accident-rate study by this Department in 1961, using data for 1957 to 1959, inclusive (1). In the previous study the expected accident rate was defined by the equation $R = 0.\overline{1}7\,V^{0.18}$, where R is the expected rate and V is the average daily traffic. If a freeway accident rate fell within 25 percent above or 20 percent below the expected rate, it was considered average. Those sections of freeway with accident rates that did not fall within these limits were considered to have a significant variation and were taken as subject matter for this comparative freeway study (Figs. 1 and 2).

The study reported here was initiated as an attempt to learn what factors in the non-average freeway sections were responsible for the higher or lower accident rates. Human factors were considered homogeneous throughout the sections unless they obviously played an unusual role in a section's accident history. In this report the emphasis is on the relative safety value of various design features.

A pilot study, involving sections 1 to 14, inclusive, was first made to determine the best approach to the study problems. A 3-yr period of accident experience, 1958 to 1960, was used. On completion of the pilot study, more recent data were available for the remaining sections and the period 1959 to 1961 was used with the exception of sections 16, 29, 30, 31, and 33 (Table 1).

The earlier study revealed 31 out of 94 sections as non-average. Accident rates based on the more recent data showed eleven of these sections actually possessed average accident rates according to the expected rate equation. That is to say, eleven of the sections that had been thought widely variant actually turned out to be normal, when greater experience became available. Even among the other 20 sections, the low rates increased and the high rates decreased, approaching the average as more experience accrued. The "expected-rate" equation was calculated from 1957 to 1959 data

Paper sponsored by Committee on Highway Safety.



SECTION #	1957-1959 DATA: ACTUAL ACC./MVM	1957-1959 DATA: EXPECTED ACC./MVM	1957-1959 DATA: % OF EXPECTED	STUDY PERIOD DATA: ACTUAL ACC./MVM	STUDY PERIOD DATA: EXPECTED ACC./MVM	STUDY PERIOD DATA: % OF EXPECTED
1	0.79	1.30	61	0.95	1.26	75
2	0.73	1.30	56	1.09	1.26	86
3	0.73	0.95	77	0.86	0.98	87
4	0.66	0.90	73	0.60	0.90	67
5	1.53	1.10	139	1.38	1.07	129
7	1.65	0.90	183	1.64	0.92	178
8	0.63	1.10	57	0.98	1.00	98
9	1.96	1.30	151	2.35	1.35	174
10	1.48	1.25	118*	1.12	1.26	89
11				1.51	1.20	126
12	0.46	1.20	38	0.62	1.18	52
13	1.78	1.10	178	1.29	1.03	125
14	1.72	1.10	156	1.52	1.03	148
15	0.51	1.00	51	1.00	1.01	99
16	0.48	1.00	48	1.20	1.07	112
17	0.74	1.20	62	0.82	1.20	68
20	0.51	1.20	43	0.63	1.20	53
21	0.58	1.20	48	0.94	1.27	74
22	0.38	1.20	32	0.60	1.17	51
23	1.54	1.20	128	1.52	1.16	131
24	1.45	0.95	153	2.98	1.44	206
25	0.77	1.25	62	0.94	1.25	75
26a				0.66	1.24	53
26b				1.47	1.27	116
27	1.49	1.10	135	1.45	1.10	132
28a	1.88	1.10	171	2.30	1.13	204
28b				0.91	1.05	87
29	0.58	1.10	53	1.21	1.12	108
30	2.08	1.20	173	4.52	1.20	376
31a	1.42	1.10	142	1.07	1.06	101
31b				1.62	1.10	147
32	1.55	1.10	141	1.13	1.08	104
33	0.33	0.95	32	0.75	0.99	76

*This portion included in study due to the known unusual nature of the alignment.

Figure 1. Accide ate vs average daily traffic.

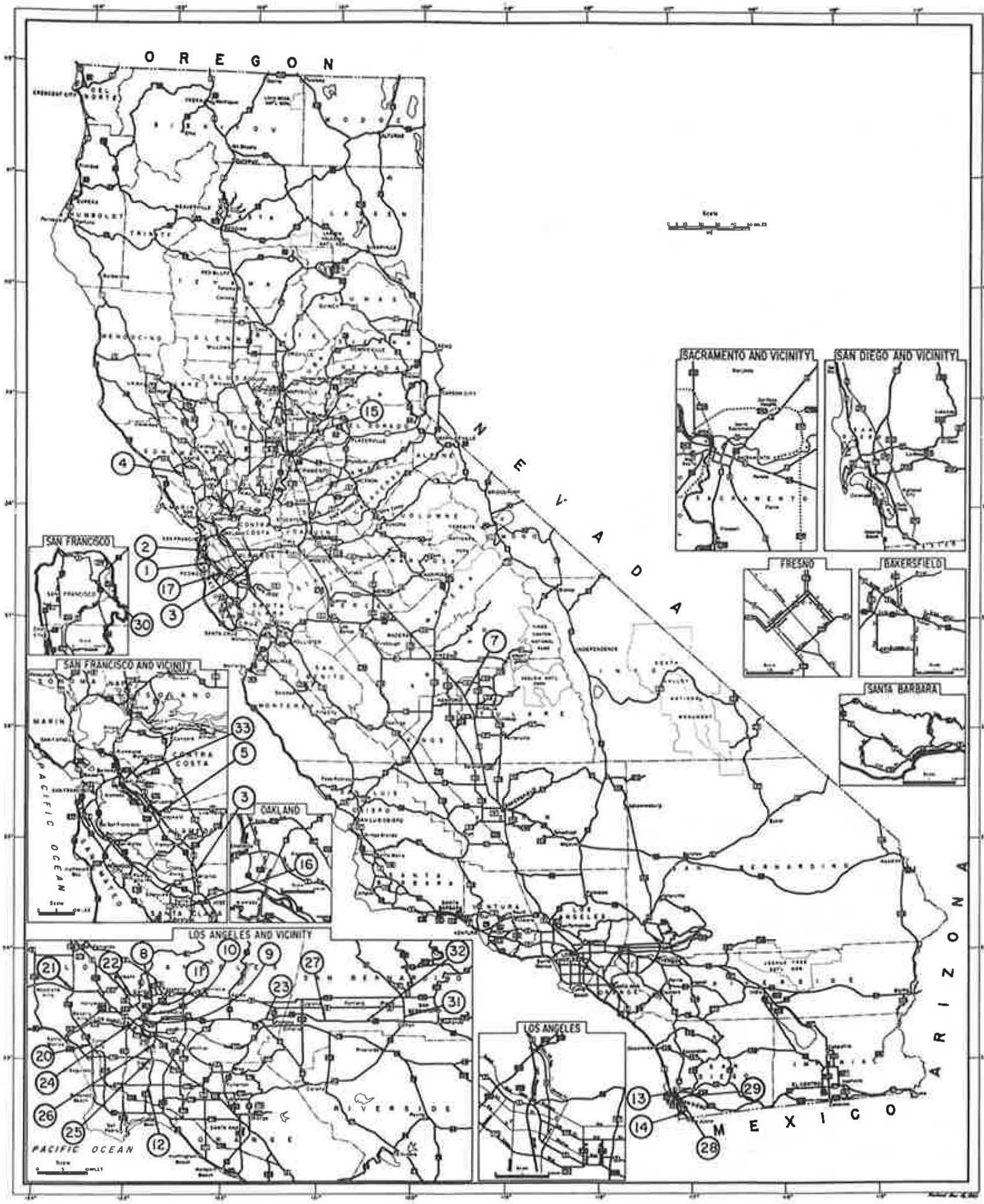


Figure 2. Study section locations.

and it is understandable that some of the sections possess accident rates that fluctuate from year to year in this manner. Some of these sections were completed and open to traffic for only a short time during the 1957 to 1959 period.

The study is divided into two parts. The first is intended as a guide to the type and range of variables encountered in a study of this nature. The various tables and graphs represented only the sections which are encompassed by this study. The limitations will no doubt be evident as each concept presented could well be a major study in its own right. The intent here is to point out some of the variables and develop a broad

TABLE 1

COMPARATIVE FREEWAY STUDY SECTIONS

Section No.	Description	Study Period	Comments
1	Bayshore Fwy.; SM-68-Var Bransten Rd. to N.C.L. of S. San Francisco	1958-1959-1960	
2	Bayshore Fwy.; SM, SF-68-Var. N.C.L. of S. San Francisco to 3rd St. in San Francisco	1958-1959-1960	
3	Nimitz Fwy.; Ala, SCl-69-SJs, A Rt. 68/69 separation to Warm Springs separation (Rt. 5)	1958-1959-1960	
4	Petaluma Fwy.; Son-1-F, C 2.23 mi N. of Marin Co. line to S.C.L. of Santa Rosa	1958-1959-1960	
5	Castro Valley Link; Ala-228-A, SLn. Rt. 69 to jct. Rt. 5	1958-1959-1960	
6	No section		
7	U.S. 99; Tul-4-F Visalia Airport interchange to 1 mi N. of Goshen	1958-1959-1960	
8	Colorado Fwy.; LA-161-LA, Pas, S.Pas Eagle Vista Dr. to Holly St.	1958-1959-1960	
9	Pasadena Fwy.; LA-165-LA 4-level structure to jct. Rt. 205	1958-1959-1960	
10	Pasadena Fwy.; LA-205-Pas, S.Pas. Jct. Rt. 165 to Ave. 64	1958-1959-1960	
11	Pasadena Fwy.; LA-205-Pas, S.Pas. Ave. 64 to Orange Grove Ave. interchange	1958-1959-1960	
12	Long Beach Fwy.; LA-167-LBch, A, Com, Lyn, SGt. Pacific Coast Highway to Atlantic Blvd.	1958-1959-1960	
13	Balboa Bypass; SD-2-SD 1 mi. S. of Balboa Ave. to 0.65 mi. N. of Balboa Ave.	1958-1959-1960	
14	Montgomery Fwy.; SD-2-SD, ChV, G, NatC. San Ysidro junction to S.C.L. of National City	1958-1959-1960	
15	Roseville Fwy.; Sac;Pla-3;3, 17-B;A, A, Rsv. Howe Ave. in Sac. Co. to N. of Atlantic St. in Placer Co.	1959-1960-1961	Portion of section open Jan. 1960 (2,000 ft E. of Folsom Rd. to N. of Atlantic St.), 1.7 mi
16	Los Gatos Fwy.; SCl-5;239-LGts, D, Cmb, SJs;D, SJs. Santa Cruz Ave. to The Alameda	1960-1961	Not open full year of 1959; portion of section open June 1960 (Bascom Ave. to The Alameda), 1.0 mi.
17	Bayshore Fwy.; SM-68-D, MlP, RdwC. Santa Clara Co. line to Bransten Rd.	1959-1960-1961	
18	No Section		
19	No section		
20	San Diego Fwy.; LA-158-ClC, LA, A Approx. 0.2 mi N. of Venice Blvd. to Ovada Pl.	1959-1960-1961	Most of section open Feb. 1959 (0.2 mi N. of Venice Blvd. to 0.5 mi N. of
22	Golden State Fwy.; LA-161;4-LA;Gndl, LA Glendale Blvd. to Burbank Blvd.	1959-1960-1961	Part of section opened Aug. 1959 (Alameda St. to Burbank Blvd.), 1.3 mi
23	San Bernardino Fwy.; LA-26-Cla, Pom, C, W.Cov. San Bernardino Co. line to Citrus Ave.	1959-1960-1961	
24	Harbor Fwy.; LA-165-LA 30th St. to 4-level structure	1959-1960-1961	
25	Long Beach Fwy.; LA-167-Com, A, Lyn, SGt. Atlantic Ave. to Firestone Blvd.	1959-1960-1961	
26a	Long Beach Fwy.; LA-167-Bell, B, Ver. Firestone Blvd. to N. jct. Atlantic Blvd.	1959-1960-1961	
26b	Long Beach Fwy.; LA-167-Bell, B, Ver, Cmrc. N. jct. Atlantic Blvd. to Olympic Blvd.	1959-1960-1961	
27	San Bernardino Fwy.; SBd-26-Mcl, Upl, Ont, D Live Oak Ave. to Los Angeles Co. line	1959-1960-1961	Part of section open Aug. 1959 (500 ft W. of Turner Ave. to 800 ft W. of Milliken Ave.), 1.7 mi
28a	U.S. 395; SD-77-SD N. of jct. Rt. 200 to Genesee Ave.	1959-1960-1961	
28b	U.S. 395; SD-77-SD Genesee Ave. to Clairmont Mesa Blvd.	1959-1960-1961	
29	S.S. Rt. 94; SD-200-SD, A W. of 25th St. O.C. to E. of Palm Ave.	1960-1961-1962	1959 data not used because part of section (W. of 25th St. to 32nd St.) not open until Dec. 1959, 1.9 mi
30	Central Fwy.; SF-2-SF Jct. Rt. 68 to Turk St.	1960-1961	Not open full year of 1959
31a	Riverside; SBd-43-F, Col, SBd. Riverside Co. line to jct. Rt. 26 & 650 ft S. of Mill St. to jct. Rt. 31	1960-1961	Not open full year of 1959
31b	Riverside Fwy.; SBd-43-F, Col, SBd. Jct. Rt. 26 (San Bernardino Fwy.) to 650 ft S. of Mill St.	1960-1961	Not open full year of 1959
32	San Bernardino Fwy.; SBd-26-D, Ria, Col. 5th St. in Colton to Live Oak Ave.	1960-1961	Not open full year of 1959; 2.1 mi portion (Live Oak Ave. to Cypress Ave.) not open until March 1960
33	Warren Fwy; Ala-227-Oak Redwood Rd. to Thornhill Dr.	1960-1961	Portion of section not open until March 1960 (Lincoln Ave. to Redwood Rd.), 0.97 mi

TABLE 2 - SUMMARY OF DATA

Group	Section	Freeway	Location	Length (mi.)	ADT	No. of Lanes	Lane Width	Median Width	Paved Shldr. Lt.	Paved Shldr. Rt.	Total Acc.	MVM	Actual Acc/MVM	Expected Acc/MVM	% of Expected
HIGH ACCIDENT SECTIONS	50	Central	SF-2-SF Jct. Rte. 68 to Turk St.	1.47	57,800	4 & 6	12'	Double Deck	2'	2'	280	62	4.52	1.20	376
	24	Harbor	LA-165-LA 30th St. to 4-Level	3.20	160,700	8	12'	Curbed 10-16'	2'	6'	1675	563	2.98	1.44	206
	28A	U.S. 395	SD-77-SD No. of Jct. Rte. 200 to Genesee Ave.	4.75	41,300	4	12'	12 & 16' Curb to 34'	2'	6'	494	215	2.30	1.13	204
	7	U.S. 99	Tul-4-? Visalia Airport Interchange to 1 mi. No. of Goshen	3.67	13,700	4	12'	46'	2'	6'	90	55	1.64	0.92	178
	9	Pasadena	LA-165-LA 4-Level to Jct. Rte. 200	1.90	108,100	4 & 6	4-12' 2-11' / 4-11' 4-10'	4' Sep. Ave.	2'	Some 5' back of C. Curb	553	227	2.35	1.35	174
	14	Montgomery	SD-2-SD, ChV, C, NatC San Ysidro Jct. to S.C.L. National City	9.16	26,300	4	12'	36-40'	5'	8'	400	264	1.52	1.03	148
	31B	Riverside	SBd-43-F, Col, SBd Jct. Rte. 26 (SBd Fwy.) to 650' So. of Mill St.	2.25	36,500	4	12'	46'	2'	10'	97	60	1.62	1.10	147
	27	San Bernardino	SBd-26-[illegible] [illegible] to L.A. Co. Line	13.56	35,100	4	12'	16-46'	2-8'	8-10'	757	522	1.45	1.10	132
	23	San Bernardino	LA-26-[illegible] SBd. Co. Line to [illegible]	11.12	49,900	6	12'	16'	2'	8'	922	606	1.52	1.16	131
	5	Castro Valley Link	Ala-228-A, SL Rte. 68 to Jct. Rte. 5	2.05	30,700	4	12'	Curbed 16'	2'	6'	95	69	1.38	1.07	129
	11	Pasadena	LA-205-Pas, S. Pas Ave. 64 to Orange Grove Ave. Interch.	1.10	58,100	6	4-11' 2-10'	6'	2'	2'	106	70	1.51	1.20	126
	13	Balboa Bypass	SD-2-SD [illegible] 0.65 mi. No. of Balboa Ave.	1.65	26,600	4	12'	Variable	2'	6'	62	45	1.29	1.03	125
		Total or Average		55.90							3511	2743	1.99		
AVERAGE ACCIDENT SECTIONS	26B	Long Beach	LA-167-Bell, B, Ver, Cmrc No. Jct. Atlantic Blvd. to Olympic Blvd.	1.54	80,000	6	12'	40'	2'	8'	199	135	1.47	1.27	116
	16	Los Gatos	SCl-5, 239-[illegible] [illegible]	10.20	32,000	4	12'	34-46'	2'	8'	296	238	1.20	1.07	112
	29	S.S.R. 94	SD-200-SD, A West of 28th St. O.C. to E. of Palm Ave.	8.52	39,800	6	12'	66'	2-8'	8-10'	449	372	1.21	1.12	108
	32	San Bernardino	SBd-26-[illegible] [illegible] to Live Oak Ave.	8.23	33,000	4	12'	46-64'	2'	8'	222	197	1.13	1.08	104
	31A	Riverside	SBd-43-F, Col, SBd Riverside Co. Line to Jct. Rte. 26 & 650' So. of Mill St. to Jct. Rte. 31	6.50	30,000	4 & 6	12'	56'	2'	10'	148	138	1.07	1.06	101
	15	Roseville	Sac, Pla-3, 17-[illegible] [illegible] Co. to No. of Atlantic St.	14.40	23,000	4	12'	36'	2'	8'	359	359	1.00	1.01	99
	8	Colorado	LA-161-LA, Pas, S. Pas Eagle Vista Dr. to Holly St.	2.00	21,900	4	12'	12' Curb to 36'	2'	8'	47	48	0.98	1.00	98
	10	Pasadena	LA-205-Pas, S. Pas Jct. Rte. 165 to Ave. 64	3.50	78,500	6	11'	6'	2'	2'	337	301	1.12	1.26	89
	28B	U.S. 395	SD-77-SD Genesee Ave. to Clairemont Mesa Blvd.	3.75	28,200	4	12'	12 & 16' Curb to 34'	2'	6'	106	116	0.91	1.05	87
	3	Nimitz	Ala, SCl-69-[illegible] [illegible] to Warm Springs Sep.	7.75	18,300	4	12'	40'	2'	6'	134	155	0.86	0.98	87
	2	Bayshore	SM, SF-68-[illegible] N.C.L. of So. San Francisco to 3rd St. in S.F.	3.20	77,100	6	12'	12' Curb to 36'	2-5'	8'	294	270	1.09	1.26	87
		Total or Average		69.39							2581	2329	1.11		
LOW ACCIDENT SECTIONS	33	Warren	Ala-[illegible] Redwood Rd. to Thornhill Rd.	2.64	21,000	4	12'	36-40'	2-5'	8'	30	40	0.75	0.99	76
	1	Bayshore	SM-68-[illegible] Broadway Rd. to N.C.L. of So. San Francisco	15.80	77,500	6	12'	36'	2-6'	8'	1271	1341	0.95	1.26	75
	25	Long Beach	LA-167-Com, A, Lyn, SGP Atlantic Ave. to Firestone Blvd.	4.98	72,400	8	12'	40'	2'	8'	370	393	0.94	1.25	75
	21	Ventura	LA-2-LA Jct. Rte. 158 (S.D. Fwy.) to Laurel [illegible]	2.73	79,300	6	12'	22'	8'	8'	222	237	0.94	1.27	74
	17	Bayshore	SM-68-[illegible] Santa Clara Co. Line to Bransten Rd.	7.90	58,800	6	12'	60'	2'	8'	415	509	0.82	1.20	68
	4	Petaluma	Son-1-P, C 2.23 mi. No. of Mrn. Co. Line to S.C.L. of Santa Rosa	20.10	12,900	4	12'	46'	5'	8'	171	284	0.60	0.90	67
	26A	Long Beach	LA-167-Bell, B, Ver Firestone Blvd. to No. Jct. Atlantic Blvd.	3.22	69,800	6	12'	40'	2'	8'	163	246	0.66	1.24	53
	20	San Diego	LA-158-C, LA, A Approx. 0.2 mi. No. of Venice Blvd. to Guada Place	3.00	38,500	8	12'	22'	6'	10'	200	320	0.63	1.20	53
	12	Long Beach	LA-167-LBch, A, Com, Lyn, SGP Pacific Coast Hwy. to Atlantic Blvd.	4.83	54,300	6	12'	16' Curb to 40'	2'	8'	250	405	0.62	1.16	53
	22	Golden State	LA-161-[illegible] Glendale Blvd. to Burbank Blvd.	5.98	51,000	6 & 8	12'	22'	6'	10'	200	331	0.60	1.17	51
		Total or Average		75.16							3292	4106	0.80		
		Grand Total or Average		200.45							11,384	9158	1.24		

Group	Section	On Ramps No. of Ramps	On Ramps Acc.	On Ramps MV	On Ramps Acc./MV	Off Ramps No. of Ramps	Off Ramps Acc.	Off Ramps MV	Off Ramps Acc./MV	Total Ramps	Total Ramps Acc.	Total Ramps MV	Total Ramps Acc./MV	Within Interchange Length	% of Total	Total Acc.	Ramp Acc.	Main Line Acc.	MVM	M. Line Acc./MVM	Between Interchange Acc.	MVM	Acc./MVM	% Align. on Curve Under 5000' R	% Align. on Grade Over 2%	Min. Curve Radius	Max. Grade (%)
HIGH ACCIDENT SECTIONS	50	4	22	51.2	0.43	2	31	28.6	1.08	6	53	79.8	0.66	← Not Readily Available →										30		600	
	24	13	105	186.5	0.56	11	203	185.6	1.11	24	308	372.1	0.83											34	26	2000	5
	28A	16	72	69.7	1.03	15	63	79.5	0.79	31	135	149.2	0.90	1.99	41.9	432	135	297	114	2.61	62	101	0.61	41	71	800	6
	7	8	10	11.2	0.89	8	11	5.3	2.08	16	21	16.5	1.27	1.82	49.6	67	21	46	27	1.70	23	28	0.82	43	—	—	—
	9	9	38	63.3	0.60	8	148	63.5	2.33	17	186	126.8	1.47	—	—	—	—	—	—	—	—	—	—	33	—	—	—
	14	22	28	31.5	0.89	21	32	27.4	1.17	43	60	58.9	1.02	4.06	44.3	186	60	126	115	1.10	214	149	1.44	14	—	—	—
	31B	8	5	13.9	0.36	8	37	14.7	2.52	16	42	28.6	1.47	—	—	—	—	—	—	—	—	—	—	42	22	2500	4
	27	21	46	30.2	1.52	22	44	31.4	1.40	43	90	61.6	1.46	—	—	—	—	—	—	—	—	—	—	13	9	3500	4.5
	23	22	81	42.4	1.91	23	49	49.4	0.99	45	130	91.8	1.42	4.56	41.3	550	130	420	260	1.65	364	348	1.05	27	35	2000	5.6
	5	4	2	10.3	0.19	5	14	13.7	1.02	9	16	24.0	0.67	0.77	37.6	39	16	23	26	0.88	56	43	1.30	36	—	—	—
	11	1	0	0.7	0	2	0	2.0	0	3	0	2.7	0	—	—	—	—	—	—	—	—	—	—	38	—	—	—
	13	3	2	3.8	0.53	4	8	11.6	0.69	7	10	15.4	0.65	—	—	—	—	—	—	—	—	—	—		—	—	—
	Total or Average	131	411	516.7	0.80	129	640	510.7	1.25	260	1051	1027.4	1.02	13.22	42.9	1282	362	920	542	1.70	718	669	1.07	25	28		
AVERAGE ACCIDENT SECTIONS	26B	3	14	35.0	0.40	3	41	28.9	1.42	6	55	63.9	0.86	0.99	64.3	96	55	41	79	0.54	103	59	1.75	33	45	1200	2.7
	16	23	30	40.6	0.74	19	39	39.5	0.99	42	69	80.1	0.86	2.97	29.1	154	69	85	68	1.25	132	169	0.78	31	16	2000	3
	29	27	45	62.2	0.72	26	79	60.8	1.30	53	124	123.0	1.01	5.40	63.4	336	124	212	241	0.88	113	131	0.86	26	39	1500	3.5
	32	11	6	8.4	0.71	12	10	10.4	0.96	23	16	18.8	0.85	—	—	—	—	—	—	—	—	—	—	7	2	4000	3
	31A	17	22	24.4	0.90	15	27	23.1	1.17	32	49	47.5	1.03	—	—	—	—	—	—	—	—	—	—	10	22	4000	3.5
	15	23	24	43.0	0.56	22	27	37.0	0.73	45	51	80.0	0.64	4.13	28.7	183	51	132	110	1.20	176	249	0.71	10	2	3000	3
	8	5	5	22.3	0.22	5	8	19.3	0.41	10	13	41.6	0.31	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	10	8	10	11.2	0.89	9	9	20.0	0.45	17	19	31.2	0.61	—	—	—	—	—	—	—	—	—	—	47	—	—	—
	28B	5	15	20.0	0.75	6	30	21.7	1.38	11	45	41.7	1.08	1.47	39.2	71	45	26	24	1.08	35	92	0.38	—	—	—	—
	3	8	8	12.0	0.67	8	9	11.6	0.78	16	17	23.6	0.72	1.00	12.9	34	17	17	18	0.94	100	137	0.73	10	—	—	—
	2	2	9	12.2	0.74	2	8	11.6	0.69	4	17	23.8	0.71	0.45	14.1	78	17	61	38	1.61	216	232	0.93	36	—	—	—
	Total or Average	132	188	292.3	0.64	127	287	283.9	1.01	259	475	576.2	0.83	16.41	31.0	952	378	574	576	1.00	875	1069	0.82	19	15		
LOW ACCIDENT SECTIONS	33	4	0	3.7	0.0	4	3	4.1	0.73	8	3	7.8	0.38	—	—	—	—	—	—	—	—	—	—	20	100	2000	4
	1	29	66	114.5	0.58	32	94	116.8	0.82	61	160	229.3	0.70	7.00	44.3	658	160	498	558	0.89	613	783	0.78	15	Basically Flat		
	25	15	35	61.0	0.57	15	43	58.6	0.73	30	78	119.6	0.65	2.80	56.3	236	78	158	219	0.72	134	174	0.77	33	12	2500	3
	21	6	14	25.3	0.55	6	34	24.4	1.39	12	48	49.7	0.97	1.41	51.6	157	48	109	135	0.81	65	102	0.64	42	8	1500	3
	17	13	27	47.5	0.57	14	34	46.4	0.73	27	61	93.9	0.65	2.14	27.1	162	61	101	146	0.69	253	363	0.70	4	4	3500	3
	4	23	8	13.4	0.60	22	20	13.8	1.45	45	28	27.2	1.03	4.43	22.0	76	28	48	65	0.74	95	220	0.43	15	—	—	—
	26A	6	15	27.3	0.55	7	17	31.0	0.55	13	32	58.3	0.55	1.40	43.5	104	32	72	111	0.65	59	135	0.44	8	—	2000	—
	20	12	19	60.7	0.31	13	26	60.3	0.43	25	45	121.0	0.37	2.21	44.2	103	45	58	153	0.38	97	167	0.58	27	24	2400	4
	12	16	22	57.1	0.38	18	37	57.9	0.64	34	59	115.0	0.51	2.30	33.7	138	59	79	138	0.57	112	267	0.42	36	5	—	—
	22	16	29	56.2	0.50	17	38	55.6	0.68	33	67	112.8	0.59	2.66	44.5	142	67	75	157	0.48	58	174	0.33	37	11	1800	3
	Total or Average	140	235	468.7	0.50	148	346	466.9	0.74	288	581	935.6	0.62	26.35	36.0	1776	578	1198	1680	0.71	1496	2386	0.62	20	11		
	Grand Total or Average	403	834	1277.7	0.65	404	1273	1261.5	1.01	807	2107	2539.2	0.83	55.98	36.0	4010	1318	2692	2796	0.96	3080	4124	0.75				

concept of the interwoven factors which apparently have an effect on accident rates. The second part is a discussion of individual sections, bringing forth facts concerning each section and making an attempt to isolate those factors which give it the accident experience it possesses.

In the development of the first part, many extremely interesting facets of the relationship between design factors and accident rates were glimpsed but not fully explored. Among these were ramp terminal shapes, bridge widths, type of interchange, and interchange spacing. But perhaps most important was the tentative finding that accident rates rise sharply with average lane volume, so that additional lanes may be justified on the basis of accident reduction even as much as on the basis of free-flow service volumes.

The present report does not provide a full understanding of how these additional facets may have affected accident rates. Research is being continued in an effort to develop facts concerning the unknowns brought out in this report. This research is a portion of the Highway Transportation Agency's 5-yr Safety Research Program, in cooperation with the U.S. Bureau of Public Roads.

SUMMARY OF DATA

Table 2 is a summary of accident, exposure, and design data for each of the study sections. The freeway sections are arranged into three groups and the data are summarized or averaged for each group. Twelve of the sections totaling 55.90 mi had significantly above average accident rates (referred to as high accident sections). Ten of the sections totaling 75.16 mi had significantly below average accident rates (referred to as low accident sections). Eleven of the sections totaling 69.39 mi had average accident rates.

Three of the sections in Table 2 have been split or designated as part a and part b because when these sections were under preliminary study, it was found that they had portions with accident concentration quite dissimilar to the remainder of the section.

The data listed in the table represent a 3-yr aggregate with the exception of sections [illegible]

miles of travel (MVM). The exposure for ramps is essentially related to the number of vehicles entering or leaving without regard to the distance traveled, and therefore, is simply million vehicles (MV).

The term "within interchange area" is used in Table 2 and throughout this report. The area referred to is that encompassing the main line, ramps and structures from the end of the farthest speed change lane taper on one end of an interchange to the end of the farthest speed change lane taper on the other end of the interchange. An acceleration lane length is measured from the gore nose to the end of the lane taper. A deceleration lane length is measured from the start of the lane taper to the beginning of the first ramp curve.

INVESTIGATION OF ALIGNMENT AND GRADE

A representative sample of eight sections (1, 15, 16, 17, 20, 23, 28, and 29) from the original 31 was taken for this investigation. The eight sections total 81 mi with 3,935 MVM of exposure. The freeways ranged in ADT between 20,000 and 100,000 and had between four and eight traffic lanes. Grades ranged from 2 to 6 percent and curves from 1,500 to 5,000 ft in radius. (A curve with a radius of 5,000 ft or greater was considered a tangent section for this study. A grade of 2 percent or less was considered level.)

The accident data for sections 1, 15, 16, and 17 were taken from accident profiles where accidents are plotted to the nearest 100 ft but there is no designation as to direction of travel. The accident data for sections 20, 23, 28, and 29 were taken from large-scale collision diagrams where accidents are plotted as accurately as possible on a scale likeness of the freeway.

The alignment was categorized by type into the following groups: straight level, straight grade, curved level, and curved grade. A further breakdown by direction upgrade or downgrade was possible for sections 20, 23, 28, and 29 because the information was plotted on collision diagrams. Table 3 gives accident data for the eight sec-

TABLE 3

EFFECT OF ALIGNMENT ON FREEWAY ACCIDENT RATES

Alignment	PDO	Inj.	Fat.	Tot.	MVM	Total Acc./MVM	Inj. & Fat. Acc./MVM
(a) Sections 1, 15, 16, 17, 20, 23, 28 & 29							
Straight level (a)	1,358	915	47	2,320	2,767	0.84	0.35
Straight grade (b)	261	131	4	396	333	1.19	0.41
Curved level (c)	393	231	8	632	560	1.13	0.43
Curved grade (d)	339	165	8	512	275	1.86	0.63
Straight (ab)	1,619	1,046	51	2,716	3,100	0.88	0.35
Curved (cd)	732	396	16	1,144	835	1.37	0.49
Grade (bd)	600	296	12	908	608	1.50	0.51
Level (ac)	1,751	1,146	55	2,952	3,327	0.89	0.36
Total	2,351	1,442	67	3,860	3,935	0.98	0.38
(b) Sections 20, 23, 28 & 29[a]							
Straight upgrade	81	52	2	135	144	0.94	0.38
Straight downgrade	110	52	1	163	144	1.13	0.37
Curved upgrade	141	74	7	222	127	1.75	0.64
Curved downgrade	174	81	1	256	127	2.02	0.65
Total	506	259	11	776	542	1.43	0.50

[a]This further breakdown of grade classifications available for these sections only.

TABLE 4

EFFECT OF TRUCK TRAFFIC ON ACCIDENT RATES

Alignment	PDO	Inj.	Fat.	Tot.	MVM	Total Acc./MVM	Inj. & Fat. Acc./MVM
(a) Section 23—Truck Traffic 11 Percent							
Straight level (a)	223	117	8	348	310	1.12	0.40
Straight upgrade (b)	33	29	0	62	41	1.51	0.71
Straight downgrade (c)	35	17	1	53	41	1.29	0.44
Curved level (d)	144	72	3	219	120	1.83	0.63
Curved upgrade (e)	39	39	3	81	48	1.69	0.88
Curved downgrade (f)	52	37	1	90	48	1.88	0.79
Straight (abc)	291	163	9	463	392	1.18	0.44
Curved (def)	235	148	7	390	216	1.81	0.72
Grade (bcef)	159	122	5	286	178	1.61	0.71
Level (ad)	367	189	11	567	430	1.32	0.47
Total	526	311	16	853	608	1.40	0.54
(b) Section 20, 28, 29—Truck Traffic 4 to 5 Percent							
Straight level (a)	277	96	3	376	446	0.84	0.22
Straight upgrade (b)	48	23	2	73	103	0.71	0.24
Straight downgrade (c)	75	35	0	110	103	1.07	0.34
Curved level (d)	42	32	4	78	91	0.86	0.40
Curved upgrade (e)	102	35	4	141	79	1.78	0.50
Curved downgrade (f)	122	44	0	166	79	2.10	0.56
Straight (abc)	400	154	5	559	652	0.86	0.24
Curved (def)	266	111	8	385	249	1.55	0.48
Grade (bcef)	347	137	6	490	364	1.35	0.39
Level (ad)	319	128	7	454	537	0.85	0.25
Total	666	265	13	944	901	1.05	0.31

tions combined, as well as a further breakdown for sections 20, 23, 28, and 29. Table 4 indicates truck influence.

Section 23, a high accident section and also a high truck traffic section (11 percent) appears by itself in Table 4. This analysis is not meant to imply that truck traffic is the sole cause of the high accident rate. However, this is probably a major contributing factor since this section has 35 percent of its alignment on grades greater than 2 per-

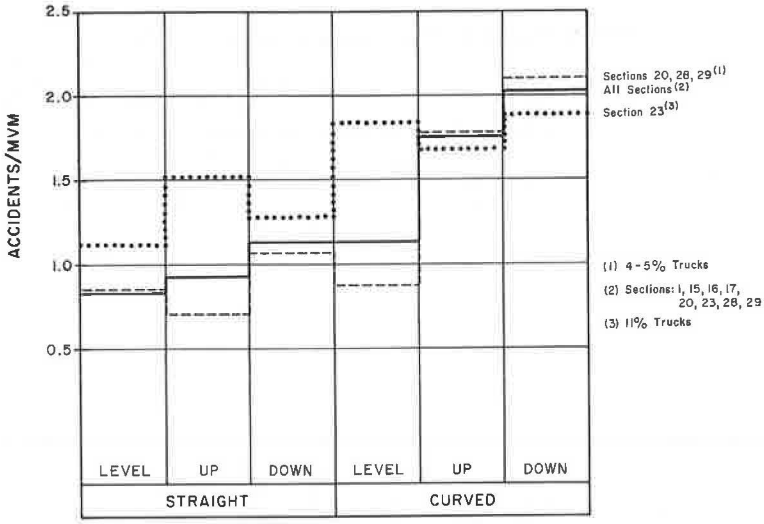


Figure 3. Accident rates by alignment type.

cent. The combination of trucks and grades will normally produce adverse conditions. This section has 41.2 percent of its alignment within interchange areas and 27 percent of its alignment on curves under 5,000-ft radius. The section is relatively old (opened 1955-1957). All of these factors contribute adversely to the accident rate.

Tables 3 and 4, and Figure 3 indicate that:

1. Straight level alignment has the lowest rates.

affect accident rates; curved grade is the greatest offender.

3. Moderate truck traffic (4 to 5 percent) appears to have little effect on accident rates, whereas heavy truck traffic (11 percent) appears to affect rates adversely on the straight upgrade and curved level alignment.

FIXED OBJECTS

Fixed objects present a major problem in both accident frequency and severity. A recent freeway fatal accident study (2) revealed that 32 percent of the 660 freeway fatal accidents in 1961 and 1962 involved fixed objects. An analysis of all types of accidents revealed that 28 percent of 36,171 freeway accidents involved fixed objects; 8.5 percent of the accidents involved fixed objects in the median and 19.5 percent involved fixed objects to the right of the traveled way. A further breakdown is given in Table 5. A field inventory of fixed objects on ten sections of freeway is given in Table 6.

TABLE 5

ACCIDENTS INVOLVING FIXED OBJECTS

Fixed Object	% of All Accidents	% of Fixed-Object Accidents
Guardrail	7	25
Fence	5	18
Signs	4	14
Pier, abutment or bridge rail	4	14
Light standard	2	7
Curbed island	2	7
Miscellaneous	4	15
Total	28	100

In an attempt to gain information as to the most vulnerable type of fixed object, the freeway accident percentages for guardrail, signs, light standards, piers, abutments, and bridge rail were adjusted in proportion so that the combined total would equal 100 percent. The resulting

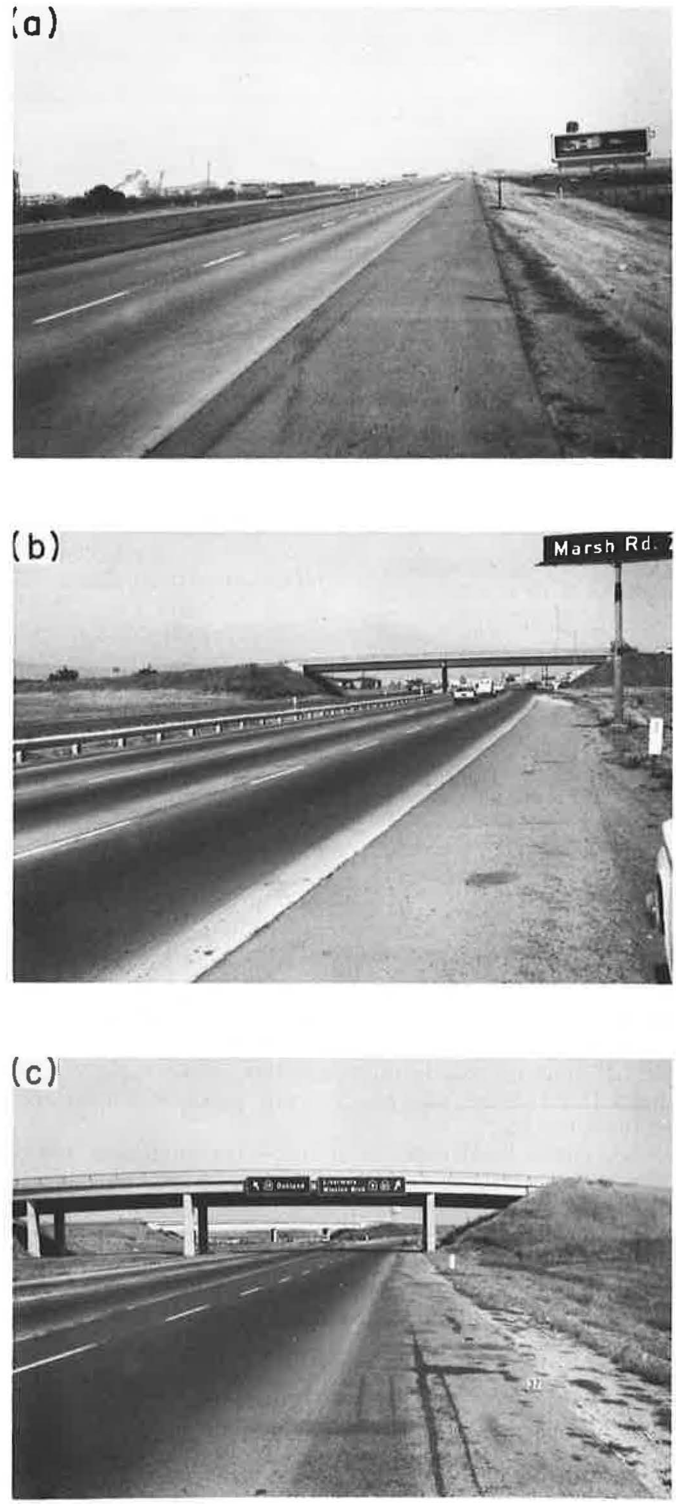


Figure 4. (a) Clear, firm roadside; (b) no pier at shoulder edge (but with sign pole); and (c) pier at shoulder edge.

TABLE 6

SUMMARY OF FIXED OBJECTS IN SECTIONS 17, 20, 21, 22, 23, 27, 28, 29, 31, 33

Fixed Objects	Areas: Interchange Off-Ramps	Interchange On-Ramps	Interchange Main Line	Between Inter-changes	Total	% of Total Fixed Objects
Guardrail[a]	602	533	206	453	1,794	46.1
Light standard	502	402	95	26	1,025	26.3
Signs[b]	237	71	112	245	665	17.1
Pillars, abutments, bridge rail	53	72	189	94	408	10.5
Total	1,394	1,078	602	818	3,892	100.0
% of total fixed objects	35.8	27.7	15.5	21.0	100.0	-

[a]Guardrail was counted in 50-ft lengths as one fixed object.
[b]Includes wood or steel posts, with or without guardrail.

TABLE 7

ACCIDENTS INVOLVING FIXED OBJECTS VS TOTAL FIXED OBJECTS IN PLACE

Fixed Object	% of Fixed-Object Accidents Involved	% of Total Fixed Objects	Ratio
Piers, abutments, bridge rail	23.5	10.5	2.24
Signs	23.5	17.1	1.37
Guardrail	41.2	46.1	0.89
[illegible]	[illegible]	[illegible]	[illegible]

percentage of accidents involved was then compared to the percentage of fixed objects actually in place. The results are given in Table 7.

The validity of this comparison may well be in question due to the lack of exposure data and the selection of a definite length of guardrail as one fixed object. At most, the comparison is an approximation which points to pillars, abutments and bridge rail as being placed in the most

[illegible]

freeway mileage in California but represent 26 percent of "unit" objects and account for 12 percent of fixed object collisions.

Figure 4 shows an example of clear, firm roadside area and two examples of fixed objects adjacent to the roadbed. The guardrail in the median (Fig. 4b) was placed to prevent U-turns across the median at the interchange, but it can easily be struck by a vehicle drifting off the traveled way. The bridge pier (Fig. 4c) could have been inclosed in the overcrossing fill cone or totally eliminated as in the case of the bridge in Figure 4b, thus eliminating a fixed object that could easily produce a fatal accident if a vehicle should drift off the traveled way.

It is difficult to determine what percent of roll-over accidents are caused by fixed objects. It has been shown mathematically and experimentally (3) that it is virtually impossible for a standard automobile which is skidding to overturn unless it strikes a fixed object (such as curbs or dikes) or its wheels are embedded in soft material. The tires simply cannot develop enough friction to produce the moment required to overturn the auto. Some fixed objects act as fulcrums when struck by the wheels of a sliding vehicle. In other instances, roll-over accidents occur when the wheels of an out-of-control vehicle imbed themselves in loose material in the median or to the right of the shoulder. Many serious accidents may well have been minor accidents or might not have been reportable accidents at all had the victims been able simply to ride them out in an upright manner ("spin").

LONG BRIDGES

There are nine exceptionally long bridges (average 1,000 ft) within the study sections. Table 8 gives the bridge descriptions along with a 3-yr accident history (1959 to 1961).

TABLE 8

LONG BRIDGES, 1959-1960-1961 DATA

Section	Description	Length Incl. Guardrail	MVM	PDO Acc.	Inj.	Fat.	Total Acc.	Acc. Involving Guardrail or Br. Rail	Acc. Involving Stalled Veh.
	(a) Without Shoulders								
1	SM-68-Bsbn, Sierra Pt. O.H. Br. No. 35-130	1,000	15.2	18	3	1	22	0	0
17	SM-68-RdwC, Redwood Cr. Br. No. 35-145	700	8.6	2	9	1	12	0	0
25	LA-167-SGt, Los Angeles Riv. Br. No. 53-828	675	11.0	8	6	2	16	9	4
23	LA-26-Pom, Rt. 26/19 Sep. Str. Br. No. 53-855	900	8.5	12	8	1	21	9	0
26	LA-167-Ver, Hobart Yard O.H. Br. No. 53-940	1,300	15.7	12	5	1	18	5	1
26	LA-167-Cmrc, East Yard O.H. Br. No. 53-842	1,600	22.4	20	9	0	29	15	2
28	SD-77-SD, San Diego Riv. Br. No. 57-126	1,000	10.6	12	6	0	18	6	2
	Total	7,175	92.0	84	46	6	136	44	9
	(b) With Shoulders								
22	LA-4-LA, Gndl, LA Riv. & Sep. Br. No. 53-1075	675	7.3	2	2	0	4	0	0
22	LA-4-Brb, Providentia Ave. Br. No. 53-1085	850	7.1	0	2	1	3	2	0
	Total	1,525	14.4	2	4	1	7	2	0

TABLE 9

Bridges	MVM	Accidents	Bridge Acc. Rate	Avg. Freeway Rate[a]	% of Avg.
Without shoulders	92.0	136	1.48	1.10	134
With shoulders	14.4	7	0.49	0.51	96

[a]Average accident rate of sections containing bridges.

Seven of the bridges had three lanes in one direction with no shoulders and 40-ft (curb-to-curb) cross-sections. The other two bridges had four lanes in one direction with shoulders and 58-ft (curb-to-curb) cross-sections. The accident data concerning the no-shoulder vs shoulder bridges are summarized in Table 9.

The bridges without shoulders show a 34 percent increase in accident rate over the rate of the freeways which they serve. Although the exposure is extremely small, the indication is that the bridges with shoulders operate at about the same accident rate as the freeways they serve. One logical explanation for this may be that a bridge without shoulders does not provide an emergency parking area. Nine of the 136 (7 percent) no-shoulder bridge accidents involved parked or stalled vehicles, whereas none of the accidents on the bridges with shoulders involved a parked or stalled vehicle. The bridge rail or guardrail was involved in 33 percent of the no-shoulder bridge accidents and 29 percent of the shouldered bridge accidents.

NUMBER OF TRAFFIC LANES

At the beginning of this study, the expected accident rate curve set the standard for determining what would constitute a good or bad section. The combined data of 4-, 6-, and 8-lane freeways were used in calculating the expected rate equation.

During the course of this study, it became evident that a 4-lane freeway usually had a higher accident rate than a 6- or 8-lane facility with the same ADT. This situation suggested an investigation to see if the accident rate vs ADT curve for the 4-lane freeways would actually show a higher average rate than the 6- or 8-lane freeways. The use of the study section data for this type of investigation was avoided in this report because many of the study sections actually pick up and drop lanes throughout their lengths and the number of lanes would not be homogeneous. To continue the investigation, freeway sections with homogeneous lane characteristics were chosen and the accident rate vs ADT curves were calculated. The investigation is not yet complete but there is a definite indication that for any given ADT a 4-lane freeway would have a higher accident rate than a 6-lane, and a 6-lane would have a higher rate than an 8-lane. When complete, the findings of the investigation will be published in a separate report.

NORMAL STATISTICS

The statistics concerning each study section are discussed in the next part of this report. These statistics are compared with an estimate of the average or normal for

TABLE 10

NORMAL ACCIDENT STATISTICS, ALL CALIFORNIA FREEWAYS, 1958-1961

Condition	% Total Acc.	Acc. Rate	Condition	% Total Acc.
No. veh. involved:			Residence of driver causing	
1	28	0.36	accident:	
2	53	0.69	California—local	82
3	15	0.19	California—not local	13
≥4	4	0.05	Out of California	3
Driver violation:			Not stated	2
Following too close	18	0.23	Age of driver causing	
Unsafe lane change	6	0.08	accident (yr):	
Improper turn	7	0.09	<15	6
Exceeding safe speed	24	0.31	15-26	25
Wrong side of road	1	0.01	27-36	28
Improper parking	1	0.01	37-46	22
Other	9	0.12	47-56	14
None	34	0.44	57-66	7
Movements:			≥67	3
Rear-end	29	0.37	Driver's condition:	
Passing side swipe	18	0.23	Had been drinking	14
Cross-median	6	0.08	Sleep	5
Stopped or backing up	14	0.18	Other defects	3
Other movements	33	0.43	No defect	78
Weather:			Vehicle condition:	
Clear	81	-	Defective	10
Cloudy	9	-	Not defective	90
Raining	6	-	Fixed object involvement:	
Fog	3	-	Hit fixed objects	
Other	1	-	(8.5% to left	
Dusk or dawn	3	-	Time of day:	
Dark—no street lights	25	-	2400-0100	
Dark—street lights	20	-	(midnight to 1 a.m.)	4
Alignment and grade:			0100-0200	4
Straight level	64	-	0200-0300	5
Straight hillcrest	2	-	0300-0400	3
Straight grade	14	-	0400-0500	2
Curved level	9	-	0500-0600	2
Curved hillcrest	2	-	0600-0700	3
Curved grade	9	-	0700-0800	6
Speed of driver causing			0800-0900	4
accident (mph):			0900-1000	3
0	9	-	1000-1100	3
1-10	5	-	1100-1200	4
11-20	6	-	1200-1300	3
21-30	8	-	1300-1400	3
31-40	13	-	1400-1500	4
41-50	23	-	1500-1600	5
51-60	24	-	1600-1700	7
61-70	9	-	1700-1800	8
≥71	3	-	1800-1900	7
Age of vehicle causing			1900-2000	5
accident (yr):			2000-2100	4
0-2	39	-	2100-2200	4
2-5	36	-	2200-2300	3
5-10	19	-	2300-2400	4
≥11	6			

freeways. The estimate is based on a computer analysis of 36,171 accidents which occurred on all California freeways during the 1958 through 1961 period. The accident rate for this period was 1.29 accidents per MVM. A summary of the analysis is given in Table 10.

In Table 10, accident rates are tabulated for the single- and multiple-vehicle category. A simple ratio was used to calculate the rates; i.e., the percentage of total accidents is to 100 as the accident rate is to the total rate of 1.29. The accident rates for the other categories were not listed due to the unknown nature of travel involvement for these categories. For example, we know that there is more travel generated during clear weather than during rainy weather but it is difficult to determine what proportion of the total travel should be assigned to each type. An attempt of this sort was made in the alignment portion of this report, but the rates determined are not tabulated as normal because the data were drawn from the study sections and not all freeways.

The drivers' violation category shows 18 percent of the accidents involved following too close violations. When accidents are involved, this type of violation represents a large percentage, but if accidents are not involved, the percentage is relatively small. It is hard to tell when a driver is following too close unless he has an accident. Some authorities believe that following too far is more hazardous than following too close, because it is more difficult for the following driver to perceive the closing rate, or differential speed of the car ahead, and also because the large differential speed in following too far accidents results in a much greater severity when an accident does occur.

Studies of traffic flow (4) show that at design hourly volume of 1,500 vph/lane, 50 percent of all drivers choose headways of 1.8 sec or less (the average headway is, of course, 3,600/1,500 = 2.4 sec), center to center. The clear headway is less than this by the amount of the car-length: speed quotient. At 2,000 vph/lane, which is often attained on urban freeways, the 50 percentile headway is 1.5 sec (average 1.8 sec), and 85 percent of all drivers choose a headway of 2.6 sec or less. Although it may seem from the foregoing that high volume automatically results in short "unsafe" headways, at an hourly volume of 1,000 vph/lane, which is considered acceptable for long-distance rural travel and is associated with free-running speeds of 60 to 65 mph, the average headway is 3.6 sec but 15 percent still choose headways of 1.0 sec or less. No wonder that a patrolman has difficulty deciding whether a driver is following too close or not, until the driver has an accident.

As stated before, these statistics are used in the discussion of the individual sections. All of the conditions have been analyzed for each section, but only those pertinent to that particular section or those that vary significantly from the normal have been included in the tables found in the discussions.

This Department is presently engaged in a study of ramp geometry as related to accident frequency. The study is in a rough draft state and the analysis is not yet completed. However, evidence analyzed to date indicates the following:

1. On-ramps have a normal accident rate of about 0.60 per million ramp vehicles; off-ramps have a normal accident rate of about 1.00 per million ramp vehicles.

2. Downhill on-ramps are the best type of on-ramp; uphill off-ramps are the best type of off-ramp.

3. Ramps associated with diamond-type interchanges are apparently the safest type. The left-hand ramps (enters or leaves the freeway at high-speed lane) experience the highest accident rates.

THE SAN FRANCISCO AREA

Section 30—SF-2-SF (New Route 80, 101)

This is the Central Skyway in San Francisco, a 1.47-mi portion of Rt. 2 between the junction of Rt. 68 and Turk St. Facts about the section are as follows:

1. 57,800 ADT, 62 MVM (1960-61), 280 accidents.
2. Accident rates—expected, 1.20; actual, 4.52 or 375 percent of expected rate.
3. The section is basically six lanes with a short portion of eight lanes. It is an

Figure 5. Central Skyway, San Francisco.

TABLE 11

ACCIDENT DATA, CENTRAL SKYWAY

Condition	% Acc.	
	Section 30	Normal[a]
Driver's condition:		
H. B. D.	3	14
Sleep	0	5
Violation—following too close	11	18
Movement—rear end	30	29
Time of day—afternoon peak 4-6 p.m.	13	15
Speed (mph):		
0	13	9
1-10	4	5
11-20	6	6
21-30	11	8
31-40	27	13
41-50	28	23
51-60	10	24
61-70	1	9
≥71	0	3

[a]Normal statistics from Table 10.

elevated viaduct, 40 ft curb to curb (no shoulders) on each roadway. About half of the section is single deck and the remainder is double deck (Fig. 5).

4. Thirty percent of the alignment is on curves between 600- and 1,000-ft radii.

5. The ramps are all two lanes and have average accident rates.

The limited design standards imposed by the elevated viaduct section create a high accident rate problem. For instance, the Embarcadero Skyway in San Francisco is similar to the Central Skyway. The Embarcadero has an accident rate of 3.05 with an ADT of 30,000. The Viaduct portion of the James Lick Freeway has better alignment (1,200-ft minimum radius curve) but still has an accident rate of 2.32 with an ADT of 130,000. The Nimitz

TABLE 12

ACCIDENT DATA, SECTION 1

Condition	% Acc.	
	Section 1	Normal
Residence—Calif., local	91	82
Weather—raining	15	6
Driver's condition—H. B. D.	17	14
Movement—rear end	57	29

(a)

(b)

(c)

Figure 6. Bayshore Freeway: (a) north end section 1, Butler Rd. O.C., rate 1.06; (b) center section 1, 19th Ave. O.C., rate 0.79; south end section 1, East Hillsdale Ave., rate 0.61.

Freeway viaduct section has mostly tangent alignment with cross-section similar to the Central Skyway. The Nimitz Viaduct has an accident rate of 1.57 with an ADT of 75,000.

Table 11 indicates that the motorists using this section are better than average from the standpoint of drinking or dozing. Violation, movement, and peak hour accidents are relatively average. Stopped vehicles show a slightly higher than normal percentage because disabled vehicles have no place to park safely. A larger percentage of accidents occurred at lower speeds than normal; however, with short radius alignment, the average speed is less than normal. When driving on an elevated freeway such as the Central, the stable objects around the driver are not as apparent as they are on a ground-level freeway. Some people believe that this sensation subconsciously increases driving speed. In other words, the sense of speed is lost much as it is when riding in an airplane. This section does not seem to bear this out; however, the speed may actually be excessive for the existing design standards. A more detailed study of viaducts involving a much larger sample may be of some value.

Apparently, the lack of shoulders and the lack of off-the-traveled-way maneuverability combined with the short radius curves play an important role in this section's high accident rate.

Section 1—SM-68-Var (New Route 101)

This is a 15.8-mi portion of the Bayshore Freeway (Rt. 68) between the north city limits of South San Francisco and Bransten Rd. in Redwood City. Facts about the section include:

1. 77,500 ADT, 1,341 MVM (1958-60), 1,271 accidents.
2. Accident rates—expected, 1.26; actual, 0.95 or 75 percent of expected rate.
3. Six lanes (a 6-mi portion was widened to eight lanes between mid-1960 and the end of 1961), 36-ft median (some median barrier installed in May 1960), and 8-ft shoulders.
4. Fifteen percent of the horizontal alignment is on curve, vertical alignment is basically flat.

Traveling south from the start of the section at the north city limits of South San Francisco to a point 10.3 mi south,

Figure 7. The Bayshore Freeway, section 17.

TABLE 13

DATA FROM SECTION 17

Condition	% Acc.	
	Section 17	Normal
Residence—Calif., local	88	82
Weather—raining	8	6
Driver's condition—H. B. D.	24	14
Movement—rear end	23	29

the freeway has an accident rate of 1.06 (expected rate, 1.27). From that point (3rd and 4th St. interchange in San Mateo), and for the next 2¼ mi, it has a rate of 0.79 (expected rate, 1.26). The remaining 3¼-mi section has a rate of 0.61 (expected rate, 1.25) (Fig. 6).

The first 10 mi of this section has a clearance to the nearest fixed object (in many places) of approximately 6 ft to the right from the edge of the shoulder; the rest has approximately 56 ft of clearance.

Figure 8. Willow Rd. interchange.

The congestion problem is evident by the large percentage of rear-end accidents (Table 12). This problem is typical of the Bayshore Freeway.

Section 17—SM-68-D, MlP, RdwC. (New Route 101)

This is a 7.9-mi portion of the Bayshore Freeway between Bransten Rd. in Redwood City and the San Mateo—Santa Clara Co. line. Facts about the section include:

1. 58,800 ADT, 509 MVM (1959-61), 415 accidents.
2. Accident rates—expected, 1.20; actual, 0.82 or 68 percent of expected rate.
3. Six-lane, 40-ft traversable median (except where median barrier is placed), 8-ft shoulders, approximately 12-ft clearance on the right where frontage roads exist and 60 ft to the right-of-way line where there are no frontage roads.
4. Alignment (Fig. 7) is relatively flat and straight with only 4 percent on curves under 5,000-ft radius (3,500-ft minimum radius) and 4 percent on grade over 2 percent (3 percent maximum).

This freeway section has a 42 percent higher accident rate southbound (0.95) than northbound (0.67). The major increase in accidents occurred between 11 p.m. and 2 a.m. During these 3 hr, there were 53 accidents southbound and only 16 accidents northbound. Table 13 shows an abnormal (24 percent) number of accidents involving drinking drivers.

The Willow Rd. interchange is a four-quadrant clover leaf without collector distributor roads (Fig. 8). The ramps have a rate of 1.54 accidents per MV. These ramps are responsible for 50 percent of the total ramp accidents in the section but carry only 20 percent of the ramp traffic. The acceleration and deceleration lanes average 550 and 350 ft, respectively. The increase in ramp accident rate is divided equally between the on- and off-ramps.

Although sections 1 and 17 are both Bayshore Freeway sections, the percentage of rear-end accidents is greater in section 1 (57 percent) than in section 17 (23 percent). Section 1 carried one-third to two-thirds more traffic than section 17 with generally the same number of lanes. The accident rates for sections 17, 1, and 2 are 0.82, 0.95, and 1.09, respectively, and the ramp rates are 0.65, 0.70 and 0.71, respectively.

In general, the alignment (especially ramps) is forced into a more restricted area as the freeway proceeds from south to north (from section 17, through 1 to 2), apparently due to increasing right-of-way cost when approaching the densely populated area. The more restricted area decreases off-the-traveled-way maneuverability. At the same time, the average daily traffic increases and peak hour traffic is heavier. Vehicles in trouble have no escape route and vehicles entering or leaving the freeway must negotiate tight ramp patterns.

THE SAN DIEGO, VENTURA, AND GOLDEN STATE FREEWAYS

Section 20—LA-158-ClC, LA, A (New Route 405)

This 5-mi section of Rt. 158 (San Diego Freeway), between Venice Blvd. U.C. in Culver City and Ovada Pl. Pedestrian U.C. near Santa Monica, displays the following characteristics:

1. 58,500 ADT, 320 MVM (1959-61), 200 accidents.
2. Accident rates—expected, 1.20; actual, 0.63 or 53 percent of expected rate.
3. Eight lanes, 22-ft median, 10-ft shoulder outside, 8-ft shoulder inside.
4. Alignment (Fig. 9) is good with 24 percent of the vertical alignment on grade over 2 percent (4 percent maximum) and 27 percent of the horizontal alignment on curve of under 5,000-ft radius (2,400-ft minimum).
5. Ramps have much lower than average rates (0.37 actual vs 0.80 expected); ramp acceleration and deceleration lane lengths both average 700 ft.

The success of this section may be attributed to better than average ramps, fairly liberal design standards, and an ADT which produces low lane densities for an eight-lane freeway. At a low lane density, emergency maneuvers can be made on the traveled way. As the lane density increases, emergency maneuvers must be made elsewhere to avoid collision with another vehicle. This section is primarily elevated and in most

Figure 9. San Diego Freeway.

TABLE 14

ACCIDENT RATES, SECTION 20

Year	ADT	Actual Acc. Rate	Expected Acc. Rate	% of Expected
1959	50,000	0.56	1.16	48
1960	59,000	0.51	1.21	42
1961	68,000	0.67	1.23	60
1962	75,000	0.78	1.25	62
1963	114,000[a]	1.13	1.35	84

[a]Freeway sections on both sides of study section opened to traffic during this period, causing sharp increase in ADT.

TABLE 15

[illegible]	Rate (per MVM)	%	Rate (per MVM)	%
Single vehicle	0.36	28	0.32	51
Multiple vehicle	0.93	72	0.31	49
Total	1.29	100	0.63	100
Rear end	0.37	29	0.08	13
All other	0.92	71	0.55	87
Total	1.29	100	0.63	100

Figure 10. Ventura Freeway.

places the 10-ft shoulder is the limit to safe off-the-traveled-way maneuvers. Table 14 gives the ADT and the accident rate for each year of the 5-yr period of 1959 to 1963.

Table 15 indicates that rear-end accidents accounted for 13 percent of the total accidents in this section, whereas the normal is 29 percent. Fifty-one percent of the accidents were single-vehicle accidents (28 percent is normal). The lack of rear-end accidents and the high percentage of single-vehicle accidents indicate that congestion is not a major problem. The fact that 51 percent of accidents were single vehicle against a normal of 28 percent does not show that this freeway is more susceptible to single-vehicle accidents, but that it is less susceptible to other types.

Section 21—LA-2-LA (New Route 101)

This 2.73-mi portion of Rt. 2 (Ventura Freeway), between the junction of the San Diego Freeway and Louise Ave., is an elevated freeway. Facts about the section include:

1. 79,300 ADT, 237 MVM (1959-61), 222 accidents.
2. Accident rates—expected, 1.27;

chain link barrier, and 8-ft shoulders.
4. Alignment (Fig. 10) is flat with only eight percent of the vertical alignment on grade over 2 percent but 42 percent of the horizontal alignment on curves between 2,000- and 2,500-ft radius.
5. Ramp rates are slightly high (0.97); acceleration and deceleration lengths average 500 and 800 ft, respectively.

During the first half of the study period, this section had an ADT of only 50,000. The freeway sections to the east and west of this section were not opened to traffic until mid-1960. As soon as the completed gaps in the freeway were opened, the ADT jumped from 50,000 to 110,000. The accident rate for this section was disproportionately low when the freeway had a volume that would produce low lane densities, but when the lane densities reached normal levels, the accident rate rose to the expected rate. A summary of accident rates for the 2.73-mi study section is given in Table 16.

TABLE 16

ACCIDENT RATES, SECTION 21

Year	ADT	Actual Acc. Rate	Expected Acc. Rate	% of Expected
1959	49,000	0.40	1.16	35
1960	79,000	0.98	1.27	77
1961	103,000	1.12	1.32	85
1962	121,000	1.10	1.36	81
1963	128,000	1.58	1.38	115

Figure 11. Golden State Freeway.

TABLE 17

ACCIDENT RATES, SECTION 22

Year	ADT	Actual Acc. Rate	Expected Acc. Rate	% of Expected
1959	41,000	0.60	1.13	53
1960	51,000	0.46	1.17	39
1961	64,000	0.60	1.23	49
1962	113,000	0.68	1.36	50
1963	114,000	0.85	1.36	63

TABLE 18

Condition	% Acc.		Acc. Rate per MVM	
	Section 22	Normal	Section 22	Normal
Violation:				
Unsafe lane change	20	6	0.12	0.08
Improper turn	25	7	0.15	0.09
Movement:				
Rear end	17	29	0.10	0.37
Passing sideswipe	32	18	0.19	0.23

Apparently, as the lane density increases, it becomes more difficult to avoid conflicting vehicle movements on the traveled way. Vehicles are thus forced to leave the traveled way in an emergency situation or conflict with other vehicles on the traveled way.

In most places, this section has only the 8-ft shoulder for emergency off-the-traveled-way maneuvers on the right. The median barrier on the left restricts the space for emergency maneuvering to 11 ft.

Section 22—LA-161/4-LA;Gndl, LA (New Route 5)

This 5.98-mi portion of Rt. 161 and 4 (Golden State Freeway), between Glendale Blvd. in Griffith Park and Cypress Ave. in the City of Burbank, displays the following characteristics:

1. 51,000 ADT, 331 MVM (1959-61), 200 accidents.
2. Accident rates—expected, 1.17; actual, 0.60 or 51 percent of expected rate.
3. Eight lanes (except for a short six-lane portion between Western and Alameda Ave.), 22-ft traversable median, 10-ft outside shoulders and 8-ft inside shoulders.
4. The ramps have better than average rates (0.59); the acceleration and deceleration lanes average 800 ft in length.

Thirty percent of this section is elevated and 70 percent is at ground level (Fig. 11). Clearance to the right within the ground level portion is fairly good (20 to 40 ft). Clearance to the right on the elevated portion is limited to 10 ft between interchanges but is approximately 30 ft within interchange areas due to the 20-ft traversable divider between the main lanes and the collector-distributor roads. The liberal design of this section is further demonstrated with shoulders on the long bridge sections, two-lane collector-distributor roads, and ramp shoulders continuing across overcrossing structures. The favorable record of this section may be attributed to liberal design standards, better than average ramp design, and good capabilities for emergency off-the-traveled-way maneuvers.

As indicated by Table 17, the accident rate in this section did not rise more than expected when the volume increased. This could be due to good off-the-traveled-way clearance allowing emergency maneuvers at higher lane densities. This section also has a low percentage of rear-end accidents (lack of congestion) but high percentages of improper turn and unsafe lane-change violations (Table 18).

LONG BEACH FREEWAY

Section 12—LA-167-LBch, A, Com, Lyn, SGt. (New Route 7)

The 6.83-mi portion of Rt. 167 (Long Beach Freeway), between the Pacific Coast Highway and Atlantic Ave. (south crossing in Compton), displays the following characteristics:

1. 54,200 ADT, 405 MVM (1958-60), 250 accidents.
2. Accident rates—expected, 1.18; actual, 0.62 or 52 percent of expected rate.
3. Six lanes, 16-ft curbed median on the south half of the section, and 40-ft traversable median on the north half of the section.
4. Thirty-six percent of the horizontal alignment is on curves under 5,000-ft radius and only 5 percent of the vertical alignment is on grade over 2 percent (Fig. 12).
5. Off-the-road clearance is good (20 to 40 ft).

The favorable record of this section is most probably a result of fairly liberal cross-section design, good alignment and good off-the-traveled-way maneuverability. Table 19 gives accident data for the section that differs measurably from the normal. This table indicates a high percentage of high-speed accidents, out-of-state residence drivers, and drivers who had been drinking.

Despite the low overall accident rate of this section, a ¾-mi portion has a comparatively poor accident record (1.80 accidents per MVM). Within the poor portion is the cloverleaf interchange at Del Amo Blvd. (Figs. 13 and 14). This interchange is on a crest vertical curve and has steep ramp grades and low visibility. This ¾-mi portion is responsible for 32 percent of all the accidents within the 6.83-mi section but has only 11 percent of the total exposure.

Section 25—LA-167-Com, A, Lyn, SGt (New Route 7)

The 4.96-mi portion of Rt. 167 (Long Beach Freeway), between Atlantic Ave. (south crossing in Compton) and Firestone Blvd., displays the following characteristics:

3. Six lanes, 40-ft traversable median, 2-ft inside shoulders, and 8-ft outside shoulders.
4. Thirty-three percent of horizontal alignment on curve with radius less than 5,000 ft (mostly 2,500-ft radius) and 12 percent of the vertical alignment on grade greater than 2 percent (3 percent maximum).
5. A large part of this section is elevated and has guardrail protection on the right. Potential off-the-traveled-way maneuverability is good in the median but is limited on the right due to the guardrail.

This section operates at 75 percent of the expected rate, probably due to its fairly liberal design standards. The only major point of accident concentration is

Figure 12. Long Beach Freeway.

TABLE 19

ACCIDENT DATA, SECTION 12

Condition	% Acc.	
	Section 12	Normal
Weather:		
Rain	2	6
Fog	8	3
Residence—Calif., local	73	82
Driver's condition—H. B. D.	25	14
Speed—61 mph	25	12
Time of day (6 to 9 a.m.)	21	13
Cross-median	12	6

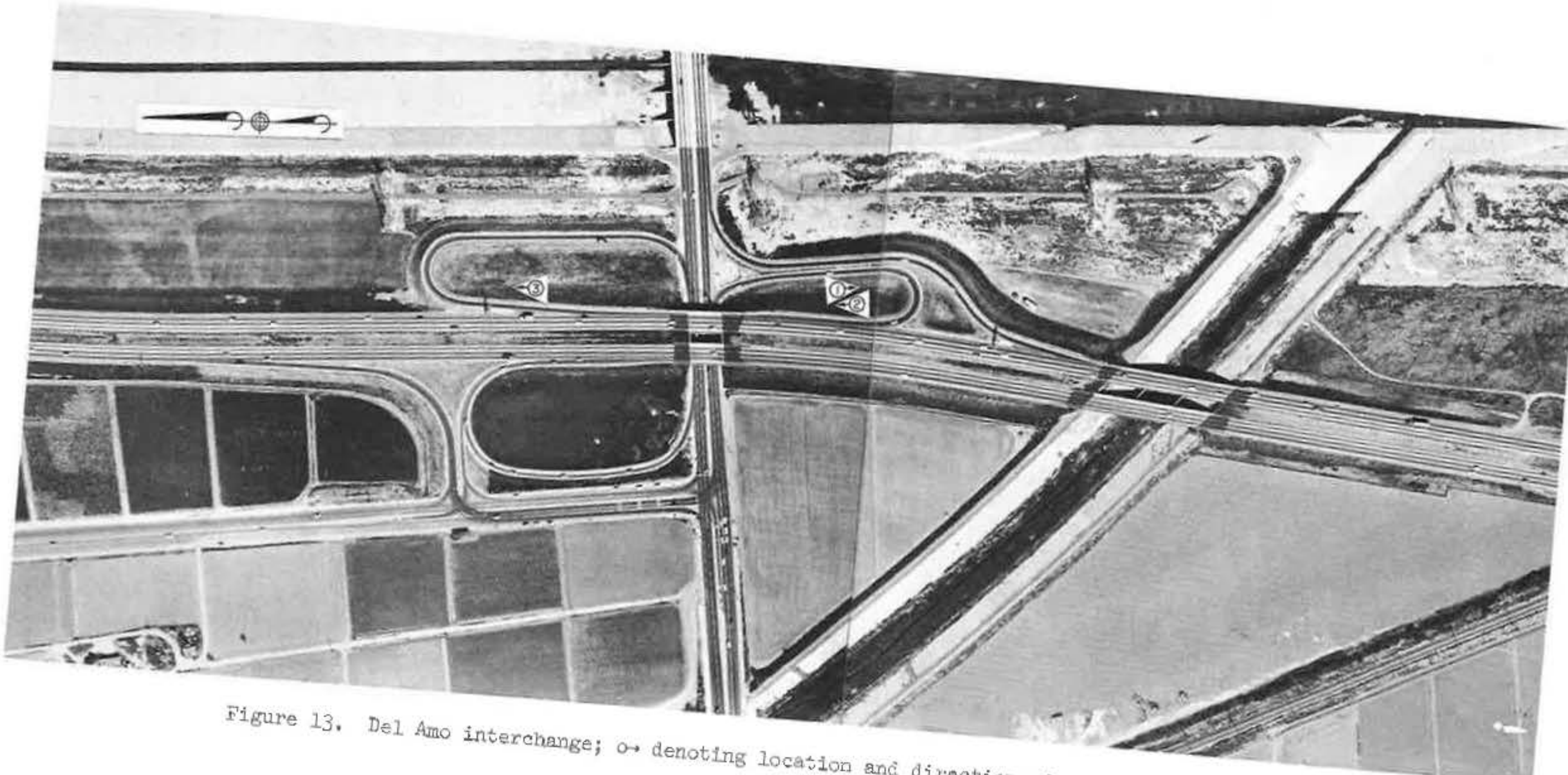


Figure 13. Del Amo interchange; o→ denoting location and direction of photos in Figure 14.

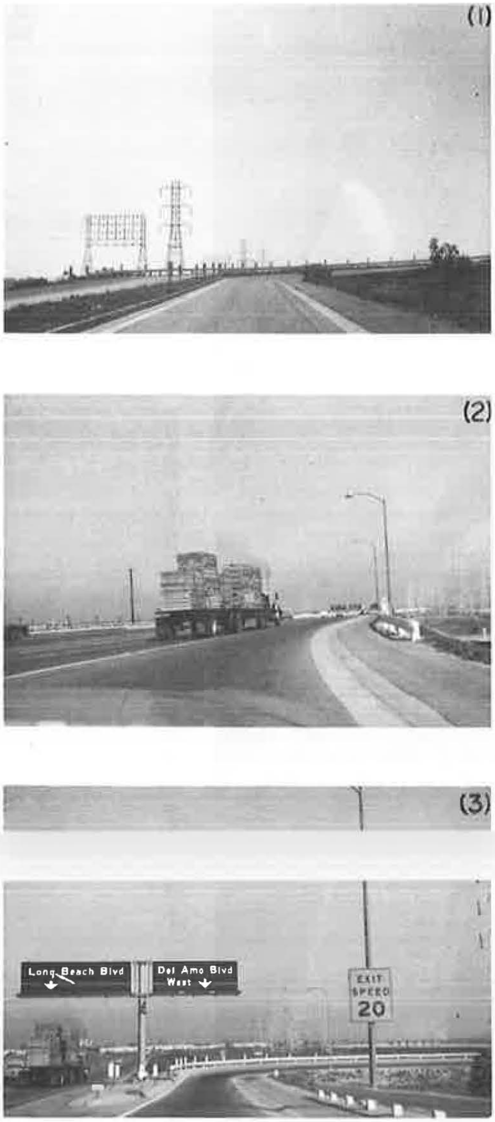


Figure 14. Del Amo interchange.

TABLE 20

ACCIDENT DATA, SECTION 25

Condition	% Acc.	
	Section 25	Normal
Weather:		
Rain	2	6
Fog	5	3
Speed—≥61 mph	23	12
Residence—Calif., local	89	82
Driver's condition—H. B. D.	25	14
Time of day (morning peak—6 to 8 a.m.)	21	13

at a 700-ft long, 40- ft wide (curb-to-curb) bridge with no shoulders. This 700-ft portion has an accident rate of 1.52. Accident data that differ measurably from the average are summarized in Table 20.

Section 26—LA-167-Bell, B, Ver. (New Route 7)

This 4.76-mi portion of Rt. 167 (Long Beach Freeway) lies between Firestone Blvd. and Olympic Blvd. in Compton. Facts about the section include:

1. 74,500 ADT, 381 MVM (1959-61), 362 accidents.
2. Accident rates—expected, 1.25; actual, 0.95 or 76 percent of expected rate.
3. Six lanes, 40-ft traversable median, 2-ft inside shoulders, and 8-ft outside shoulders.

The 3.22-mi portion from Firestone Blvd. to Atlantic Blvd. (north crossing in Vernon) is designated in Table 2 as section 26a. The next 1.54 mi from Atlantic Blvd. (north crossing in Vernon) to Olympic Blvd. is designated as sec-

[illegible] alignment in section 26a is on curve with radii less than 5,000 ft (3,000 ft minimum), whereas 33 percent of section 26b has alignment under the 5,000-ft standard (1,200 ft minimum). Section 26a is mostly at ground level with good potential off-the-traveled-way maneuverability for approximately 40 ft on the right and in the median. It has no grades over 2 percent. Section 26b has 45 percent of its vertical alignment on grades between 2 and 3 percent. The section is primarily elevated with little or no potential for off-the-traveled-way maneuvering. The section also has two 40-ft curb-to-curb bridge sections totaling 3,000

TABLE 21

RAMP RATES

Section	On-Ramps	Off-Ramps	All Ramps
26a	0.55	0.55	0.55
26b	0.40	1.42	0.86
Normal	0.60	1.00	0.80

TABLE 22

ACCIDENT RATES, LONG BEACH FREEWAY

Section	Mi	ADT	Actual Acc. Rate	Expected Acc. Rate	% of Expected
12	6.83	54,200	0.62	1.18	52
25	4.96	72,400	0.94	1.25	75
26a	3.22	69,800	0.66	1.24	53
26b	1.54	80,100	1.47	1.27	116

ft in length. The ramps in section 26b have less liberal design characteristics than those in section 26a. The section 26b ramps have only about half as much acceleration and deceleration length and sharper radius curves. The ramp rates for the two sections are given in Table 21.

A summary of accident rates for the Long Beach Freeway sections, given in Table 22, indicates a general increase in ADT from section 12 to section 26b, yet the design standards, if anything, have decreased.

HARBOR AND PASADENA FREEWAYS

Section 24—LA-165-LA (New Route 11)

The 3.2-mi portion of Rt. 165 (Harbor Freeway), between the four-level interchange and 30th St., displays the following characteristics:

1. 160,700 ADT, 563 MVM (1959-61), 1,675 accidents.
2. Accident rates—expected, 1.44; actual, 2.98 or 206 percent of expected rate.
3. Eight lanes basic with the following variations: (a) four-level interchange to 2nd St., 10 lanes (weaving section) for 0.5 mi; (b) 2nd St. to Wilshire Blvd., 0.55 mi, six lanes with two-lane collector-distributor roads in each direction; (c) Wilshire Blvd. to 11th St., 0.6 mi, four-lane southbound, three-lane northbound with the two-lane collector-distributor road continued for the northbound traffic; and (d) 11th to 30th St., 1.55 mi, eight lanes. A ½-mi portion was detoured for the first 11 mo of the study period, during the construction of the Santa Monica interchange structures.
4. Thirty-four of the horizontal alignment (Fig. 15) is on curves under 5,000-ft radius (minimum curves 2,000-ft radius) and 26 percent of the alignment is on grade greater than 2 percent (5 percent maximum).
5. Fills and concrete walls limit off-the-traveled-way maneuverability to the 8-ft shoulders.

This section has an ADT ranging from 125,000 to 180,000 veh/day. There are 24 ramps within the 3.2-mi section for an average of 7.5 ramps per mile. The ramp ADT's range from 4,000 to 25,000 veh/day and the northbound to westbound and northbound to eastbound freeway-to-freeway connections at the four-level interchange each accommodate more than 70,000 veh/day. In spite of the high volumes, the on-ramps have an average accident rate of 0.56 (normal, 0.60) and the off-ramps have an average rate of 1.11 (normal, 1.00).

Figure 15. Harbor Freeway.

The reader should, for the time being, avoid the conclusion that the interchange spacing is necessarily a causative factor in the higher than average accident rate on this section. The 24 ramps in 3.2 mi are the equivalent of six full interchanges, which would mean an average spacing of 0.53 mi. Other freeway sections with closely spaced interchanges (in this range) do not have excessive rates. The ramp accident study now under way will shed more light on this subject.

There are two major freeway-to-freeway interchanges in the section of the Harbor Freeway being considered here. There are also three two-lane collector-distributor road terminals, in addition to the freeway interchange ramp terminals, which are also two-lane ramps. The tremendous volumes using these ramps and the jockeying for position associated with these volumes are undoubtedly contributing factors in the accident rate. However, placing the ramps farther apart would not reduce these volumes but might increase them. As a matter of fact, an attempt was made to space the ramps at a greater distance by incorporating them with long collector-distributor roads, and this may be part of the problem, since it causes most of the ramp traffic to have two decisions instead of one for each entry or exit vehicle. (The accident rates shown here include accidents on the collector-distributor roads and ramps as well as those on the main line).

The staggering main line volume and the limited cross-section design with little or no off-the-traveled-way maneuverability makes this freeway section very susceptible to accidents. The accident rate would most likely be even higher if it were not for the fairly good horizontal alignment.

Table 23 compares data from Section 24 with the normal data from Table 10. The peak hours are not involved in the unusually high percentage of the total accidents within the section. One explanation for this is that the staggering main line volume causes congestion during many hours of the day.

It is interesting to note that although there are huge and notorious weaving volumes on this section of the Harbor Freeway (e.g., the weave in 1,800 ft between the Hollywood interchange and 1st St. is 3,700 vph during the morning peak hour), the number of accidents attributed to sideswiping which should be associated with weaving is no higher than average. A greater than average percentage of accidents seem to be occurring at the lower speeds. Rear-end accidents and following-too-close violations are [illegible] parently better than average within this section.

TABLE 23

ACCIDENT DATA, SECTION 24

Condition	% Total Acc. Section 24	% Total Acc. Normal
Driver's condition:		
H.B.D.	7	14
Sleep	1	5
Time of day—(afternoon peak 4-6 p.m.)	17	15
Violation—following too close	51	18
Movement—rear end	37	29
Sideswipe	20	18
Speed (mph):		
0	3	9
1-10	6	5
11-20	14	6
21-30	17	8
31-40	20.5	13
41-50	25	23
51-60	12	24
61-70	2	9
≥ 71	0.5	3

Section 9—LA-165-LA (New Route 11)

The 1.9-mi portion of Rt. 165 (Pasadena Freeway) lies between the four-level structure and the junction of Rt. 205. Facts about the section are as follows:

1. 109,000 ADT, 227 MVM (1958-60), 533 accidents.
2. Accident rates—expected, 1.35; actual, 2.35 or 174 percent of expected rate.
3. Six lanes from the 4-level interchange to the Castelar St. overcrossing (0.9 mi), primarily four 12-ft lanes and two 11-ft lanes; eight lanes through the remainder of the section (1.0 mi), primarily four 11-ft lanes and four 10-ft lanes; 6-ft curbed median with a portion of the section on separate roadways, and no shoulders with the exception of some 5-ft portions behind the rolled gutter-type curb.

This section is the northerly extension of the Harbor Freeway (section 24). Both

Figure 16. Eastbound on Pasadena Freeway, showing left-hand off-ramp to Golden State Freeway.

sections experience the problems which are associated with extremely high volumes. Section 9 is old (completed in 1948) and the lower design standards to which it was built increases the accident probability already augmented by the high volume. Thirty-three percent of the horizontal alignment is on curve with radii less than 5,000 ft. The lane widths are substandard; the median is narrow and at places replaced by a concrete wall. There are no shoulders with the exception of some 5-ft portions behind the rolled gutter-type curb. The section has four tunnels without shoulders where off-the-traveled-way maneuverability is impossible. The ramps possess short or no speed change lanes.

Also contributing to this higher accident rate is the presence of three low-standard, high-volume, left-hand off-ramps (Fig. 16). These ramps are located at Castelar St. (Hill St.), Riverside Dr. and Figueroa St. The southbound ramp at Castelar St. is a continuation of the main line tangent alignment as the freeway curves to the right. The other two ramps have sharp curves to the left with mainline downhill grade. All three of these ramps have high accident rates and account for 64 percent (118 out of 186) of all ramp accidents in the section although carrying only 23 percent of the ramp volume. The average ramp rate for the three off-ramps is 4.04; the normal rate for off-ramps is 1.00.

There is one low volume (300 veh/day) left-hand on-ramp at Amador St. that does not have a poor accident record.

TABLE 24

ACCIDENT DATA, SECTION 9

Condition	% Acc.	
	Section 9	Normal
Violation—unsafe lane change	22	6
Movement—rear end	45	29
Hit fixed object	37	28
Defective vehicle	5	10
Residence—Calif., local	90	82
Driver's condition—H.B.D.; sleep, and other	11	22

Table 24 indicates that drivers using this freeway section are more responsible than normal. Drivers involved in accidents had fewer defects, such as drinking or fatigue, and the vehicles they were driving were in better condition. The percentage of accidents involving fixed objects is high because of the lack of shoulders and the large number of fixed objects close to the traveled way, including continuous lighting. A congestion problem is suggested by the high percentage of unsafe lane change violations and rear-end accidents.

Section 10—LA-205-Pas, S. Pas. (New Route 11)

The 3.5-mi portion of Rt. 205 (Pasadena Freeway), between the junction of Rt. 165 and Ave. 64 (Marmion Way) interchange, displays the following characteristics:

1. 78,500 ADT, 301 MVM (1958-60), 337 accidents.
2. Accident rates—expected, 1.26; actual, 1.12 or 89 percent of expected rate.
3. Six 11-ft lanes, 6-ft curbed median (median barrier in place after July 1961), and no shoulders on the right with the exception of a few emergency parking areas.

This section is a continuation of the Pasadena Freeway to the east of section 9. It was completed in 1940 and is California's first freeway. The alignment is poor with

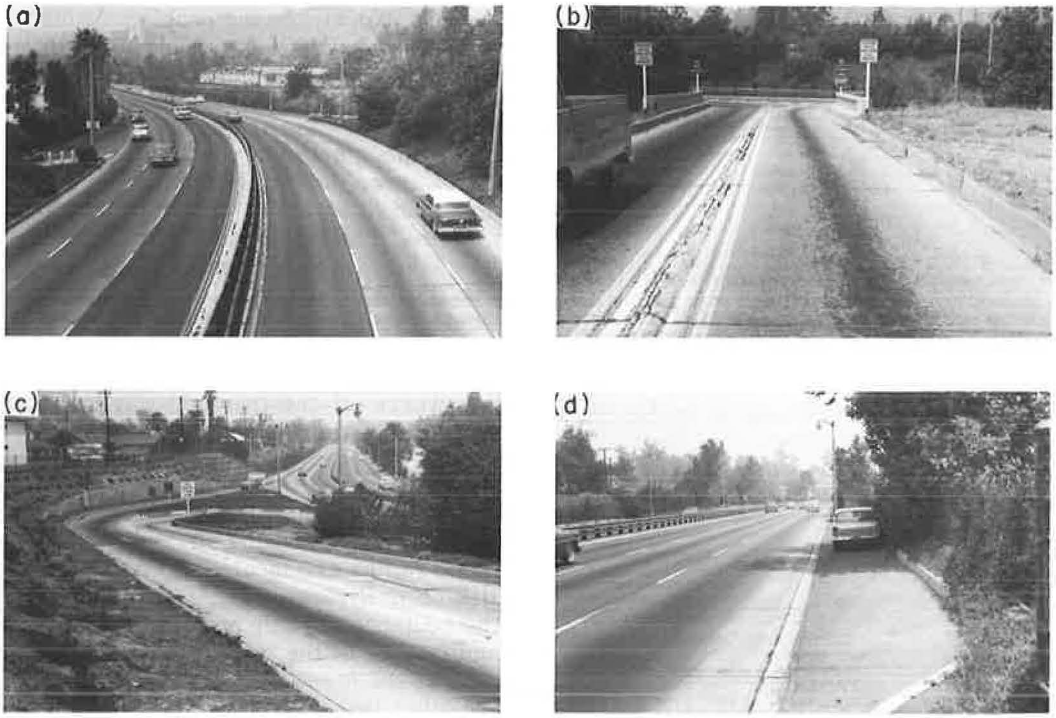


Figure 17. Section 17: (a) Pasadena Freeway; (b) northbound off- and on-ramp at Ave. 43; (c) southbound off- and on-ramp at Ave. 43; and (d) emergency parking area.

grades up to 5 percent. Forty-seven percent of the horizontal alignment is on curves with less than 5,000-ft radius (Fig. 17a). Ramp acceleration and deceleration lanes are far below present standards with some on-ramps having stop signs at the nose (Fig. 17b and c). Figure 17d shows one of the emergency parking areas which are almost the only possibility for off-the-traveled-way maneuvering. In spite of the apparent design deficiencies, this section operates with an average accident rate (Table 25).

Section 11—LA-205-Pas, S. Pas (New Route 11)

The 1.1-mi portion of Rt. 205 (Pasadena Freeway), between the Marmion Way interchange and the Orange Grove Ave. interchange, displays the following characteristics:

1. 58,100 ADT, 70 MVM (1958-60), 106 accidents.
2. Accident rates—expected, 1.20; actual, 1.51 or 126 percent of expected rate.
3. Six lanes (2- to 10-ft lanes and 4- to 11-ft lanes), 6-ft curbed median (barrier in place after July 1961), and no shoulders with the exception of the emergency parking area.

This section is the continuation of the Pasadena Freeway to the east of section 10 and the design situation is practically the same (Fig. 18), with the major difference being the reduction of two-lane widths from 11 to 10 ft. Section 11 has 38 percent of its alignment on curves with radii less than 5,000 ft, whereas section 10 has 47 percent of its alignment on curves with radii less than 5,000 ft.

TABLE 25

ACCIDENT DATA, SECTION 10

Study Period	Actual Acc. Rate	Expected Acc. Rate	% of Expected
1958, 1959, 1960	1.12	1.26	89
1961	1.41	1.25	113
1962	1.49	1.25	119

Section 11 operates at only 126 percent of the expected rate in spite of the apparent

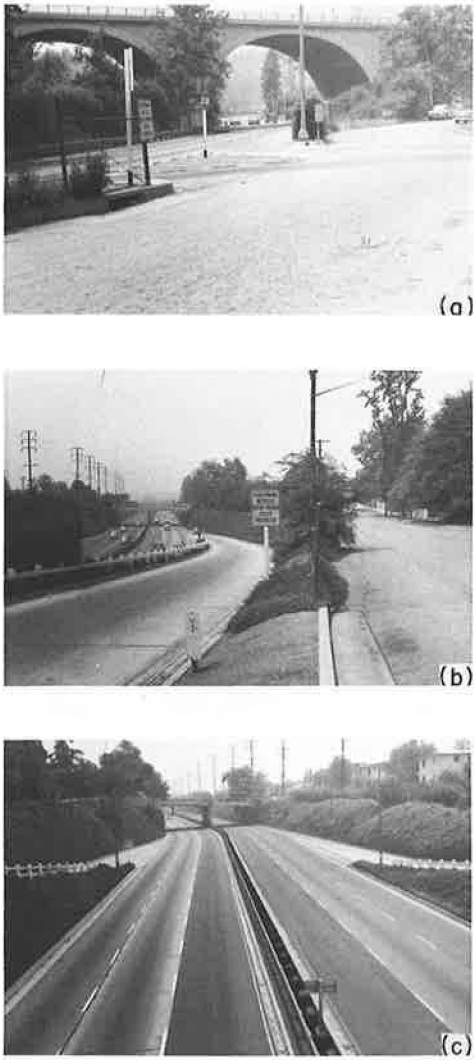


Figure 18. Section 11: (a) at Salonica Ave.; (b) southbound on-ramp at Orange Grove Ave., with stop sign; and (c) northbound on-ramp, with stop sign, and southbound off-ramp at Orange Grove Ave.

TABLE 26

ACCIDENT DATA, SECTIONS 10 AND 11

Condition	% Acc.	
	Sections	Normal
Hit fixed object	55	28
Movement—rear end	27	29
Violation—unsafe lane change	15	6
Defective vehicle	4	10
Residence—Calif., local	88	82
Driver's condition—H.B.D., sleep, and other	9	22

design deficiencies. Both sections 10 and 11 are lacking in potential off-the-traveled-way maneuverability as indicated by the high percentage of accidents which involved fixed objects (Table 26).

Apparently congestion is not a major problem within these sections. As pointed out before, sections 9, 10 and 11 are continuous on the Pasadena Freeway from the four-level structure to the Orange Grove Ave. interchange. The design standards are quite similar, yet section 9 has an ADT greater than 100,000; 45 percent of the accidents were of the rear-end type and 37 percent involved fixed objects. Sections 10 and 11 have two-thirds the ADT with only 27 percent rear-end accidents but 55 percent of the accidents involving fixed objects. Apparently the vehicles in the high-volume area are hitting each other and in the low-volume area they hit fixed objects.

SAN BERNARDINO AND RIVERSIDE FREEWAYS

Section 23—LA-26-Cla, Pom, C, WCov (New Route 10)

The 11.12-mi portion of Rt. 26 (San Bernardino Freeway), between Citrus St. in the City of Covina and the Los Angeles-San Bernardino Co. line, displays the following characteristics:

1. 49,900 ADT, 608 MVM (1959-61), 922 accidents.
2. Accident rates—expected, 1.16; actual, 1.52 or 131 percent of expected rate.
3. The west half of this section has six lanes and a 16-ft curbed median with median barrier installed in December 1961. The east half was widened from four to six lanes in December 1960. It has a 16-ft traversable median with no median barrier during the study period (Fig. 19).

This section has an ADT range of from 60,000 on the west end to 45,000 on the east end. The west half of the section, which includes the Kellogg Hill portion, had an accident rate of 2.0 with an expected rate of 1.20. The high accident rate in this west portion is probably due to the greater differential in speeds be-

Figure 19. San Bernardino Freeway.

Figure 20. On-ramps, section 23, showing lack of merging distance.

tween truck and automobile traffic. Trucks can only average about 16 mph on the 4 to 6 percent upgrade portion (3.5 mi long), whereas cars have no trouble maintaining normal highway speeds of 55 to 65 mph. The truck traffic in this section is 11 percent and the average for freeways is about 6 to 8 percent. In general, the west half of this section has lower design standards than the east half. The west half has 55 percent of its vertical alignment on 4 to 6 percent grade and 30 percent of its horizontal alignment on curve (between 2,000- and 3,500-ft radius). The east half has 14 percent of its vertical alignment on 2 to 3 percent grade and 11 percent of its horizontal alignment on curve (between 3,000- and 5,000-ft radius).

The east half of the section had an accident rate of 1.3 with an expected rate of 1.1. It had an accident rate of 1.45 during the 2-yr period before conversion to six lanes and a rate of 1.15 for the year after conversion. The construction operation (from four to six lanes) did not cause a severe accident problem. Approximately 20 accidents could be partially attributed to the construction operation.

The four diamond interchanges on the far east end of this section have extremely short acceleration lanes, averaging 260 ft from nose to end of taper (Fig. 20). These on-ramps were designed in accordance with the acceleration distance theory (5), that long straight ramp roadways provide enough distance to accelerate to 0.7 of freeway design speed and, therefore, no merging distance is necessary. The seven on-ramps accounted for 46 accidents during the study period for a ramp accident rate of 3.33 (normal, 0.60). The ramp rate for the entire section is 1.42 and if these seven ramps were excluded, the rate would be 1.03. Apparently, the ramp terminals are too short to allow proper merging and the entering vehicles must slow or stop, which in turn causes many rear-end accidents.

Section 27—SBd-26-Mcl, Upl, Ont, D (New Route 10)

The 13.58-mi portion of Rt. 26 (San Bernardino Freeway) lies between the San Bernardino-Los Angeles Co. line and Live Oak Ave. near the City of Fontana. Facts about the section include:

1. 35,100 ADT, 522 MVM (1959-61), 757 accidents.
2. Accident rates—expected, 1.10; actual, 1.45 or 132 percent of expected rate.
3. This section had four lanes for all except the last month of the 3-yr study period, at which time the easterly 6 mi of the section were opened for six lanes of traffic. The freeway was widened on the inside (median side) between January and December 1961. There was little increase in accident rates during the construction period. The section has a 16- to 46-ft median, 10-ft shoulders, 8 percent of its length on fill section, 10 percent on cut section, and 82 percent constructed at ground level.
4. This section has good alignment (Fig. 21) with only 13 percent of its horizontal alignment on curve under 5,000-ft radius (3,500-ft minimum) and only 9 percent of its vertical on grade over 2 percent (4.5 percent maximum).

This section has an ADT range of from 45,000 at the west end to 29,000 at the east end. The section also experiences an exceptionally high Sunday afternoon volume which is a result of weekend travelers returning home from trips into the Imperial Valley, the Palm Springs area, and the Salton Sea area. This volume is at its greatest peak during the months of October through April when the temperature is mild in the desert area.

Twenty-five percent of the total accidents in this section occurred westbound between 3:00 and 9:00 p.m. on Sundays in these seven months. During this period, there was only 16 MVM of travel for an accident rate of 11.9. This situation is most critical in the western 3-mi portion of this section, within which are three diamond interchanges similar to and immediately east of the ones mentioned in section 23. Within the 3-mi section and during the critical period, 80 accidents occurred with only 4 MVM of travel for an accident rate of 20.0. Here again the ramp merging areas were apparently too short to accommodate merging vehicles without causing a bottleneck in the already critical traffic flow.

Many of the accidents which occurred during this critical period were rear-end accidents at fairly high speeds (following too far). As a result, the injuries per accident (0.89) and the vehicles per accident (2.7) were higher than normal, which is about 0.76 and 1.9, respectively. Many of the accidents were a result of further congestion caused by other accidents.

The volumes during this critical period ranged between 1,000 and 1,500 vph/lane. Apparently the higher accident rates occurred when the volumes were between 1,200 and 1,500 vph/lane for a period of from 1 to 3 hr.

At lane volumes of less than 750 vph, speeds are almost independent of volume. Between 1,000 and 1,500 vph/lane, average speed drops from about 55 to 45 mph, owing to lack of passing opportunities. The westbound traffic coming from the Palm Springs area had been driving for a distance of 20 to 30 mi in a stream of high-speed characteristics, with volumes less than 1,000 vph/lane. When they catch up with the upstream end of the moving queue which forms as the volume and density build up and the speed drops, they have the sun in their eyes and are possible fatigued and irritable at having to go home after the weekend in the desert. As a result, they apparently have trouble perceiving the fact that a significant change in speed of the cars ahead has taken place. Once one accident has happened, of course, traffic stops and then the situation is compounded, with secondary collisions (counted as separate accidents) occurring at the upstream end of the stopped queue.

If the 6-hr period on Sunday had had an average accident rate, the total section would have had an accident rate of 1.05 with an expected rate of 1.10.

The most probable solution to this accident problem may be to increase the freeway capacity by adding additional lanes. The 5-mi portion of freeway between Vineyard Ave. and the county line was widened to six lanes and opened for traffic in December of 1961. This portion had an accident rate of 0.96 in 1962 with an expected rate of 1.13. The on-ramp acceleration lanes at the diamond interchanges were also lengthened during the widening project.

Section 23 (adjacent to section 27 to the west) experienced only a mild degree of the 6-hr Sunday afternoon congestion problem. Only 10 percent of the accidents in section 23 occurred during the critical period with approximately 5 percent of the vehicle miles. Apparently the six-lane portion in that section did alleviate the congestion or the upstream end of the high-density flow was generally farther east.

Section 32—SBd-26-D, Ria, Col (New Route 10)

The 8.23-mi portion of the San Bernardino Freeway, between Live Oak Ave. near the City of Fontana and 5th St. in Colton, displayed the following characteristics:

1. 33,000 ADT, 197 MVM (1960-61), 222 accidents.
2. Accident rates—expected, 1.08; actual, 1.13 or 104 percent of expected rate.
3. Four lanes, median mostly traversable (46- to 66-ft variable) with only a ½-mi section of 12-ft curbed median, good potential off-the-traveled-way maneuverability with clearance to fixed objects of 30 to 60 ft on the right in most places.
4. Alignment (Fig. 22) is good with only 7 percent of the horizontal alignment on curves under 5,000 ft (4,000-ft minimum) and 2 percent of the vertical alignment on grade greater than 2 percent (3 percent maximum). The on-ramp acceleration lanes

Considering the design aspects and the ADT of this section, it would be expected that the overall accident rate would be lower than average. In actuality, this section is subject to the same Sunday congestion problem experienced by the adjacent section 27.

Twenty-four percent of the accidents occurring in this section happened westbound between 3:00 and 8:00 p.m. on Sunday during the months of January, February, March, October and November. Seventy percent of these Sunday accidents occurred in a 2-hr period between 5:00 and 7:00 p.m. The accident rate for this 2-hr period is 23.0. If the 5-hr period on Sunday had had an average accident rate, this section would have had an accident rate of 0.90 with an expected rate of 1.08. A six-lane section may well relieve this problem as illustrated in sections 23 and 27.

Figure 22. San Bernardino Freeway.

Section 31—SBd-43-F, Col, SBd. (New Route 395 and 15)

This 8.55-mi portion of Rt. 43, which includes parts of the Riverside and San Bernardino Freeways, lies between the Los Angeles-Riverside Co. line and the junction of Rt. 31 in San Bernardino. Facts about the section include:

1. 32,000 ADT, 198 MVM (1960-61), 245 accidents.
2. Accident rates—expected, 1.07; actual, 1.19 or 111 percent of expected rate.

TABLE 27

STATISTICS, SECTION 31

Location	ADT	No. Lanes	Median (ft)	% Curve <5,000-Ft Radius	% Grade >2%	Off-Traveled-Way Maneuverability	No. Left-Hand Ramps	Actual Acc. Rate	Expected Acc. Rate	% of Expected Rate
Co. line to jct. Rt. 26	25,000	4	46	11	29	Good	0	1.07	1.02	105
Jct. Rt. 26 to Mill St.	36,500	4	42	42	22	Fair	2	1.62	1.10	147
Mill St. to jct. Rt. 31	37,000	6	22, Sep. Rdw.	10	15	Fair-Poor	5	1.07	1.10	97

(a)

(b)

Figure 23. Section 31: (a) near south city limits of Colton (accident rate, 1.07); and (b) near Colton Ave. (accident rate, 1.62).

The 2.25-mi portion from the junction of Rt. 26 to Mill St. (Table 2, section 31b) has an above average accident rate. The remainder of the section (Table 2, section 31a) is about average (Table 27, Fig. 23).

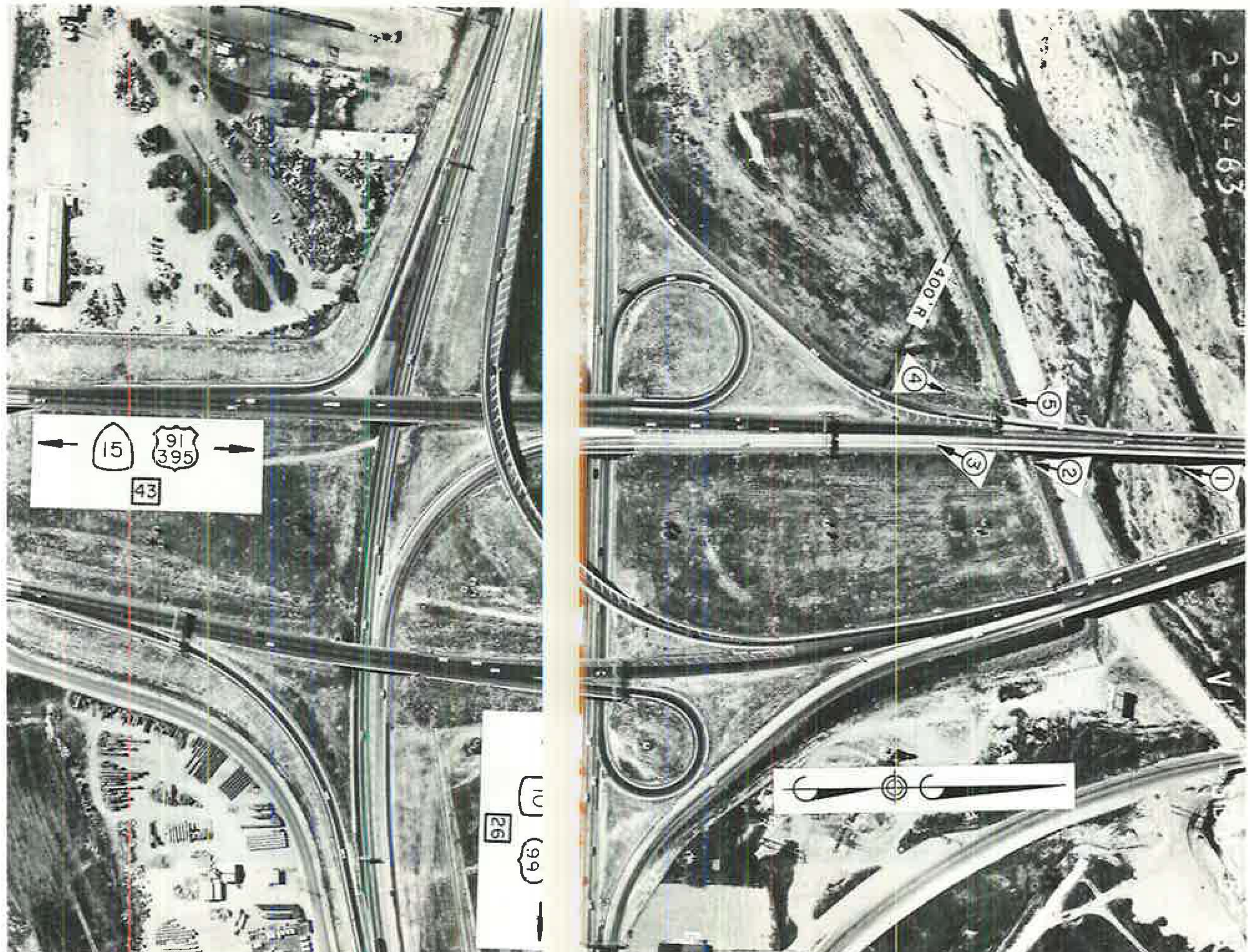


Figure 24. Junctions San Bernardino and Riverside F ways; o→ location and direction of photos in Figures 25 and 26.

Figure 25. Drivers' view of off-ramp, southbound Rt. 43 to westbound Rt. 26.

Figure 26. Construction of new ramp; southbound on US 91 to westbound on San Bernardino Freeway in Colton.

The southbound Rt. 43 to westbound Rt. 26 off-ramp is a major contributor to the high accident rate of section 31b (Fig. 24). The ramp accounted for 21 accidents with 5.3 MV entering (actual rate of 3.96 vs expected rate of 1.00). The vehicles apparently entered the ramp at too great a speed to negotiate the 400-ft ramp curve. The ramp is signed for 40 mph, but it has the appearance of being a higher speed ramp. The main line type overhead signing for the southbound-westbound turn may also contribute to the illusion. Figure 25 illustrates the drivers' view. Construction has begun on a new structure that will bypass this ramp and provide a high-speed freeway-to-freeway connection (Fig. 26).

FREEWAYS IN THE SAN DIEGO AREA

Section 28—SD-77-SD (New Route 395)

This section is on the Cabrillo Freeway and is the 8.5-mi portion of Rt. 77 north of Rt. 200 (A St.) to Clairmont Mesa Blvd. Facts about the section include:

1. 35,600 ADT, 331 MVM (1959-61), 600 accidents.
2. Accident rates—expected, 1.10; actual, 1.82 or 165 percent of expected rate.
3. Four lanes (with the exception of a ½-mi portion of eight lanes), 12- and 16-ft curbed median variable to 54-ft traversable, 2-ft shoulders on the left and 8-ft shoulders on the right.

TABLE 28

Section	Length (ft)	ADT	Actual Acc. Rate	Expected Acc. Rate	% of Expected
28a	4.75	41,300	2.30	1.13	204
28b	3.75	26,200	0.91	1.05	87

Figure 27. Section 28a.

This section has been split and designated as section 28a (A St. to Genesee Ave.) and section 28b (Genesee Ave. to Clairmont Mesa Blvd.) in Table 2. Section 28a was built in 1948 to considerably lower design standards than Section 28b which was built in 1956 and 1958. The accident problem is concentrated in the southerly 4.75-mi section as illustrated in Table 28.

Traveling from south to north, the first mile of section 28a has an accident rate of 1.25 and an expected rate of 1.00. It has a 54-ft traversable median, grades under 2 percent and over 1,400-ft radius curves. Off-the-traveled-way maneuverability to the right is poor and median traversability is hampered by trees (Fig. 27).

Figure 28. Washington St. interchange; o→ location and direction of photos in Figure 29.

Figure 29. Washington St. interchange.

Figure 30. Section 28b; actual accident rate, 0.91; expected accident rate, 1.05.

TABLE 29

ACCIDENT DATA, SECTION 28

Condition	% Acc.	
	Section	Normal
Driver's condition:		
H.B.D.	15	14
Sleep	3	5
Violation—following too close	33	18
Movement—rear end	35	29
Speed—31-40 mph	16	13
Time of day—afternoon peak 4 to 6 p.m.	32	15

The next 2.5-mi portion of section 28a has an accident rate of 3.0 with an expected rate of 1.10. It has a 6- to 16-ft curbed median, 76 percent of its vertical alignment on grade of from 4 to 6 percent and 54 percent of its horizontal alignment on curves between 600- and 4,000-ft radius. Within this 2.5-mi portion is the Washington St. interchange (Figs. 28 and 29). The ½-mi portion which includes this interchange has an accident rate of 5.6. The interchange area is situated on a 600-ft radius horizontal curve with a +4 percent grade entering and a -6 percent grade leaving. There is a 12-ft curbed median, little or no off-the-traveled-way maneuverability to the left or right; the situation is further complicated by six high-volume (4,000 ADT average) ramps.

The next 1.25 mi of section 28a has an accident rate of 1.60 with an expected rate of 1.10. It has a 5 percent maximum grade, 3,000-ft minimum radius horizontal curve and a 1,000-ft bridge with no shoulders at the San Diego River. The bridge has an accident rate of 1.7.

The on-ramps within section 28a have an average acceleration lane length of 300 ft and an average ramp accident rate of 1.03 accidents per million ramp vehicles as compared to a normal of about 0.80.

The next 3.75 mi (section 28b) have an accident rate of 0.91 with an expected rate of 1.05. The horizontal alignment is primarily tangent and the grades are less than 2 percent. Off-the-traveled-way maneuverability is good in most places (Fig. 30). The two interchanges within this section have the main lane shoulders carried through the interchange area. The ramp acceleration lanes have an average length of 720 ft. The combined ramp rate is 1.08 with the major offenders being the off-ramps (average rate, 1.38).

TABLE 30

ACCIDENT DATA, SECTION 13

Condition	% Acc.	
	Section 13	Normal
Single vehicle	52	28
Vehicle age (yr):		
0-2	24	39
5-10	39	19
Driver's age—15-26 yr	46	25
Driver's condition		
H. B. D.	14	14
Sleep and other	30	8
Violation		
Following too close	24	18
Unsafe lane change	14	6
Movement		
Rear end	44	29
Passing sideswipe	2	18
Hit fixed object	44	28
Light conditions—daylight	37	52

Figure 31. Montgomery Freeway.

TABLE 31

ACCIDENT DATA, SECTION 14

Condition	% Acc.	
	Section	Normal
Single-vehicle accidents	39	28
Speed—41-50 mph	40	23
Light condition—daylight	35	52
Time of day—12:00 to 7:00 a.m.	48	23
Residence—Calif., local	71	82
Driver's age—15-26 yr	43	25
Driver's condition:		
H. B. D.	43	14
Sleep and other	9	8
Automobile age (yr):		
0-2	40	39
5-10	32	19
Violation:		
Following too close	12	18
Unsafe lane change	6	6
Movement:		
Rear end	42	29
Passing sideswipe	9	18
Hit fixed object	36	28

Section 28, as a whole, has a peak hour congestion problem; Table 29 indicates a high percentage of peak hour accidents followed by the high percent of following-too-close violations and rear-end type accidents.

The rear-end type accident is more frequent in section 28a (41,300 ADT) than in section 28b (28,200 ADT). Figure 30 is typical of the northern portion of this study section.

Section 13—SD-2-SD (New Route 5)

This section is on the Balboa bypass, from 1 mi south of Balboa Ave. to 0.65 mi north of Balboa Ave. on Rt. 2 in San Diego. Facts about the section include:

1. 26,600 ADT, 48 MVM (1958-60), 62 accidents.

2. Accident rates—expected, 1.03; actual, 1.29 or 125 percent of expected rate.

3. Four lanes, variable median width, 2-ft inside shoulders and 8-ft outside shoulders.

This section of freeway operates at 125 percent of the expected rate, has fair to good off-the-traveled-way maneuverability, and is located in the heart of the Balboa-La of young drivers, including a large percentage of servicemen, frequent this area. As Table 30 indicates, the accidents involved a large percentage of young, inexperienced, irresponsible, fatigued drivers in old cars speeding in early morning hours.

Section 14—SD-2-SD, ChV, G, NatC. (New Route 5)

The 9.16-mi portion of Rt. 2 (Montgomery Freeway) between the San Ysidro junction and the south city limits of National City. Facts about the section include:

1. 26,300 ADT, 264 MVM (1958-60), 400 accidents.

2. Accident rates—expected, 1.03; actual, 1.52 or 148 percent of expected rate.

3. Four lanes, 36- to 40-ft deterring earth median, 14 percent of horizontal alignment on curve under 5,000-ft radius (Fig. 31).

This section of US 101 is the major route between San Diego and Tijuana,

Figure 32. Otay River bridge, showing northbound lanes looking south; guardrail on other end of bridge.

Figure 33. Off-ramp to National Ave.

Mexico. Tijuana is a typical border city with scores of bars and night clubs operating on a 7-day-a-week, 24-hr-a-day basis and providing popular entertainment for thousands of sailors and marines in the San Diego and Oceanside areas.

As Table 31 indicates, there was a high percentage of accidents involving young drivers and drivers who had been drinking. There was a high percentage of early morning, single-vehicle accidents involving older automobiles. A high percentage of rear-end type accidents is usually indicative of a congestion problem; however, this section has a relatively low ADT and peak hour accidents are not excessive.

There are four relatively narrow bridges within this section. Two are located at the Sweetwater River Channel and two are at the Otay River. They are two lane, 26 ft in width (curb to curb) and have flared guardrail approaches (Fig. 32).

Thirty-one percent of the total accidents occurring in this section were concentrated within the 5,400-ft length that includes the bridges. This portion carried only 16 percent of the MVM for an accident rate of 2.94. If the 5,400-ft portion were excluded, the remaining 8.14 mi would have an accident rate of 1.24 with an expected rate of 1.03.

There are two ramps within this section that seem to have some degree of accident concentration. The first is a two-lane, left-hand on-ramp from eastbound Rt. 199 to northbound Rt. 2 at Palm City. This ramp has an accident rate of 1.16 which is about twice normal for an on-ramp. The second ramp is the northbound off-ramp to National Ave. It is a buttonhook-type ramp with a 250-ft deceleration lane which ends with a 70-ft radius curve that has a 37° delta (Fig. 33).

In July 1958, a shoulder stripe was installed in advance of the off-ramp. In August 1958, the ramp was fog sealed for color contrast with the main line, and the plywood fence with black and white zebra stripes was installed in the head-on position down the deceleration lane. In 1959, there were 9 ramp accidents with only 0.29 MV entering. In January 1960, the shoulder stripe was lengthened in advance of the deceleration lane and under the structure. There were 4 accidents in 1960, 4 in 1961, and 3 in 1962 with approximately 0.32 MV entering per year.

MISCELLANEOUS LOCATIONS

Section 4—Son-1-F, C (New Route 101)

This 20.1-mi portion of Rt. 1 between Petaluma and Santa Rosa (Petaluma-Santa Rosa Freeway) displays the following characteristics:

1. 12,900 ADT, 284 MVM (1958-60), 171 accidents.
2. Accident rates—expected, 0.90; actual, 0.60 or 67 percent of expected rate.
3. Four lanes, 46-ft traversable median, 8-ft outside shoulders, and 5-ft inside shoulders.
4. Off-the-traveled-way clearance is good with 40- to 60-ft clearance to the right-of-way line.
5. Only 15 percent of the total alignment within this section is on curve.

Figure 34. Petaluma—Santa Rosa Freeway.

The success of this section in accident experience can be attributed to good alignment standards, liberal cross-section design, good off-the-traveled-way emergency maneuverability and low volume even for a four-lane freeway (Fig. 34).

As Table 32 indicates, accidents in this section involved more driver defects, greater speeds and a high percentage of single-vehicle accidents. Yet in spite of the high percentage of driver defect-caused accidents, the overall rate is very low due to the high geometric standards.

TABLE 32

ACCIDENT DATA, SECTION 4

Condition	% Acc. Section	% Acc. Normal
Weather		
Rain	9	0
Fog	7	3
Speed—> 60 mph	26	12
Driver's condition		
H. B. D.	22	14
Sleep	18	5

Section 7—Tul-4-F (New Route 99)

This section is the 3.67-mi portion of the US 99 freeway from the Visalia Airport interchange to 1 mi north of Goshen. Facts about the section are as follows:

1. 13,700 ADT, 55 MVM (1958-60), 90 accidents.

2. Accident rates—expected, 0.92; actual, 1.64 or 178 percent of expected rate.

3. Four 12-ft traffic lanes, 8-ft shoulders on right, 2-ft shoulders on left and 46-ft traversable median (Fig. 35).

The Visalia Airport interchange is a major contributor to the high accident rate

0.85 mi long or 23 percent of the total section accounted for 60 percent of the accidents in the total section. Of the 54 accidents occurring in the interchange, 20 were directly attributed to the 7 ramps within the interchange. Two of these ramps (F & G) accounted for 13 of the 20 ramp accidents (Fig. 36). These accidents were all weaving accidents occurring in the weave between the two ramps (F & G) which is a two-lane weave across the main lane northbound freeway traffic. Some of the accidents occurred when vehicles in the outside main lane attempted to cross the inside lane into the left-hand off-ramp (G). Ramp (G) is a two-lane PCC left side off-ramp and it is confusing to the main line traffic as to which actually is the continuation of the main lanes (Fig. 37).

Figure 35. Section 7.

All traffic from Visalia to Hanford must weave with the freeway traffic. Traffic from Hanford to Visalia does not have to weave across freeway traffic; however, there is an element of surprise at the off-ramp (B) (Fig. 38). The ramp is hidden from view by the underpass until the motorist is almost too close to the ramp nose to make the turn decision. Southbound freeway traffic going to Visalia expreiences the same sensation. The left-hand off-ramp from Visalia to southbound on the freeway (Ramp A) is also confusing because of the limited visibility (Fig. 39).

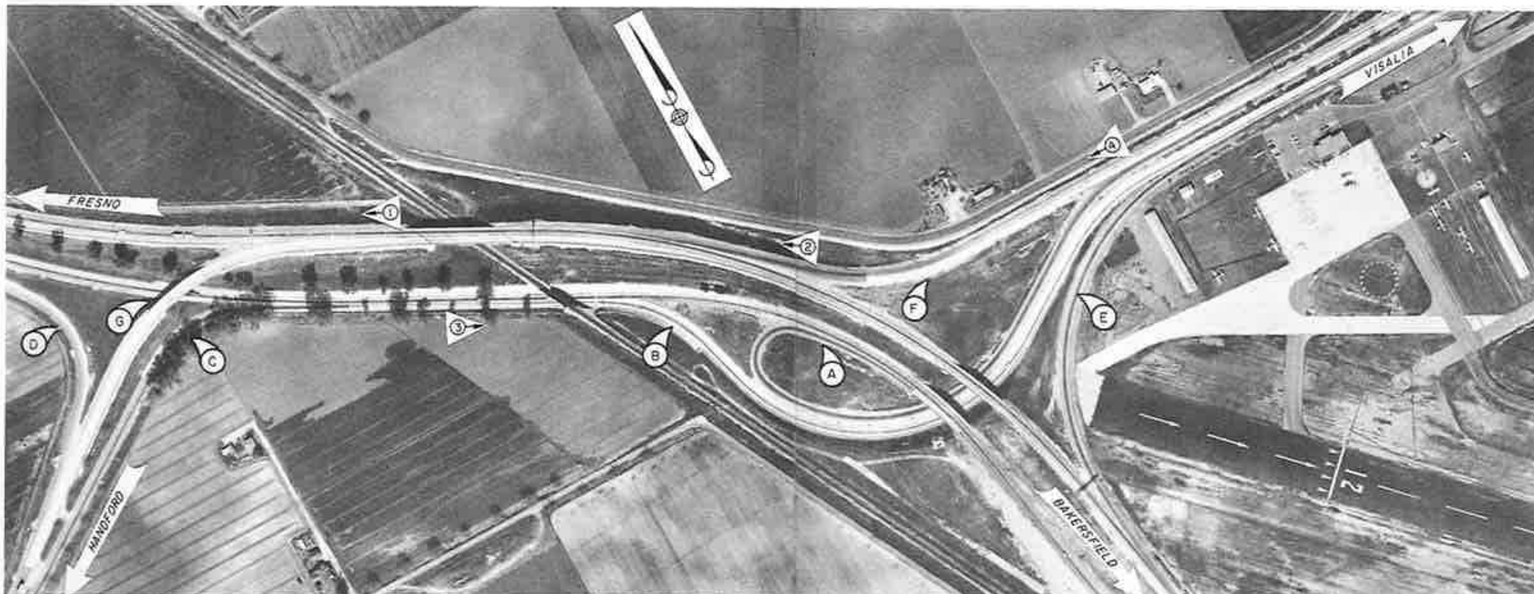


Figure 36. Visalia Airport interchange; o→ location and direction of photos in Figures 37 and 38.

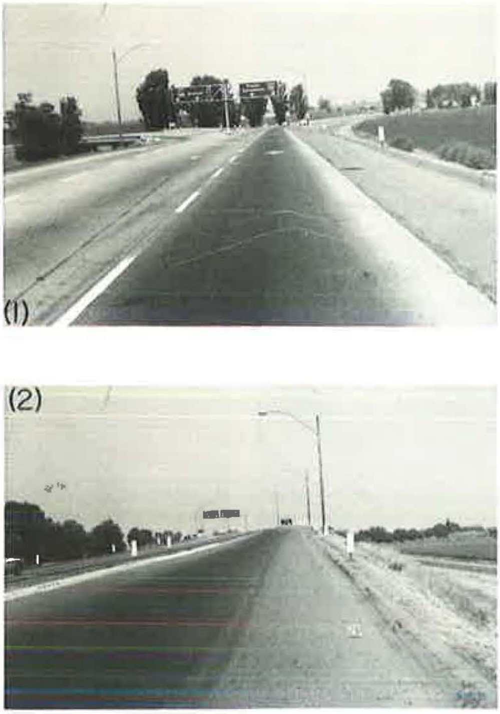


Figure 37. Ramps, Visalia Airport inter-change.

Figure 38.

TABLE 33

DATA FROM VISALIA FREEWAY SECTION

Condition	% Acc.	
	Section 7	Normal
Single vehicle	39	28
Light—daytime	44	52
Alignment—straight level	42	64
Residence—Calif., local	42	82
Driver's condition—sleep	15	5
Defective vehicle	15	10
Hit fixed object	37	28
Movement:		
Rear end	16	29
Passing sideswipe	24	18
Violation:		
Following too close	6	18
Unsafe lane change	14	6

Figure 40. Hamilton Ave. interchange, southern portion.

The 2.82-mi portion of the freeway section which excludes the airport interchange has an accident rate of only 0.80. This portion includes the two interchanges at Goshen.

The comparisons given in Table 33 point to the factors which were perhaps the major contributors to the total section's accident rate. There were fewer local drivers and more fatigued drivers involved than normal. The single-vehicle accidents generally involved the fatigued or sleepy driver hitting fixed objects during the evening hours.

Figure 41. Hamilton Ave. interchange; o→ location and direction of photo in Figure 40.

The table indicates only 42 percent for straight level alignment accidents; however, this section has 43 percent of its alignment on curve.

Section 16—SC1-5;239-LGts, D, Cmb, SJs;D, SJs (New Route 17 & 280)

The San Jose-Los Gatos Freeway, a 10.2-mi portion of Rt. 5 between San Jose and Los Gatos, displays the following characteristics:

1. 32,000 ADT, 238 MVM (1960-61), 268 accidents.
2. Accident rates—expected, 1.07; actual, 1.20 or 112 percent of expected rate.
3. Four lanes, 34- to 46-ft traversable median, and 8-ft shoulders.

This freeway section has an average accident rate. Off-the-traveled-way maneuverability is fair. Thirty-one percent of the alignment is on curves under 5,000-ft radius and 16 percent is on grade greater than 2 percent.

Figure 40 is a view of the southern portion of the Hamilton Ave. interchange, looking south. Figure 41 is an aerial view of the entire interchange. The ½-mi section that includes the interchange has an unusually high accident rate of 2.6. Immediately preceding the interchange, the northbound traffic must negotiate a 2,300-ft radius curve on a -3 percent grade. There are two 150-ft long, 30-ft curb-to-curb (each direction) bridges within the curve.

The northbound off-ramp to Hamilton Ave. has the appearance of the traveled way extended at the end of the curve. The ramp has a 260-ft radius curve with a deceleration length of 500 ft. It has an accident rate of 4.5 with 2 million vehicles entering.

The interchange is partially hidden from view by the railroad overhead when northbound and by a crest vertical curve preceding the interchange when southbound.

Other Sections

Sections 2, 3, 5, 8, 15, 29 and 33 operate relatively close to the expected accident rate and present no new concept or design feature for discussion. The sections were not abnormally hindered by the careless or drinking driver, and the designs could best

SUMMARY AND CONCLUSIONS

Thirty-three sections (200 mi) of freeways with widely divergent accident rates and a total of 11,384 accidents were analyzed. The accidents occurred during the course of 9,198 MVM of travel, to yield an average accident rate of 1.24 accidents per MVM. The accident rates for the individual sections range from 4.52 to 0.60 accidents per MVM.

The first part of the study indicated that some design features were associated with higher accident rates than others. The trends which developed are as follows:

1. The alignment was broken down into 6 types which are, in order of low to high accident type, straight level, straight upgrade, straight downgrade, curved level, curved upgrade, and curved downgrade. With heavy truck traffic, the straight upgrade is apparently more detrimental than the straight downgrade and all of the curved classifications are practically the same.

2. Fixed objects are involved in about 28 percent of all freeway accidents. Piers, abutments and bridge rails are apparently the most vulnerable, with signs, guardrails, and light standards following in that order. Curbs, dikes and drainage structures frequently act as fulcrums to convert simple skidding into roll-over accidents.

3. Long bridges with shoulders have better accident rates than long bridges without shoulders.

4. Ramps associated with diamond-type interchanges are apparently the safest type. On-ramps generally have better accident rates than off-ramps. The downhill on-ramp is the best type of on-ramp and the uphill off-ramp is the best type off-ramp. The left-hand ramp (enters or leaves the freeway at high-speed lane) has a higher accident rate than any other class.

5. For any given ADT, a four-lane freeway has a higher accident rate than a six-lane freeway and a six-lane freeway has a higher accident rate than an eight-lane.

The second part of the study indicated, in general, that the high accident sections possessed the lower alignment standards. They had an average of 25 percent of their horizontal alignment on curves with radii less than 5,000 ft, whereas the low accident sections averaged 20 percent. The vertical alignment of the high accident sections averaged 28 percent on grades greater than 2 percent. The low accident sections averaged 11 percent.

Forty-three percent of the high accident sections' total length was within interchange areas as compared to 36 percent for the low accident sections. The interchange on a horizontal or vertical curve was more apt to be found in the high accident sections.

The high accident sections had an average ramp accident rate of 1.02 accidents per MV, whereas the low accident sections had an average ramp rate of 0.62. The high ramp rates were associated with the left-hand connections, tight ramp turns and short speed change lanes.

The high accident sections possessed more narrow bridges and less clearance to fixed objects. Narrow curbed medians were characteristic of the high accident sections and the wide traversable medians were generally associated with the low accident sections.

A few of the high accident sections experience congestion of short duration (weekend traffic) that resulted in extremely high accident rates during those periods. Two of the high accident sections experienced an unusual proportion of accidents involving irresponsible or drinking drivers.

The low accident sections were characterized by open-type roadside which would allow the driver to maneuver safely off the traveled way in the case of emergency. Some of the low accident sections did not provide good off-the-traveled-way clearance, but in every case the lane density (ADT/lane) was low enough to allow sufficient maneuvering on the traveled way.

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Study Techniques for Planning Freeway Surveillance and Control

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Four study techniques, found to be quite useful in planning the peak period freeway surveillance and control activities on the Gulf Freeway in Houston, are presented: (a) entrance ramp origin-destination studies, (b) input-output studies of closed freeway subsystems, (c) aerial photography, and (d) input-output studies of critical intersections in the study area. Data from these studies can be used to plan peak period ramp controls because the demand and capacity can be estimated at each bottleneck (both on the freeway and on the frontage roads and streets) and to plan arterial street controls to provide for diverted traffic because the travel patterns of freeway interchange traffic can be determined. The duration and severity of control at each ramp which are required to prevent congestion can be estimated. The data are also useful in before-and-after comparisons.

•THE INCREASING severity of peak period freeway congestion has led to the establishment of freeway surveillance projects to study this problem. In addition, many other operational studies have been undertaken to determine the causes of peak period congestion. Because of the increasing severity of peak period freeway congestion and the increasing interest in its remedies, a need exists for a means or technique of studying a system composed of one direction of a freeway, perhaps several miles in length. Specifically, it must be possible to locate the critical bottlenecks in the system and to develop a control plan or geometric design changes to alleviate the problem of congestion. In addition, a need exists for a means of measuring completely the effect on the freeway system of any control. This evaluation should be made very quickly after the end of the peak period.

Traditionally, data collection procedures have used point studies, such as time-lapse photography, to determine operational characteristics of the traffic stream at a point. In some cases, several point studies have been used to study a length of freeway but, in these, primary interest has been devoted to the behavior at each individual point. Aerial photography can be used to study a length of freeway and is very useful to determine certain characteristics (especially density), but it is difficult to obtain flow rates and volume counts from the air photos. Few attempts have been made to study a length of freeway as a system.

When considering one direction of a congested freeway during the peak period and attempting to determine operational controls to prevent or reduce this congestion, several things are of interest. Freeway congestion sets in at a location when the demand exceeds the capacity. Hence, for a freeway which is regularly congested during the peak periods, one fact is already established. It is known that the demand exceeds the capacity somewhere on the freeway under consideration during its congested period. The problem is to determine at which locations and by how much this occurs.

Paper sponsored by Committee on Freeway Operations.

On many congested freeways the area of congestion extends for several miles. In these cases there are probably several bottlenecks and several exit and entrance ramps in the congested area. The operation at any location is a function of the operation at many other locations, so the problem is really that of studying a system of interdependent locations.

The need, therefore, exists for a technique of (a) identifying the bottlenecks, (b) estimating the capacity of each bottleneck, and (c) estimating the demand at each bottleneck so the magnitude and duration of the excess demand can be determined. This is needed to develop a rational peak period control system which can hold the demand at each bottleneck less than or equal to its capacity. However, the freeway cannot be considered independently of other traffic arteries in the same area when developing a peak period control system for the freeway traffic. The traffic operation on the frontage roads and major arterials that intersect the freeway may be affected as much or more than the operation on the freeway itself. Therefore, the same requirements that were set forth for studying a one-directional freeway system apply equally well to studying those streets that accommodate the freeway interchange traffic (i.e., traffic that enters and/or leaves the freeway within the system of interest) as well as the local or non-freeway traffic; namely, to locate critical bottlenecks and to estimate the demand and capacity of the bottlenecks. Also the development of a control system requires the reassignment in time or space of a portion of the freeway interchange traffic, and the calculation of new estimates of demand rates at all of the interchange locations on the freeway and streets. This reassignment requires some knowledge of the travel patterns on the surface street systems, the available alternate routes and the capacity restrictions on these routes.

SCOPE AND OBJECTIVES

The primary objective of this report is to present some study techniques which have proved valuable in planning a peak period freeway surveillance and control project. Some of these techniques were developed especially for use in the study of freeway operations of the inbound Gulf Freeway system as part of the Gulf Freeway Surveillance Project in Houston, but they all have general applications to the field of freeway surveillance and control. Some data and applications of the Houston data are presented to illustrate the methods, but this should not detract from the generality of the approach to the problem and the methods discussed herein.

The techniques presented, when used in combination, fulfill the following objectives:

1. To locate the critical bottlenecks on the freeway and arterial streets;
2. To determine the capacity flow rates at each bottleneck;
3. To determine the demand pattern at each freeway entrance ramp;
4. To determine the demand pattern at each critical freeway bottleneck;
5. To determine by how much and for how long the demand exceeds the capacity at each bottleneck;
6. To provide data suitable for interpretation in terms of the type of control systems required to prevent freeway congestion;
7. To provide data that are suitable for use in before-and-after studies to be used for evaluating control experiments and can be analyzed immediately after the data are collected each day;
8. To provide data suitable for predicting the effect of a control and/or geometric change;
9. To determine the travel patterns of the freeway interchange traffic on the city street system; and
10. To provide data suitable for estimating the effects of a control system on the demand pattern at each freeway entrance ramp.

ORIGIN-DESTINATION STUDIES

Study Technique

The study of travel patterns on city streets is generally developed from a flow diagram or flow map of the traffic volumes using the city street system. These flow maps are developed from a series of spot counts at major intersections and along major thoroughfares which give point data for the total number of vehicles using a particular street. These volumes are then combined to determine the demand of traffic on these various arterials. At some of the major intersections turning movement counts are made to indicate the demand for separate turning lanes, separate signal indications, or control for turning movements. These data do not give the entire picture of the travel patterns in that there is no way to separate short trips from through movements, or to distinguish the traffic that is destined for the freeway from the other traffic. To better understand the travel patterns of the portion of the interchange traffic which enters the freeway in the system of interest, origin and destination (O-D) survey techniques were selected to determine not only the location of the origin and destination of this interchange traffic but also the routes used to approach the freeway and to exit from the freeway.

All study techniques designed to give information on the origin and destination of motor vehicle trips were considered for use in this survey. Two requirements of the study affecting the design of the technique were (a) that interference of traffic must be minimized when working with freeway traffic; and (b) that the survey is made for a specific group of motorists, namely, the interchange traffic.

The "lights-on" technique was considered because it can be applied to a specific segment of the traffic such as entrance ramp traffic (1). This procedure instructs the motorists in the traffic stream to be studied to turn on their headlights. Observers stationed at various locations record the lane in which the vehicles are traveling and the time of day as they pass. This technique was rejected because the data would be limited only to travel patterns of the interchange traffic on the freeway lanes.

The roadside interview technique was rejected because of the delay and inconvenience to the motorist and the possible distraction to the freeway traffic. Home or business interviews would require a very large sample to obtain efficient data from the particular segment of freeway traffic being studied.

A technique which combines the use of field observations and mailed questionnaires was developed for use in a Los Angeles freeway study and later adopted for use in a study in Chicago (2, 3). This procedure requires that the license plate numbers of all

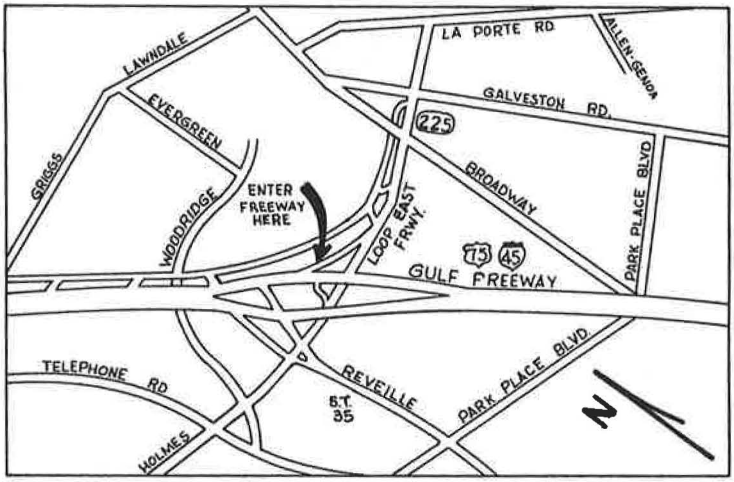
of the vehicles' owners are then obtained from the motor vehicle registration records and questionnaires with return postage guaranteed envelopes inclosed are mailed to these persons. Since favorable returns were received in these studies in Los Angeles and Chicago, the same type of questionnaire form and mail return procedure was selected for this study.

However, the method of distribtuion of the questionnaires was changed. Each motorist entering the ramp during the study period was stopped and issued a questionnaire, a return postage guaranteed envelope and a letter of explanation (Figs. 1 and 2). The forms could usually be issued at a rate faster than the vehicles could merge into the freeway so the motorists experienced little additional delay. This technique also reduces the possibility of vehicles queueing at the ramp and thus distracting the attention of the freeway motorists. This procedure has several advantages:

1. Hundred percent distribution is assured. This means that 100 percent of the traffic of interest has an opportunity to complete and return the questionnaires. When forms are distributed by mail, the owner of each vehicle is contacted and he is not necessarily the driver of the vehicle.

2. Recording and addressing errors are eliminated. The requirement to record the license number of each vehicle entering the freeway increases the possibility of not contacting the driver of the vehicle, either by recording the wrong license number, by looking up the wrong license number, or by copying down the wrong address from the records.

3. Motorists may fill out the form immediately after the trip is completed, thus eliminating some of the problems of incomplete forms and bad data. The motorist is able to recall more accurately the details of the trip and to give good information on such questions as the time of arrival and time of departure for the trip, which are



The following questions concern the trip being made at the time you receive this questionnaire.

1. Please draw a line directly on the above street map showing the route you followed in reaching the indicated entrance ramp. If the origin of the trip is not included in the area shown, extend the route to the border of the map.

2. Where did this trip begin?

____
Street Address City Time of Day

3. Where did this trip end?

____
Street Address City Time of Day

4. What exit ramp did you use to leave the freeway? (Check One)

___Exit No. 7-Wayside	___Exit No. 4-Calhoun-Elgin	___Exit No. 1-Sampson
___Exit No. 6-Telephone	___Exit No. 3-Cullen	___Pease Street
___Exit No. 5-Lombardy	___Exit No. 2-Scott	___US 75 North-Calhoun St.

5. How often is this trip made between 6:30 and 8:30 a.m. (Check One)

___Seldom	___Twice per week	___Four times per week
___Once per week	___Three times per week	___Five or more times per week

6. Do you ever use other routes to make this trip? ___yes ___ no.

If yes, what major streets are used?______________________________

____

After you have completed this questionnaire, please mail it back to us in the addressed envelope at your earliest convenience to the Texas Highway Department, Research Project, P. O. Box 26656, Houston, Texas 77032.

THANK YOU FOR YOUR COOPERATION

Figure 1. Questionnaire for O-D study, Gulf Freeway.

sometimes difficult to recall when related to a trip that occurred one or more days past.

4. Time of arrival for each vehicle can easily be determined by taking periodic time checks and noting the number of the questionnaire being distributed. By issuing the questionnaires in order, the numbering system can indicate the rate of flow for the metered input to the freeway and permits the data to be analyzed for shorter time periods.

As the questionnaires are returned through the mails, the forms can be coded and the origins and destinations can be assigned zone numbers according to the official zoning index of the city being studied so that the information obtained from this survey will complement that obtained in metropolitan O-D surveys of the city. This is helpful in expanding the sample of questionnaires that are returned to the study. Since there are several parameters and many questionnaires involved, the data should be translated to punch cards so that machine sortings and tabulations can be made.

COMMISSION
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TEXAS HIGHWAY DEPARTMENT

STATE HIGHWAY ENGINEER
D. C. GREER

IN REPLY REFER TO
FILE NO.

Dear Motorist:

We need your help in a Special Traffic Study of the Gulf Freeway which is being conducted in cooperation with the Texas Transportation Institute. This study has the objective of providing safer and more efficient operation on the Gulf Freeway. In order to develop better traffic operation on freeways, it is necessary to learn how the individual motorist uses them.

You are not required to sign any form and the information provided by you will be kept confidential. We would appreciate your completing the attached questionnaire as accurately as possible and returning it to our office.

Your participation and cooperation in this survey will be greatly appreciated.

Figure 2. Letter of explanation.

Analyses

The analysis of the data consists primarily of arranging the data [illegible] as possible. The first step in preparing the data for presentation is to determine the reliability of the data and to expand them for 100 percent return. It was found in studies conducted on the Gulf Freeway inbound traffic for the morning peak period that more than 90 percent of the traffic using the entrance ramps within the study area were repeat drivers who made the same trip five or more times a week (4). This indicated that it was not necessary to repeat the study more than one day. The percent return was exceptionally good on most of the eleven ramps being studied. A 40 percent return for a mailed questionnaire is considered good, and the results received in Houston were in the range of 45 to 65 percent return for individual ramps and averaged 55 percent for the entire study. A breakdown of the percent return by time period or by section of the area observed indicated a uniform rate of return from all segments of the traffic so that a straight expansion based on percent return was considered reliable.

The following sections show how the information can be analyzed and tabulated for application to the planning and development of a control system.

Traffic Assignment.—The route used to approach the freeway can be determined from a sketch made on the questionnaire by the driver and from the location of the origin of the trip with respect to the entrance ramp used. This information is tallied for each trip on a map of the major arterial streets leading to the freeway. Since the time period in which each questionnaire was distributed is known, these data can be obtained for any time period to indicate the volumes of interchange traffic on the street system at any time of day. As the questionnaires are tabulated, volumes of interchange traffic using each of the various streets leading to the freeway are calculated. These volume counts can be used to determine the percent of turning movements at major intersections involving interchange traffic. These data can be summarized for each entrance ramp, or a combination of several entrance ramps, and by time of day for one or more entrance ramps. Figure 3 shows a summary of the number of vehicles

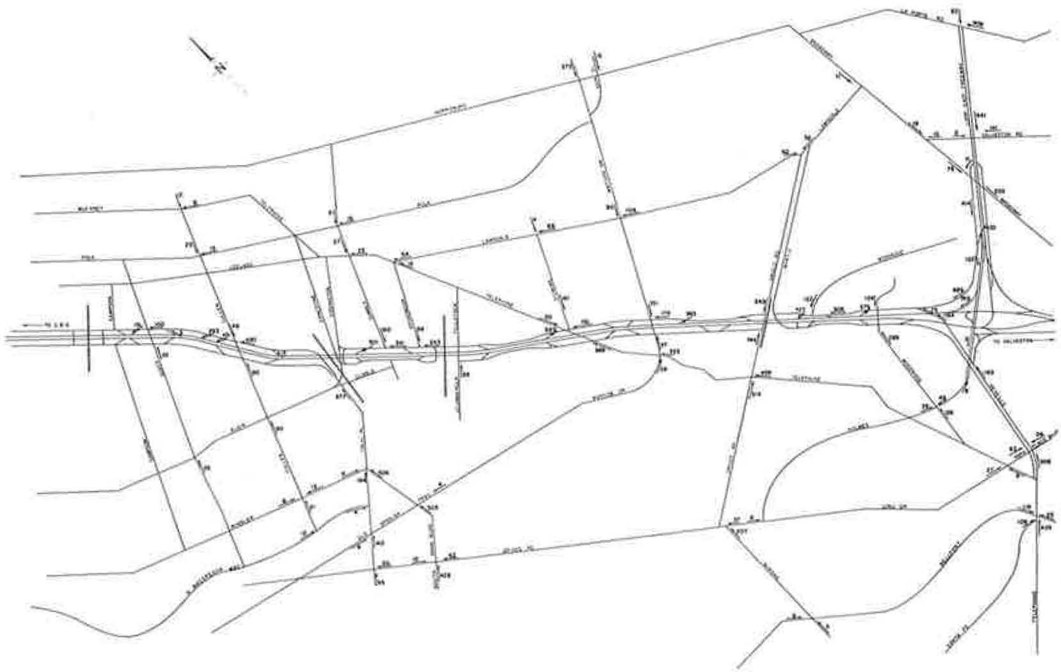

Figure 3. Interchange traffic assignment, 6:45 to 8:30 a.m. volumes.

entering the freeway during the entire study period of 6:45 to 8:30 a.m. for the Gulf Freeway survey. Figure 4 indicates the same information for a single ramp.

The volumes shown at some distance away from the freeway are assigned to the system by engineering judgment based on the knowledge of the street system and location of the trip origins. These estimates can be misleading due to the omission of minor streets that can be used as feeders to the major street system. However, the estimate becomes more reliable at the arterial streets closer to the freeway because most traffic uses the major arterial system to approach the freeway, and the sketches included on the questionnaires provide some information on the approach routing.

Summary Tables.—Some of the data can be summarized in tables for one or more entrance ramps (Table 1). The data on the percent return, the exit ramp used, the frequency of use, and the use of alternate routes can be presented in time intervals compatible with the study control time. The distance in miles between the entrance ramp and every downstream exit ramp can be included on this form to indicate the number of short freeway trips that are generated from each of the entrance ramps in the study area.

Of particular interest in the estimation of demand at the freeway bottlenecks is the percent of vehicles entering the freeway at any entrance ramp which exit at each downstream exit ramp. Table 2 shows a summary of these data obtained from the studies on the inbound Gulf Freeway (4).

Freeway Trip Lengths.—Based on the information developed in the preceding tables, the distribution of trip lengths of traffic entering the study section from the entrance ramps can be determined, as well as the average freeway trip from each ramp (3, 4).

Freeway Area of Influence.—With the addresses of the origins and destinations of the interchange traffic trips known, the area of influence of the freeway can be determined. The locations of the origins can either be plotted separately according to street address or can be grouped according to the official zoning system if the zones are broken down into very small areas. A tabulation of the trips from zone of origin to zone of destination can be made, and the desire lines drawn from the centroid of the zones indicate graphically the overall influence of the freeway (Fig. 5).

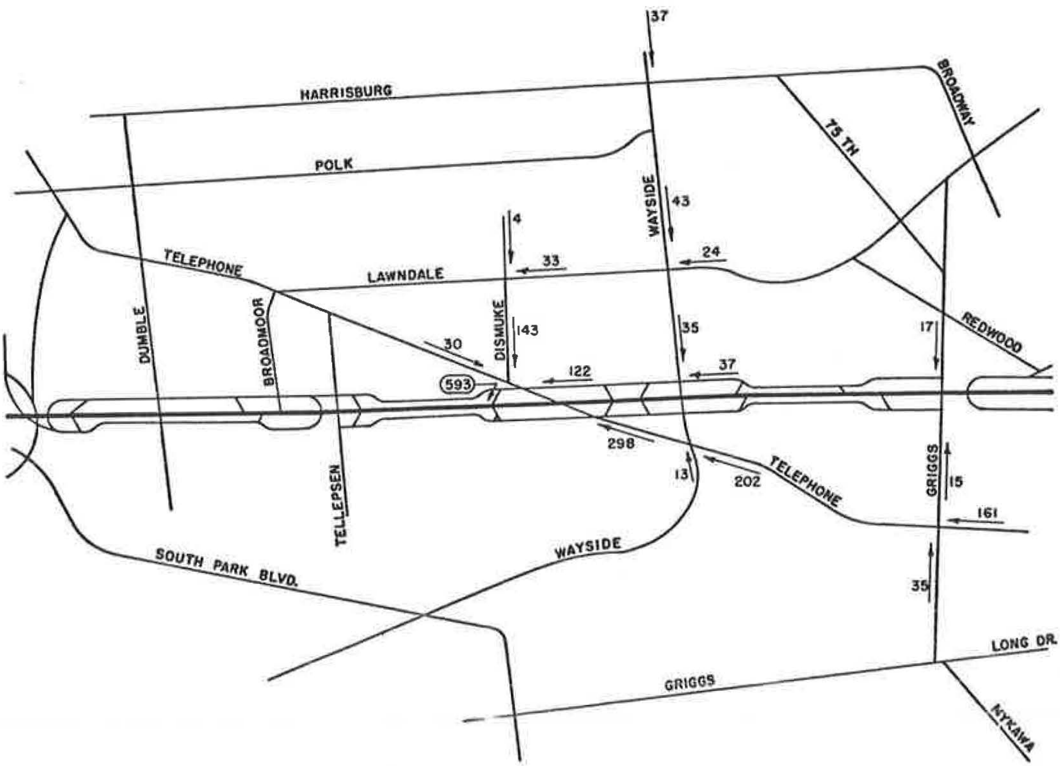

Figure 4. Assignment of interchange traffic entering Gulf Freeway at telephone entrance ramp.

TABLE 1

RESULTS OF QUESTIONNAIRES RECEIVED FROM GRIGGS-MOSSROSE RAMP

Question	Time of Day Traffic Entered Freeway (a. m.)							Total	Expanded Totals
	6:50 to 7:00	7:00 to 7:15	7:15 to 7:30	7:30 to 7:45	7:45 to 8:00	8:00 to 8:15	8:15 to 8:22		
Total distributed	24	145	143	164	179	71	24	750	750
Total returned	14	101	93	113	114	45	13	493	750
Percent return	58.4	69.6	65.0	69.0	63.6	63.5	54.1	65.8	100.0
Exit ramp used:									
No. 9—Woodridge	-	-	-	-	-	-	-	-	-
No. 8—Griggs	-	-	-	-	-	-	-	-	-
No. 7—Wayside	1	15	15	22	40	14	4	111	169
No. 6—Telephone	-	1	7	5	7	3	2	25	38
No. 5—Lombardy	-	4	3	8	8	2	-	25	36
No. 4—Calhoun	-	7	4	8	10	4	1	34	52
No. 3—Cullen	-	2	3	5	5	1	1	17	26
No. 2—Scott	-	2	3	1	1	1	-	8	12
No. 1—Sampson	2	7	7	8	3	2	1	30	46
Pease	8	39	36	32	18	10	3	146	221
US 75—Calhoun	2	19	12	22	22	6	1	84	128
Did not indicate	1	5	3	2	-	2	-	13	20
Frequency of use:									
Seldom	-	-	-	-	-	3	-	3	5
Once/wk	-	1	-	-	-	1	-	2	3
Twice/wk	-	-	2	-	-	-	1	3	5
Three/wk	-	2	-	-	-	-	-	2	3
Four/wk	-	-	1	1	2	1	-	5	8
Five or more/wk	14	96	90	112	112	40	12	476	723
Did not indicate	-	2	-	-	-	-	-	2	3
Other routes:									
Yes	5	37	37	35	58	19	7	189	301
No	9	61	56	76	55	24	6	287	497

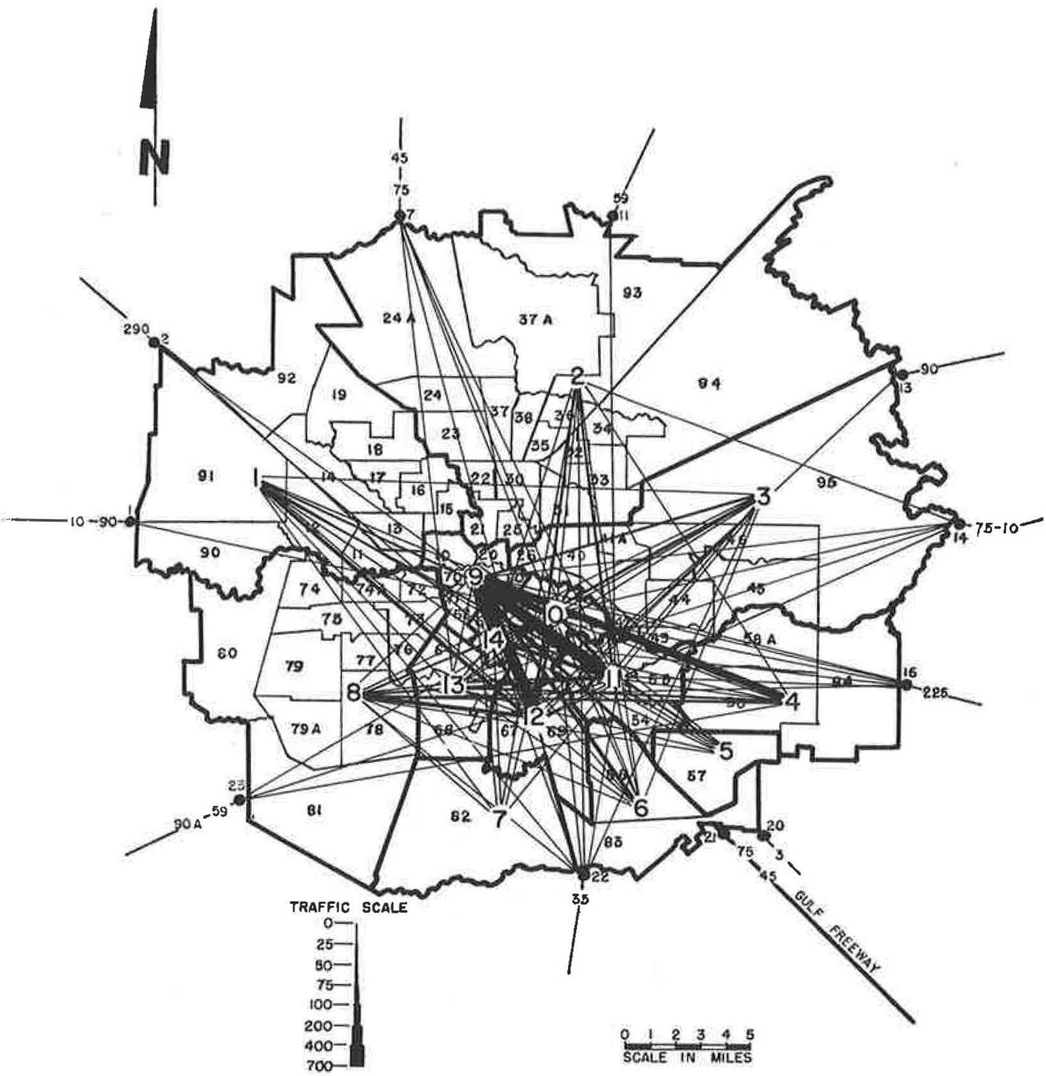

Figure 5. Houston District map, trip desire lines, Gulf Freeway, 6:45-8:30 a.m.

The areas of influence of the individual entrance and exit ramps can be indicated in similar plots (Fig. 6). The origins and destinations of the interchange traffic that used one of the entrance ramps within the study section are linked by desire lines drawn to the freeway ramps used. These desire lines indicate the distribution of the freeway trip lengths from each of the entrance ramps and the possible diversion routes that could be used by some of the interchange traffic.

The areas of influence for individual exit ramps can also be shown in a similar manner.

Application of Data. —Data developed from the O-D studies of freeway interchange traffic have direct applications to the planning and implementation of freeway control. Flow maps of the interchange traffic volumes on the city streets provide the information for determining where the traffic might divert if access to the freeway is changed. Possible alternate routes are easily determined from the traffic assignment map.

The persons to be affected by changes in control could be contacted through the mails or by personal handouts at the ramps to advise them of the control procedure and alternate routes.

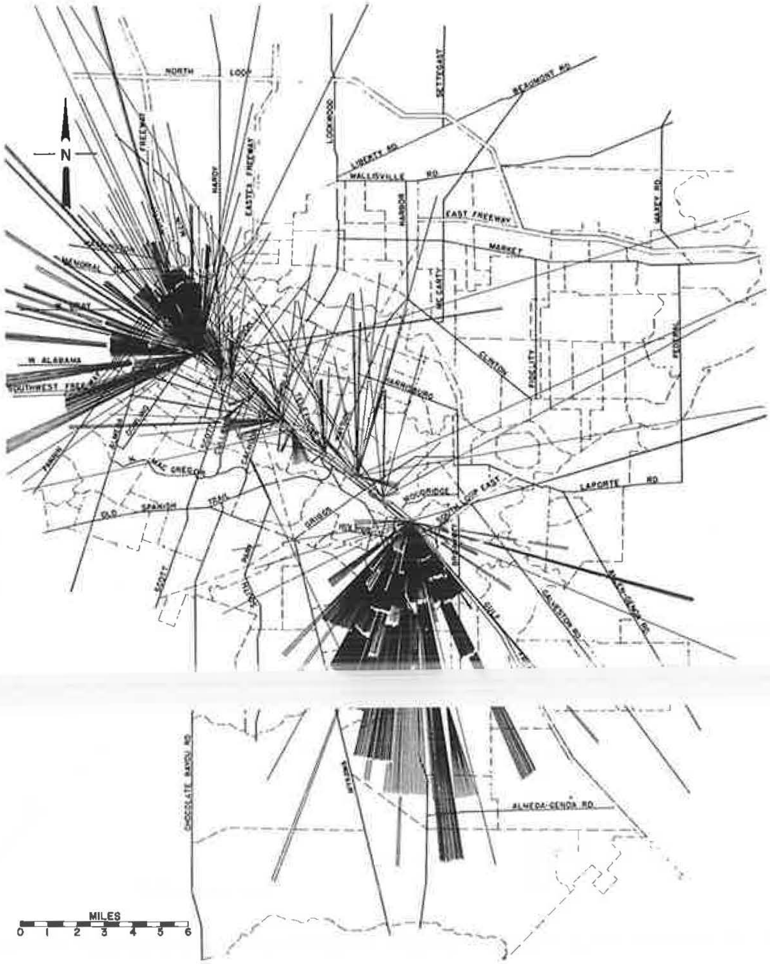

Figure 6. State 35 entrance ramp area of influence, O-D desire lines via freeway link, 6:45-8:30 a.m.

TABLE 2

O-D TABLEAU, PEAK-PERIOD

Entrances	Percent Exiting at												
	SH 225	SH 35	Woodridge	Exit 8	Wayside	Telephone	Lombardy	Calhoun-Elgin	Cullen	Scott	Sampson	Pease Dist.	Calhoun Dist.
Freeway at Broadway	7.2	5.7	7.3	2.1	11.2	3.8	0.7	9.5	2.6		Not calculated		
Detroit	-	-	7.8	2.4	12.9	4.4	0.8	10.9	3.0		Not calculated		
SH 225	-	-	0.25	0.25	2.5	0.5	1.2	7.9	5.4	2.5	5.4	38.9	35.2
SH 35	-	-	-	1.5	6.1	2.7	3.6	6.4	3.5	2.8	6.3	37.3	20.8
Woodridge	-	-	-	0.0	8.6	1.3	3.6	8.3	4.7	2.6	6.0	35.3	29.6
Mossrose (+ K Illegal)	-	-	-	-	23.1	5.2	5.2	7.1	3.5	1.7	6.3	30.4	17.5
Griggs	-	-	-	-	-	1.8	2.3	8.3	3.4	2.9	10.3	42.3	28.7
Wayside	-	-	-	-	-	0.4	1.8	15.1	5.3	1.3	4.9	36.1	35.1
Telephone	-	-	-	-	-	-	-	13.4	4.2	2.9	4.6	42.5	32.4
Dumble	-	-	-	-	-	-	-	-	5.6	1.1	5.1	44.7	43.5

The distribution of trip lengths can point out the need for design or control changes that would eliminate the very short trip in favor of the longer trips.

The traffic assignment map indicates the more promising locations for the establishment of advisory or regulatory control to divert traffic to alternate routes.

FREEWAY DEMAND-CAPACITY STUDIES

Freeway Subsystem Input-Output Studies

This study technique has its theoretical basis in the continuity equations of traffic flow which have been discussed in previous reports (5, 6). These equations state that at any instant the rate at which vehicles are entering a closed system equals the rate at which they are leaving the system plus the rate at which they are being accumulated within the system. Alternately stated, the change in the number of vehicles in a closed system in a time period equals the difference between the number of vehicles which enter and the number of vehicles which leave the system during the time period.

When using this study technique in the operational analysis of a congested freeway system, the first step is to determine the boundary points of the system of interest. The upstream boundary should be upstream of all congestion so the counts at this location represent the demand on the freeway. The downstream boundary should be downstream of present congestion and possible future congestion. (If control measures are successful in increasing the flow out of the presently congested system, congestion may develop at some downstream locations.) Air photo data can be quite useful in guiding the selection of the boundaries of the system of interest.

When the system of interest is defined, manpower requirements and availability will probably make it necessary to divide this system into closed subsystems for the analysis. Each closed subsystem consists of a freeway input count, a freeway output count and a count of each of the intermediate entrance and exit ramps. Two men are required at each freeway count location and one man is required at each ramp count location.

The subsystems should be mutually exclusive and collectively exhaustive; that is to say, they do not overlap but together they include the entire system of interest. A known or suspected bottleneck should be chosen as the division points between subsystems. In this way the (freeway) counts there can be used in the estimation of the capacity of these bottlenecks.

Data from all count locations should be recorded simultaneously at regular intervals and, to eliminate the need to reset the counters and to facilitate the analyses, should be recorded in cumulative form. Five-minute time periods proved very satisfactory on the Gulf Freeway studies. In this way the total number of vehicles entering and leaving the system in the time interval, as well as the change in the number of vehicles within the system, can be determined.

The counts are progressively started and stopped by driving a signal car through the subsystem being studied. At the beginning of the study the signal car drives through the subsystem and the upstream freeway counters begin by counting the signal car as soon as it crosses their reference line. Each man counting at an entrance ramp begins by counting the first vehicle on his ramp to enter the freeway after the signal car passes; the exit ramp counters begin with the first vehicle to leave the freeway after the signal car passes. The freeway output counters also begin by counting the signal car as it crosses their reference line.

The driver of the signal car counts and records the net number of vehicles which pass him in the study subsystem (both when starting and stopping the counts). If this is done, the number of vehicles within the closed subsystem is known as soon as the last count is started. The number of vehicles in the subsystem is also known once each 5 min (each time the data are recorded), since the net number of vehicles crossing the cordon line is known. (One can immediately see the similarity between this technique and a parking accumulation study. Indeed, a peak period freeway study is all too often a parking study.) By progressively stopping the counts in the same way, a check on the counts is obtained since the total number of vehicles counted entering the system should equal the total number counted leaving the system.

At freeway count locations, the second man was used to obtain speed samples—one sample per lane per minute. In this way the quality as well as quantity of flow was determined and these data were used to aid in the location of critical bottlenecks.

Limitations of This Type of Study.—When properly conducted, a study of this type yields a wealth of valuable data. There are, however, many things which can void part of these data.

It is essential that the watches of the study personnel be synchronized before the study so that the count data are all recorded at the same times. This is necessary to assure that at each recording time the number of vehicles in the system is accurate. If, for example, a freeway count is recorded 30 sec late, it could be as much as 50 veh in error (assuming a three-lane section flowing at 100 vpm). This error would also be reflected in the number of vehicles in the system, which might be about 50 to 500 when correct. Thus, a 50-veh error could be a large percent error and would be especially serious if it were carried through the entire study period. This can be a more noticeable problem on short subsystems in which the number of vehicles in the subsystem is small.

Similarly, the accuracy of the counts is extremely critical in studies of this type. One inaccurate count can void all of the input-output analyses.

Also, because of the interdependence of count locations, it is essential that all of the study personnel arrive on time for the study. This can become a special problem when the morning peak traffic is studied (due to early hours involved), as in the case reported here. When less than the proper number of men show up, the closed system study has to be postponed (unless substitute personnel are available).

Data Analyses.—Several analyses can be made of the data of each of the subsystem studies.

Total Input to the Subsystem vs Time.—If for each of the subsystems, coordinated counts were made at two locations on the freeway and all of the entrance and exit ramps

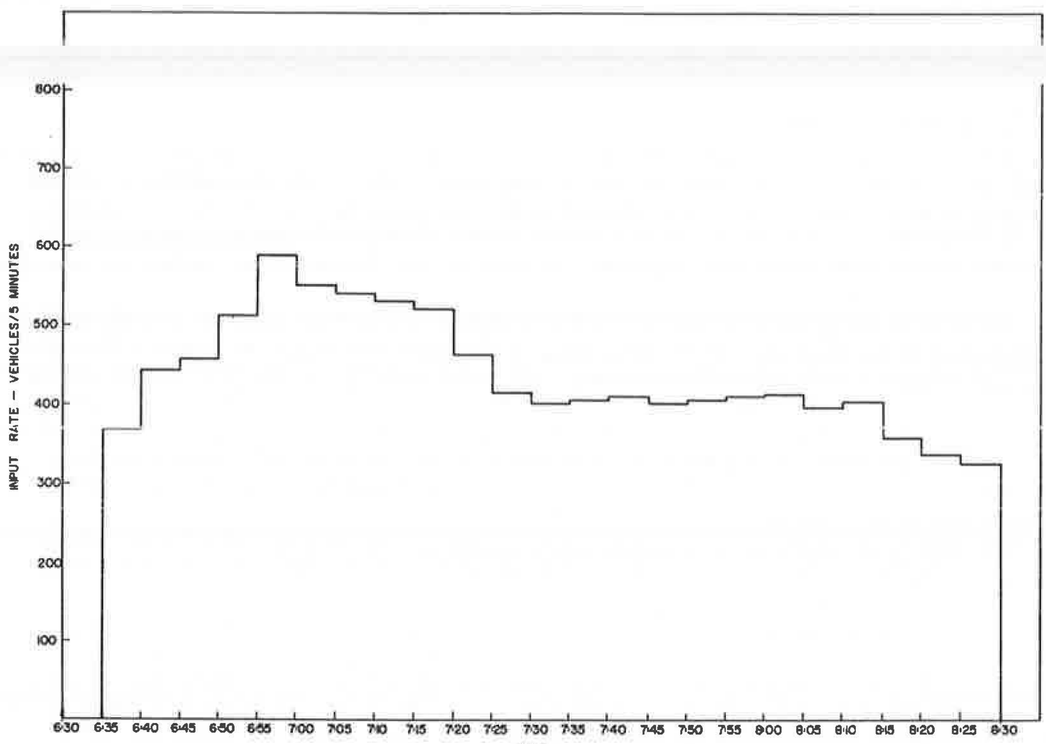

Figure 7. Input rate to Broadway—Griggs subsystem.

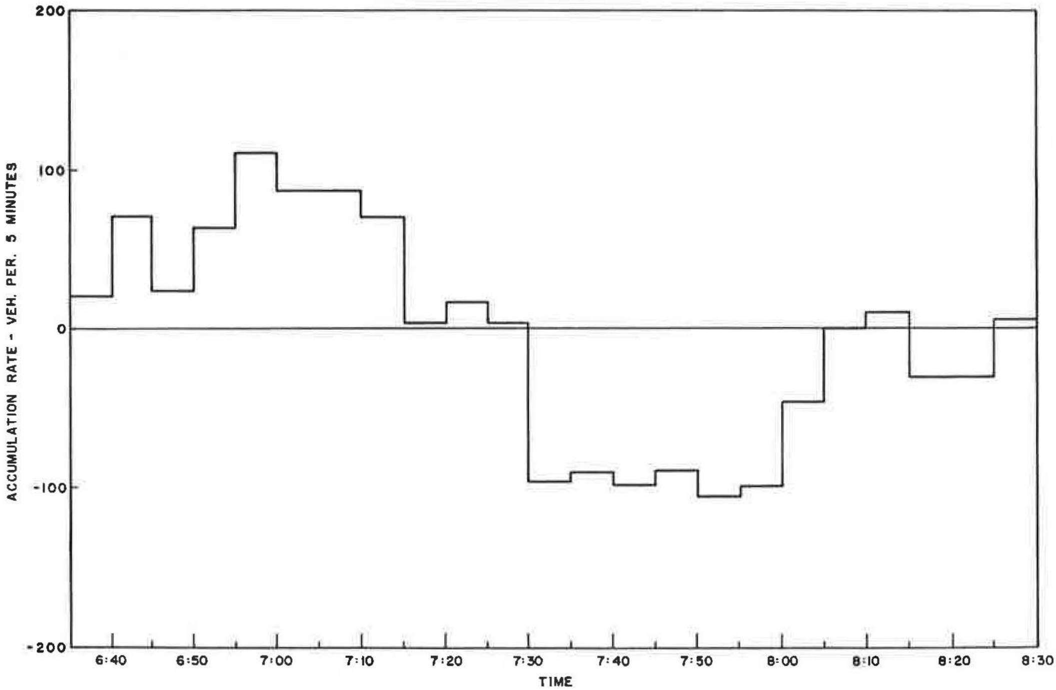

Figure 8. Rate of accumulation on freeway from Broadway to Griggs overpass.

in between, for each subsystem, then, the input locations consisted of the upstream freeway location and all of the entrance ramps in the subsystem.

Since all count data were recorded simultaneously each 5-min, the total number of vehicles entering the subsystem in each 5-min period could be determined by adding the individual inputs for each 5 min. When this is done, the resulting total input can be plotted by 5-min time periods. Figure 7 shows an example of this plot obtained for one subsystem on the Gulf Freeway studies.

Total Output from the Subsystem vs Time.—In a similar fashion, the total number of vehicles leaving each subsystem in each 5-min period can be determined. The resulting total output of each subsystem can be plotted by 5-min time periods.

Accumulation in Each Subsystem vs Time.—In a closed system, for any time period the difference between the number entering the system and the number leaving the system is the change in the number in the system. In each of the freeway subsystems (each a closed system), the difference in the total input and total output for a 5-min period is the number of vehicles accumulated or stored in the system during the time period.

The difference between the total input and total output for each system can be calculated and plotted. Figure 8 shows an example of this type of plot obtained for the same subsystem on the Gulf Freeway studies. From this figure it can be seen that the greatest rate of accumulation takes place in this subsystem from 6:50 to 7:15 a.m., that steady-state congestion prevails from about 7:15 to 7:30 a.m., and that the subsystem clears from 7:30 to 8:05 a.m. This indicates that the demand exceeds the capacity in this subsystem primarily between 6:50 and 7:15 a.m., and the entire period of congestion lasts from 6:50-8:05 a.m.

Number of Vehicles in Each Subsystem vs Time.—For each subsystem, at the time the data were recorded, the total number of vehicles which had entered the system and the total number which had left the system from the beginning of the time period are known. The difference between these gave the number of vehicles remaining in the system.

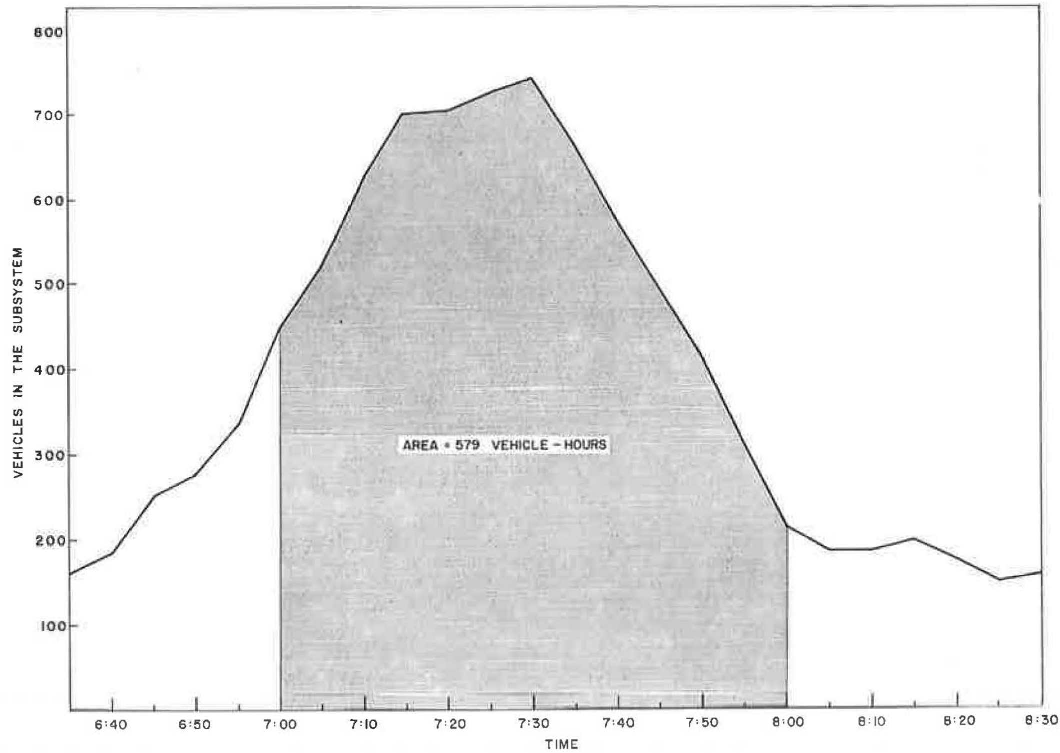

Figure 9. Number of vehicles on freeway from Broadway to Griggs overpass.

The number of vehicles in each subsystem can be plotted as a function of time. Figure 9 shows an example of a plot of the number of vehicles in one freeway subsystem on the inbound Gulf Freeway.

Since the density in a subsystem equals the number of vehicles in the subsystem divided by the subsystem's length, the number of vehicles in each subsystem can easily be converted into density which can be plotted against time.

Total Travel Time in Each Subsystem During the Peak Hour. —The area under the curve of the number of vehicles in a system vs time in any time period is the total travel time accumulated by all vehicles while they are in the system (5, 7). This analysis can be performed to obtain the total travel time for each subsystem for the peak hour or any other time period. The total travel time of 579 veh-hr on one of the inbound Gulf Freeway subsystems can be seen in Figure 9.

The total travel time in a freeway subsystem can be calculated within 1 to 2 hr after the end of a study by one man with a desk calculator. This speed of analysis makes this technique quite useful in before-and-after studies to determine the effect of a control measure since it is desirable to know this effect immediately.

Accuracy and Error Distribution. —Since the total input count during the study period should equal the total output count for each subsystem, a check on the accuracy of each day's data is available.

The errors are distributed evenly throughout the study period. If, for example, on a given day the total input count was 25 veh greater than the total output count and there are 25 5-min time periods (6:30 to 8:35 a.m., for example), the output can essentially be increased by one vehicle in each 5-min time period. In the Gulf Freeway studies no data were used in the analyses if the discrepancy between input and output counts was greater than 1 percent.

Capacity Counts. —In addition to the freeway counts which are a necessary part of the closed subsystem studies, additional freeway counts can be made for the purpose

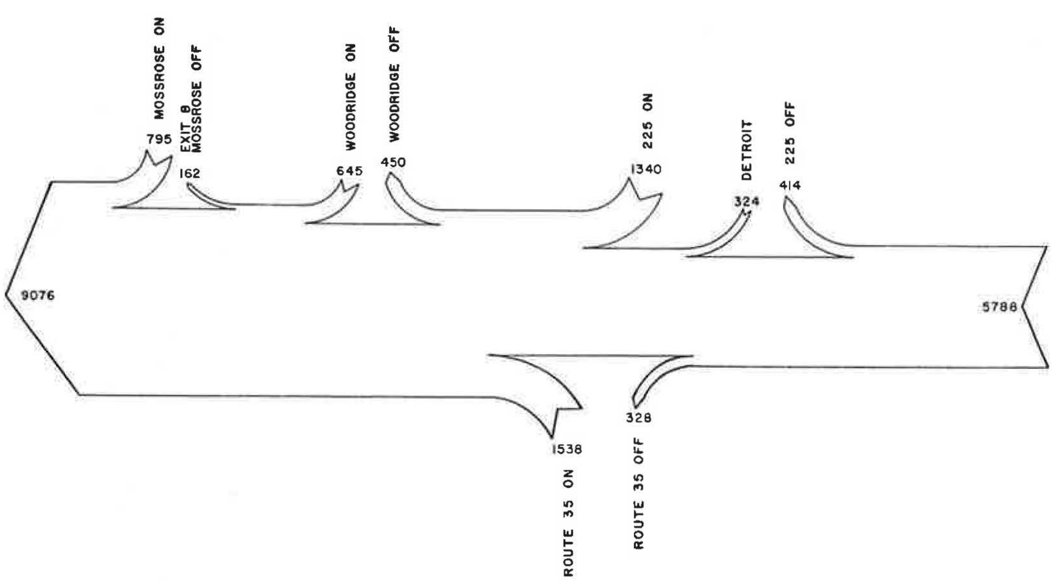

Figure 10. Study period flow map, Broadway-Griggs subsystem.

of refining estimates of the capacity flow rate at each critical bottleneck. In these studies, the volume count data were recorded each 5 min, and 15-min volumes were used for the purpose of estimating capacities.

Speed samples were obtained at the count locations (a sample of one vehicle per lane each minute using stopwatches to time sampled vehicles through a trap) during each of these counts. The speed data were useful in indicating when the flow at a location was reduced due to a backup of congestion from downstream.

Application of Data

Estimated O-D Data for Freeway.—The O-D surveys at the entrance ramps to a freeway can be made with little trouble. Similar surveys for the freeway at the upstream input source, in all but a few exceptions, are not feasible. Hence, it is necessary to estimate the peak period O-D data for this freeway input. The necessary information at this location is the distribution of traffic at this location to the downstream exits, i.e., the percent of vehicles at the upstream freeway input which exit at the downstream ramps. An example of some of the computations used to determine these O-D percentages for the freeway input for the Gulf Freeway studies is used to present the technique.

The O-D studies yielded information on the percent of vehicles which leave the freeway at each downstream exit for each entrance ramp included in these studies. However, similar data were not available for the Detroit St. entrance ramp and the freeway near Broadway (both at the upstream end of the system of interest). It was possible to combine the existing O-D data with the closed system counts to estimate this data for the other two input sources (Detroit entrance and freeway near Broadway).

Figure 10 is a flow map containing the numbers used for some of the estimations. Two very obvious calculations were made first. Of the 5,788 veh which entered the system on the freeway at Broadway, 414 left on the SH 225 exit ramp and 328 left on the SH 35 exit ramp. (It is almost impossible for vehicles to enter the Detroit entrance ramp on the right and exit at SH 35 on the left—especially during the hours studied.) This means that during the study period, 7.2 percent of the vehicles on the freeway at Broadway exit at SH 225 and 5.7 percent of them exit at SH 35.

Of the 450 veh exiting at the Woodridge exit ramp, 3 (0.25 percent of 1,340) are estimated to have come from the SH 225 entrance. This leaves 447 which had to come from the freeway at Broadway and the Detroit entrance. The assumption was made that the O-D characteristics of the vehicles entering at the Detroit ramp are the same as those of the vehicles which are on the freeway just upstream of the Detroit entrance ramp, i.e., those vehicles on the freeway at Broadway which do not exit at SH 225. Of the 5,698 veh downstream of the Detroit entrance ramp, 447, or 7.8 percent, exited at Woodridge. Thus, 7.8 percent of the vehicles entering at the Detroit entrance and 7.3 percent of the vehicles on the freeway at Broadway (92.8 percent of 7.8) exited at Woodridge.

Similar analyses were made to estimate the exiting percentages at the other output locations of the Broadway-Griggs subsystem. A similar, somewhat more difficult procedure (combining the data of more than one subsystem) was used to estimate the percentages of vehicles from the Detroit entrance and freeway at Broadway which exit downstream of the Broadway-Griggs subsystem. The estimated O-D data for the freeway at Broadway and for the Detroit St. entrance ramp are included in Table 2.

Estimates of Demand at Freeway Bottlenecks (10).—Volume count data at each input to the freeway can be combined with the O-D data and freeway capacity estimates to obtain the demand rate at each of the known or suspected freeway bottlenecks. The demand at a given bottleneck can come from all of the upstream freeway inputs. In the case of the in-bound Gulf Freeway, all upstream inputs include the freeway at Broadway and all of the entrance locations between Broadway and the bottleneck. The demand at a bottleneck can be influenced by the capacity of upstream bottlenecks.

Some vehicles from each of the upstream inputs may go through the bottleneck and some may exit upstream of the bottleneck. Thus, only some portion of each upstream input volume represents demand at the bottleneck. If there are no bottlenecks upstream of the one under consideration and if for each input the percentage of vehicles which pass through it is known, an estimate of the demand in the i^{th} time period is:

$$D_i = \sum_{j=1}^{n} P_{ij}V_{ij}, \quad i=1,\ldots.m \tag{1}$$

fraction of vehicles from the j^{th} input which are destined for the bottleneck, again during the i^{th} time period.

On the inbound Gulf Freeway the farthest upstream (suspected) bottleneck was at the SH 225 merge location and the freeway subsystem upstream of this location is shown in Figure 11. The number at each input is the decimal fraction of vehicles from that input which pass through the SH 225 merging section. Thus, for any time period, the demand at the SH 225 merging section equals 0.871 × freeway volume at Broadway + 1.00 × volume at the Detroit entrance ramp + 1.00 × volume at the SH 225 entrance ramp.

In estimating the demand for a series of bottlenecks on one direction of a freeway, it is best to first estimate the demand at the farthest upstream bottleneck using Eq. 1. It is then assumed that at this bottleneck the flow equals demand or capacity depending on (a) if demand is less than capacity and (b) if there is a storage of vehicles upstream of the bottleneck (caused when demand exceeds the capacity). The demand at the next downstream bottleneck, disregarding the bottleneck upstream of it, can be obtained using Eq. 1. This must then be altered to take account of the storage of vehicles at the upstream bottleneck. The demand at this downstream bottleneck can then be compared to its capacity. The procedure can be repeated to obtain estimates of the demands at successive downstream bottlenecks.

Five-minute demands were computed in this way at six bottlenecks on the inbound Gulf Freeway. Figure 12 shows the demand estimate at the Griggs Rd. overpass bottleneck.

In these computations no attempt was made to take into account the temporal separations of the various locations. In other words the demand at a bottleneck was estimated during a certain time period using the upstream inputs during the same time

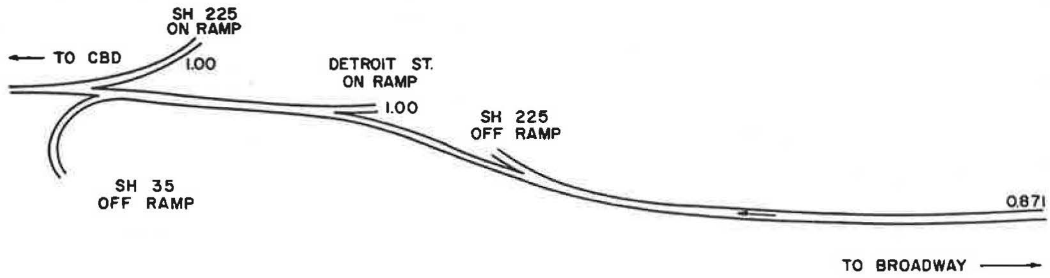

Figure 11. Freeway subsystem upstream of SH 225 merging section.

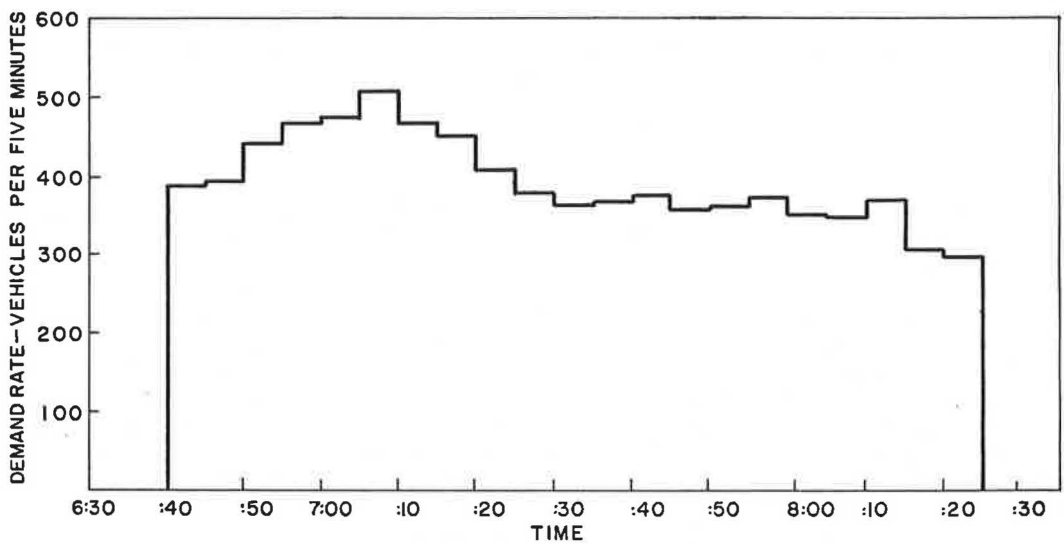

Figure 12. Demand at Griggs overpass.

period, thereby disregarding the travel times between the inputs and the bottleneck. A more sophisticated demand analysis on a longer system could take this into account.

Estimates of Capacity at Freeway Bottlenecks.—The estimates of the capacity flow rates at the freeway bottlenecks were straight forward (based on manual counts at the bottlenecks) and little explanation is needed. The count data were recorded by 5-min time periods and the highest 15-min flow for each day of data was used as the basis for the estimated capacity values. Many days of data were collected to estimate the capacity values accurately.

Interpretation of Demand-Capacity Data

Before congestion develops on a freeway, the demand is less than the capacity at each bottleneck on the freeway. As the peak period progresses, the demand will increase to an amount greater than the capacity at one or more bottlenecks and congestion will develop. The excess of demand over capacity is stored on the freeway in the form of a queue of high density. As the queue backs past upstream exit ramps, vehicles which are going to exit without passing through the bottleneck are trapped in the queue and are thereby delayed. Thus, the queue reflects not only the amount of

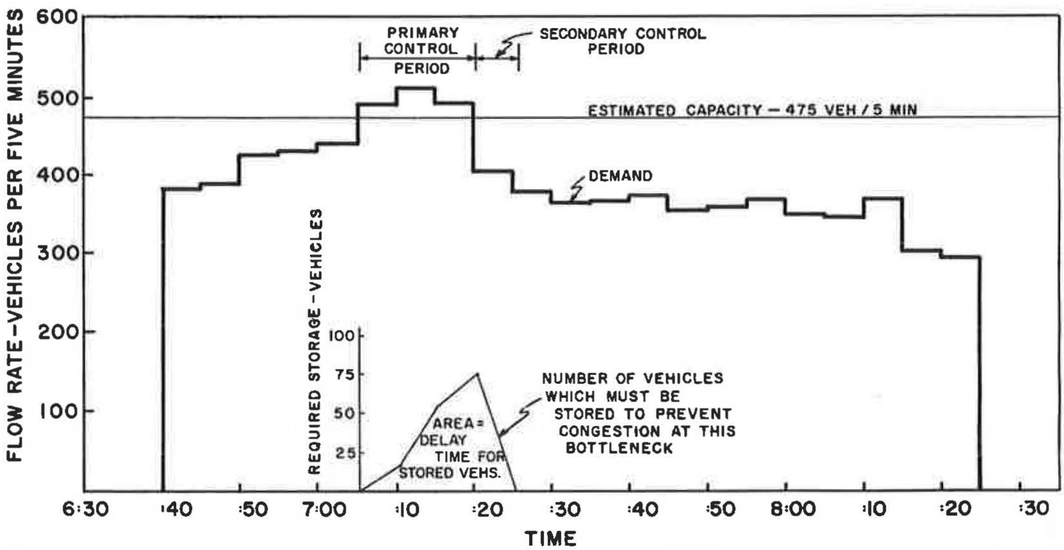

Figure 13. Demand and capacity at hypothetical freeway bottleneck.

excess demand but also some of the exiting vehicles. If the queue extends for several miles, as is frequently the case, the number of exiting vehicles trapped in the queue may be quite large.

If a control system is to prevent congestion, it must prevent the demand from exceeding the capacity at each bottleneck in the system. At any bottleneck the excess of demand over capacity would have to be stored (on the ramp, frontage road, or street) or diverted during the primary control period, which is the time during which the [illegible] below the capacity, some of the stored vehicles can be released onto the freeway at a rate which again will keep the total demand at the bottleneck less than or equal to the capacity. The portion of the control period during which stored vehicles are released could be called the secondary control period.

The storage of the excess of demand over capacity as described in the previous paragraph somewhat implies an entrance ramp metering scheme. If a short-time ramp closure scheme were envisioned, the ramp would have to be closed only during the primary control period since no vehicles would be stored (i.e., all would be diverted). This would alter the demand at other ramps, however.

Since the demand-capacity study technique outlined in this report can be used to estimate the demand rate as a function of time and also the capacity flow rate at each bottleneck, it yields data which are amenable to interpretations regarding peak period control. From the demand and capacity estimates, it is possible to estimate for each bottleneck the length of time which the demand exceeds the capacity (primary control period), the amount of excess demand in each time period, the number of vehicles which would be stored at the end of each time period and the length of time required to clear the stored vehicles (secondary control period).

Figure 13 shows the demand and capacity at a hypothetical bottleneck. From 7:05 to 7:20 a.m. the demand exceeds the capacity so this is the primary control period. The number of vehicles which must be stored rises during the primary control period, reaching a maximum at the transition from the primary to the secondary control periods. The figure shows that a maximum of about 75 vehicles must be stored or diverted at this bottleneck and the maximum storage occurs at 7:20 a.m.

The secondary control period which is required to clear all of the stored vehicles in this example lasts from 7:20 to about 7:26 a.m. After 7:26 a.m., the demand is less than capacity and all stored vehicles have been cleared so the system can maintain congestion-free operation without strict controls. Hence, the total control period lasts from 7:05 to 7:26 a.m.

The delay accruing to all vehicles while they are stored is equal to the area under the curve of the number of stored vehicles vs time as shown in Figure 13.

FRONTAGE ROAD INTERSECTION STUDIES

The primary function of the frontage road is to provide access to the freeway, but of increasing importance is the ability to provide additional capacity for the peak period movement of freeway traffic to avoid overloading the freeway. The flexibility afforded by this extra capacity can be useful under normal operation of the freeway but can be even more beneficial in cases of severe reduction of capacity on the freeway, as in the case of accidents. The at-grade intersections at the cross streets limit the capacity of the frontage road. It is of importance, then, to have the highest capacity possible at these intersections.

In most instances the intersections are part of a signalized diamond interchange so that the problem of increasing capacity is that of providing the maximum green time on the critical approach. Some design modifications can also be made to improve the flow.

Three studies that can be made for each approach to the intersection are capacity-demand determination, turning movement counts, and geometric design analysis.

Study Techniques

Capacity-Demand Studies.—Observations are made to determine the average starting delay and headway for each movement from the approach, and the capacity per cycle is calculated. These values are then checked from observed counts of the maximum number of vehicles that enter the intersection during one cycle.

The demand for each approach is determined by the input-output count procedure described earlier in this chapter. The boundary points of the system are the stop

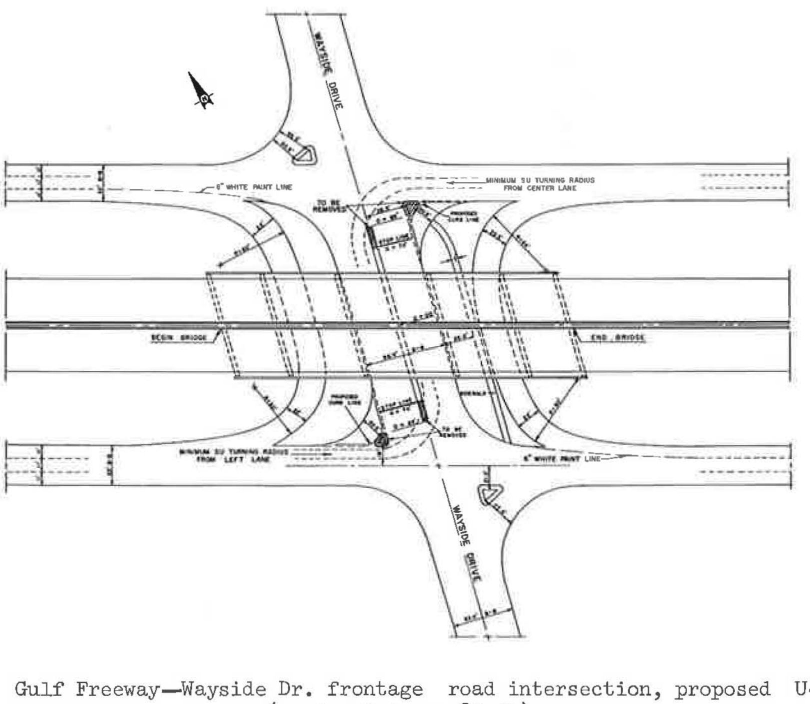

Figure 14. Gulf Freeway—Wayside Dr. frontage road intersection, proposed U-turn bays (scale: 1 in. = 80 ft).

line at the intersection and the end of the queue waiting on the approach. The system is closed by making counts at all driveways or streets that intersect the system between the boundary limits.

Two men on each approach are sufficient if there are few side counts to be made. The counts are started simultaneously by a voice or hand signal. The observer at the output of the system notes the number of vehicles in the system at the start. Data at all count locations are recorded simultaneously at regular intervals. To eliminate the need to reset the counters, the counts are recorded in cumulative form. The counts are stopped simultaneously by a voice or hand signal and the observer at the output station notes the number of vehicles remaining in the system.

It is essential that the counts at all locations be recorded at the same time. Since the time intervals for recording data are usually very short, a 10- or 15-sec difference can have a significant effect on the results. Also, the fact that the output of the system is concentrated in the shorter time intervals of the green phases of the signal cycle stresses the need for coordinated time checks.

It is advisable to select a time interval that is not an even multiple of the cycle length. For example, if a cycle length of 60 sec is being used, a time interval other than x minutes should be used, where x is a whole number. If the recording interval coincides with the cycle length, the readings will be made at the same position relative to the end of the green phase. To obtain comparable data on other approaches, or on the same approach but on different days, the same relative position would have to be used. By using a time interval other than a multiple of the cycle length, average data for the approach are obtained.

Turning Movement Counts.—The distribution of traffic by lanes and by turning movements is very important in the establishment of lane-use controls and the design of signal timing. The demand for each traffic movement is determined from vehicle counts recorded at the end of the green phase for the approach. The lane distribution is obtained from queue counts in the separate lanes recorded at the beginning of the green phase for the approach. At diamond interchanges, the left-turning vehicles from the frontage road to the cross street are observed at the second intersection to determine the number of U-turn movements.

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from plan views to determine if the capacities of the approaches can be increased. The diamond interchange in Figure 14 can be improved if:

1. Minimum turning radii are provided for the type of vehicles using the intersection and the type of control being employed. These two factors may change from time to time without including the necessary design modifications.
2. Additional lanes can be added to the approaches to accommodate the traffic demand.
3. U-turn bays or turning roadways can be added if sufficient demand for these turning movements is evident. The pattern of turning movements on any approach can change quickly as land use in the area changes.

Data Analyses

Capacity-Demand Studies.—Several analyses can be made on the data taken from each approach.

Total Input to the Approach vs Time.—The input volumes to the system represent the traffic demand on that intersection approach. These volumes can be plotted vs time and compared to a horizontal line representing the capacity of the approach as shown in Figure 15.

Number of Vehicles in the System vs Time.—At each time the data are recorded, the total number of vehicles which had entered the system and the total number which had left the system from the beginning of the time period are known. The difference between these gives the number of vehicles remaining in the system, i.e., waiting for the traffic light.

Since the length of the system is variable and equal to the distance from the stop line to the end of the queue, the number of vehicles in the system is the number of

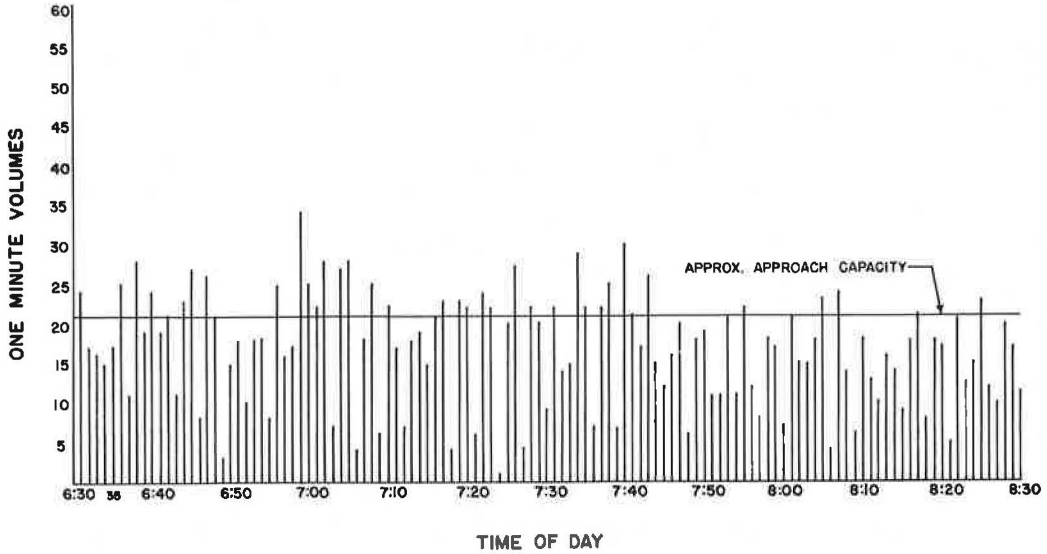

Figure 15. Wayside southbound approach, 1-min volumes, peak hour volume = 1,057.

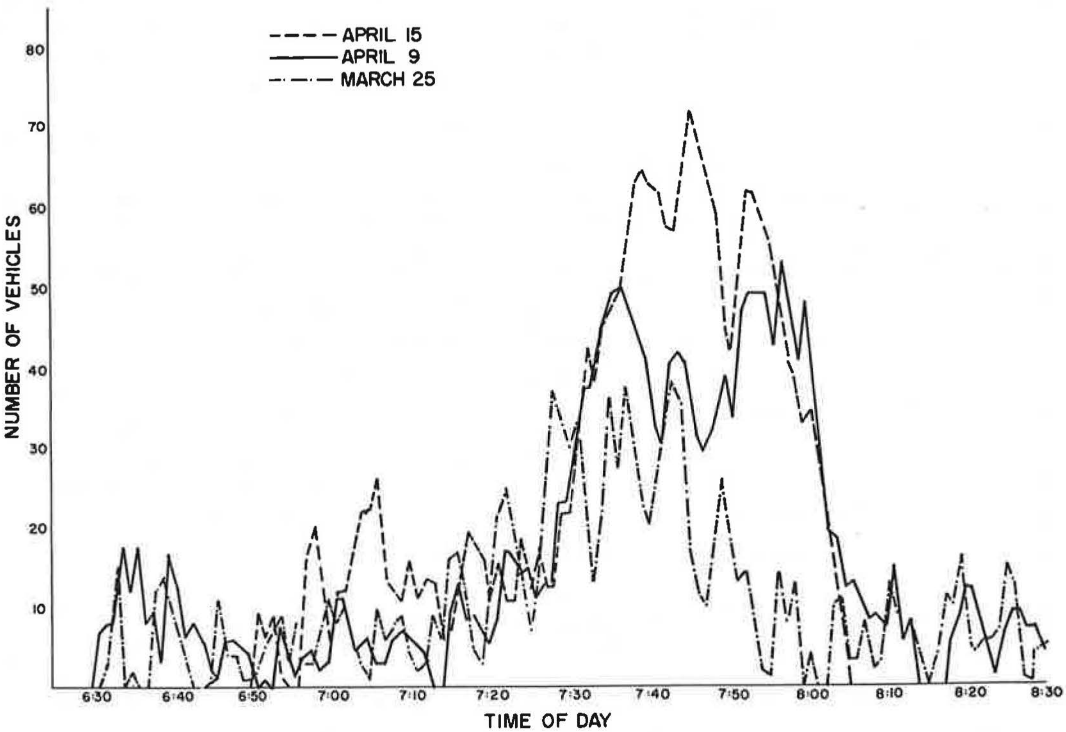

Figure 16. Wayside frontage road westbound, queue lengths, average vehicle delay between 7 and 8 a.m., 25.7 veh-hr.

vehicles in the queue delayed by the traffic signal. The number of vehicles in the system can be plotted as a function of time (Fig. 16). The area under the curve of the number of vehicles in the system vs time in any time period is the total travel time accumulated by all vehicles while they are in the system. In Figure 16 the average total travel is 25.7 veh-hr during the 1-hr period from 7:00 to 8:00 a.m.

Turning Movement Studies.—These studies include two types.

Output Movements from System vs Time.—The total traffic demand on an intersection approach is represented by the total input volume to the system over the peak period. This demand is divided into the several traffic movements which are based on the vehicle counts at the output of the system and can be plotted against time and compared to the capacity for each movement.

Total U-turn Movements.—The number of U-turn movements made during some time period indicates the demand for the U-turn bay and the resulting increase in capacity of the left-turn movement from the frontage road if separate turning roadways are provided.

Application of Data

Capacity-Demand Studies.—The curves for the number of vehicles in the system can be useful in the reapportionment of green time at an intersection. A comparison of the curves for each of the four approaches will indicate if the total travel time can be reduced by shifting green time from one approach to another.

Total travel time parameters can be used to evaluate the effects of changes in design and control such as (a) freeway control procedures that require rerouting the traffic onto the frontage roads and major arterials, (b) changes in signal timing or lane-use control on the intersection approaches, or (c) modifications in the geometric design of the approach, such as adding turning lanes and increasing turning radii.

Turning Movement Studies.—The U-turn movements can be important in the operation of a diamond interchange. The capacity of the total interchange can be reduced by one for every U-turn vehicle because of the effect on the cross street traffic. The usual diamond interchange signal phasing uses two phase overlaps (Fig. 17). For these phase

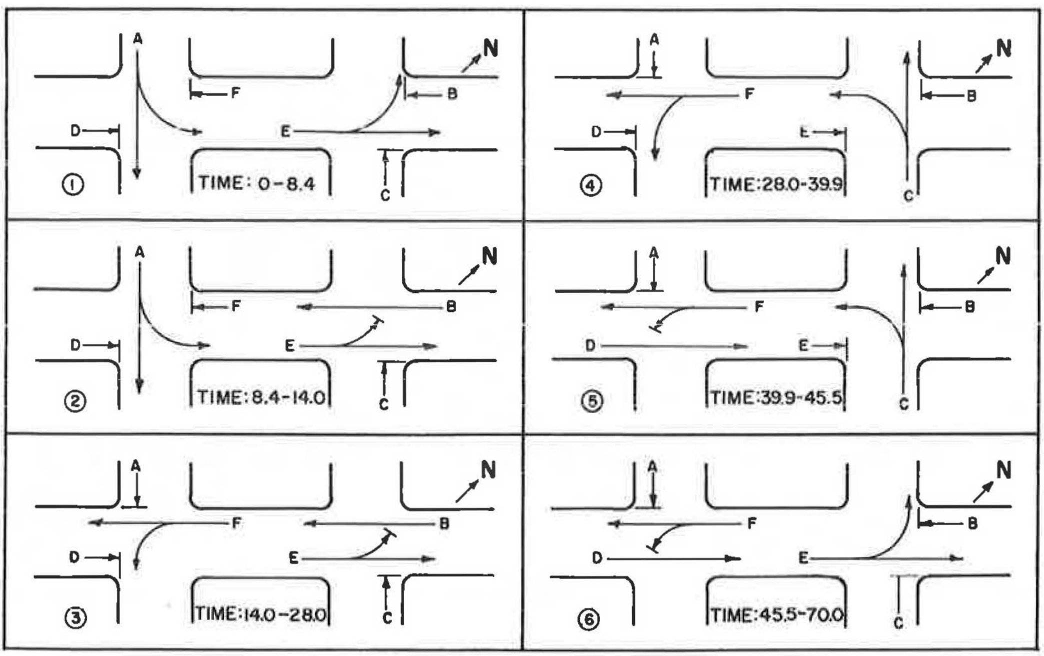

Figure 17. Phasing of traffic movements, 5.6-sec overlap existing signal timing.

overlaps to be effective, the area between the two intersections should be clear of stopped vehicles, but this timing design does not clear those vehicles that make U-turns. Therefore, they are stopped at the second intersection and reduce the capacity of the cross street.

The overall delay caused by the intersection is also increased by the time the U-turn vehicles must wait at the second intersection.

Evaluation of Geometric Design. —The analysis of the geometric design of the intersections can determine if the capacity of an approach can be improved by design modifications. The application of these results will depend on the studies of the traffic demand requirements.

AERIAL PHOTOGRAPHY

The input-output studies provide a great deal of very valuable data for a particular subsystem being studied. To obtain a true picture of the conditions of traffic on the freeway, the system should include the entire length of freeway. However, a simultaneous study of several miles of freeway would require a very large field crew if input and output counts were made at each access point.

Another method of data collection, aerial photography, can be used to obtain some data which can also be obtained by the input-output studies and the air photo technique requires a field crew of only three men. The length of the study area is limited only by the frequency of coverage required.

The application of aerial photography to traffic studies is not new, but only in the last 5 yr have there been any extensive studies of traffic operations by this method. With the expansion of study areas into systems that cover several miles, the costs of conducting aerial surveys is now comparable to those of conducting ground studies for obtaining certain types of data.

Study Technique

Time-lapse photography where individual pictures are taken at short intervals of time should be used in a study of this type. The information to be taken from the film determines the technical requirements of the photograph. One of the most natural applications of aerial photography in the study of freeway operation is in the determination of the number of vehicles within a system (similar to the data obtained in the closed system input-output studies). Therefore, only vehicle counts will be made and each picture can cover as large an area as will permit identification of the individual vehicles. Also, since no time-dependent measurements, such as vehicle speeds or gaps, are taken from the film, a minimum overlap from picture to picture is acceptable and the interval between pictures does not have to be constant.

A procedure was adapted for use in the study of the Gulf Freeway, a distance of 6 mi. The flight crew consisted of three men—a pilot, photographer, and an assistant. The plane was a fixed-wing Cessna 170. The flight plan was to make as many runs of the study area as possible during the 2-hr peak period. The plane flew in a counterclockwise pattern from one end of the freeway to the other at an altitude of 1,500 ft, approximately 300 to 500 ft to the side of the freeway. Photographs were taken through the window of the plane in both directions with a hand-held 35-mm camera equipped with a 50-mm lens. A minimum overlap was used. During each run the assistant recorded the time of the first and last pictures for the run, reloaded a spare camera, set up an identification board which was photographed at the beginning of each run, tagged the exposed film, and made appropriate notes of the flights.

Data Analysis

The times of day for the start and finish of each filming run were recorded. The time interval between pictures within a run was assumed to be constant so that the time of day for each picture could be estimated. Since several pictures on a run were required to cover each subsystem, the time of day for the picture taken at the center of each subsystem was used to reference the number of vehicles in the system.

The number of vehicles in each of the subsystems was determined by first selecting matched points on adjacent pictures so that there was no overlap of the roadway. The matched points at the beginning and end of the freeway subsystems were those used in the input-output studies. The number of vehicles observed between the matched points on each slide were counted and summed for all slides included in each subsystem. The total represents the number of vehicles in the system at the time of day the center picture was taken. It is assumed that the number of vehicles in the subsystem does not change during the time interval required to fly its length.

The number of vehicles in the subsystem can be plotted against time from this data and the total travel time can be computed. Traffic volumes cannot be determined from the aerial photographs but supplementary ground counts at a few locations can supply this information. The data can be analyzed using any freeway subsystems of interest. The same photographs can be used to make similar studies on the frontage roads as well as to study the queues on freeway entrance ramps or signalized intersections near the freeway. This technique also provides a good parameter, total travel time, for measuring the effect of change in before-and-after studies.

Aerial photographs provide data for other studies such as the lane distribution of traffic, the effect of accidents on traffic operations, the length of time required to clear accidents from the moving lanes, the effect of trucks on the traffic stream, and shoulder usage.

USE OF STUDY TECHNIQUES ON GULF FREEWAY SURVEILLANCE PROJECT

The Texas Transportation Institute, in cooperation with the Texas Highway Department and the U.S. Bureau of Public Roads, is conducting a peak period freeway surveillance study on the Gulf Freeway in Houston. The studies of the type described in this report reached full momentum about Jan. 15, 1964. Until September 1964, most of these studies were confined to the inbound Gulf Freeway in the morning peak period. Studies of the outbound freeway traffic are planned for the near future.

O-D questionnaires were distributed to motorists on each inbound entrance ramp

on the data from these studies, including, for each entrance ramp, the distribution of traffic among the downstream exits (4).

The inbound Gulf Freeway was divided into five closed subsystems and input-output studies were run on each. One week of data was collected for each subsystem. From these studies it was possible to identify three major bottlenecks, to establish flow patterns at each entrance ramp and the upstream freeway input, and to estimate the total travel time accruing to all vehicles in each subsystem during the peak period.

The freeway counts from the closed subsystem studies and supplementary capacity counts were used to estimate the capacity of each of the major bottlenecks. The entrance ramp and freeway input count data, combined with the O-D data and bottleneck capacity estimates, were used to estimate the demand at each of the three freeway bottlenecks (8).

Critical intersections in the freeway area of influence, especially those of the frontage roads and the crossing arterials, were studied and some signal retiming was accomplished (9).

A ramp control plan was developed based on the demand and capacity estimates at the three major bottlenecks. The philosophy of the controls was to keep the demand less than or equal to capacity at the two downstream bottlenecks. (A critical frontage road intersection temporarily precluded a more comprehensive and restrictive control plan which would probably have diverted a large volume of traffic through the critical signal but which would have further improved freeway operations.) In August, four ramps were closed for 20, 25, 40 and 40 min) during their primary control periods and one was metered for 40 min at a rate based on the demand and capacity estimates. The control was highly successful and produced a marked improvement in the freeway operation and had little or no adverse effect on the alternate routes. The details of this study are contained in another report.

Air photo studies were made to provide a before-and-after comparison for this study.

CONCLUSIONS

1. The O-D technique for obtaining travel patterns of the freeway interchange traffic provides very reliable results. Only in locations where the stoppage of traffic is hazardous should the method of questionnaire distribution be changed.

2. The design of the questionnaire can be improved. The definition of the trip for which the questions are asked should be included in the explanation to avoid confusions of morning and afternoon trips.

3. The high percent return indicated an acceptance by the public of this type of survey and their interest in a cure to the problem of peak period freeway congestion.

4. The information received from this type of survey has been most valuable in several phases of research on the development of control techniques for improving freeway operations.

5. The study technique using input-output counts on closed freeway subsystems is an excellent method of quantifying the problem of freeway congestion.

6. The input-output data are also quite useful in before-and-after comparisons since total system travel time is one by-product.

7. Demand at freeway bottlenecks can be estimated by combining count data at each freeway input with the O-D data and bottleneck capacity data.

8. By comparing the demand rate to the capacity flow rate at a critical bottleneck, it is possible to estimate the duration and severity of control that would be required to prevent congestion, as well as the number of vehicles that would have to be stored (queued) or diverted from the freeway at the control location.

9. Aerial photography is a good method for determining the number of vehicles using a given facility at a given time. The technique using a 35-mm camera was found to be sufficient for this determination.

10. The design evaluation of existing frontage road intersections can, in many cases, prove fruitful by leading to improvements in design or control which can increase the capacity of the intersection.

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Effect of Grades on Service Volume

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The problem of determining effects of trucks or any slow-moving vehicle on the operating characteristics of a section of multilane road is discussed. The action of trucks in reducing the service volume of a road is described and is related to the number of trucks, speed of trucks (steepness of grade), and length of grade. Relationships between these factors are developed and presented in the form of a proposed design chart for determining equal service volumes which would be suitable for rural conditions for any combination of grade, autos, and trucks. The use of this chart in determining when additional lanes should be added and the effects of trucks on maximum capacity of a road are described.

•THE PURPOSE of this paper is to develop a level-of-service curve for multilane highways that will show the effect of grades and trucks on the service volume for a given level of service, and more particularly, for the level of service that has been adopted as design capacity for rural freeways by AASHO, i.e., 2,000 cars per hour on two lanes in one direction on a level grade.

The 1950 edition of the Highway Capacity Manual stated that one truck was equivalent to two passenger cars in level terrain, four passenger cars in rolling terrain, and eight passenger cars in mountainous terrain (1). This statement can be plotted in graphical

cars per hour on two lanes (1,000 veh/hr/lane). Thus, with 10 percent trucks in level terrain, the service volume would be 1,820 vph, consisting of 182 trucks and 1,638 autos, and in rolling terrain it would be 1,540 vph, consisting of 154 trucks and 1,386 autos. This was shown in Table 9 of the Highway Capacity Manual as 91 and 77 percent, respectively, of passenger car "capacity" on level terrain.

The Highway Capacity Manual truck equivalency factor (1) was a broad approximation and many design engineers have felt a need for capacity charts which were more sensitive to such measurable factors as length and steepness of grade. How long does a grade have to be to qualify as mountainous? Are the hills of the San Francisco Bay Area "rolling" and the Sierra Nevada "mountainous"? Trucks seem to have as much effect in the former as in the latter location. In fact, a large portion of the Interstate highway across the Sierra follows river grades and the trucks roll along at 50 mph there just the same as they do in level terrain.

Attempts have been made to determine truck equivalency factors by measuring time headways between vehicles which were classified as car-car, car-truck, truck-car, truck-truck, in each lane. These attempts, in which the author participated, were not successful because observations had to be at a precise (unobserved) combination of trucks and cars to help determine the locus of the graph shown in Figure 1. In Figure 1, truck equivalent is the slope of the line. If observations are made at any pair of ordinates except right on the sloping line, they do not have any quality which would help determine either the position or slope of the line. For example, the two points (x_1, y_1) and (x_2, y_2) shown in Figure 1 are of no use in plotting the graph and really only show the traffic demand at the places and times of observation. Actual measurements at several points in California showed extreme variation in truck equivalents

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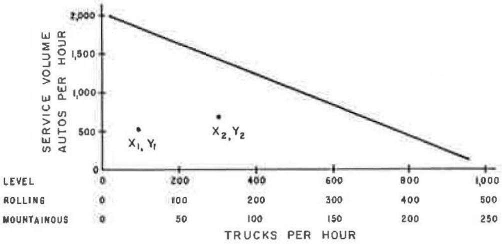


Figure 1. Effect of trucks on service volume, two lanes in one direction (1).

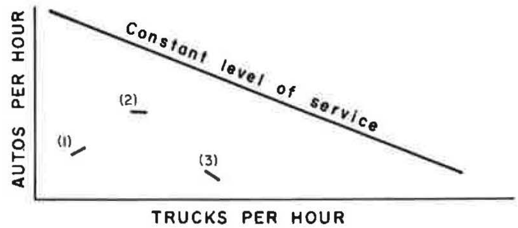


Figure 2.

and some absurdities such as a truck equivalent less than 1 on a plus 6 percent grade. These would plot on Figure 1 somewhat as shown in Figure 2.

The slopes shown at points 1, 2, and 3 of Figure 2 are only of small dimensions ($\Delta y/\Delta x$) but they were calculated for each observation and plotted (erratically, as indicated). A plus slope would indicate a truck equivalent of less than 1, and is perfectly possible to calculate from observed data, but as can be seen, this does not say anything useful about the "capacity" curve itself.

One of the problems involved relates to a clear understanding of what is meant by a "level of service," or what was formerly called "practical capacity." Basically it is a matter of probability. Figure 3 shows time-distance graphs of several vehicles in a stream of traffic on a short segment of two-lane one-way roadway.

It is seen that all the vehicles in Figure 3 enjoyed free movement except No. 8 which had to slow down and wait for No. 9 to gain an acceptable headway before it could move into L_2 and pass No. 7. There is a chance that this will happen no matter how low the volume is. The chance (probability) increases as each of the following increases:

1. Volume (number of vehicles per unit of time) of slow vehicles and of faster vehicles;
2. Difference in vehicle speeds (increased steepness of grade causing this to increase); and
3. Length of road under consideration (length of grade).

At some volume far below capacity, the chance of this happening is high enough that the traveling public decide they need a wider road. This is basically an economic decision. That is to say, the public appear to be willing to keep on building and paying for more highways as long as they encounter restrictions to free flow exceeding some amount. As interpreted by the American Association of State Highway Officials (AASHO), on rural highways this amount is not more than 30 hr/yr during which 2,000 passenger cars per hour or more pass a given point on two lanes in one direction on level grade.

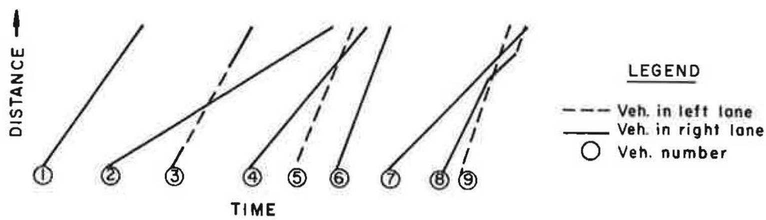


Figure 3.

DEVELOPMENT OF DESIGN CHART FOR RURAL CONDITIONS

Basically, we are concerned with finding an acceptable level of operation and relating it to length of grade, steepness of grade, and number of trucks. We assume that the acceptable flow rate is related to the frequency of passing opportunities that occur for vehicles overtaking a slow-moving vehicle. Passing opportunities are related to the three factors noted.

What do we mean by acceptable level of operation for a section of rural freeway with two lanes in one direction? In the design sense, we should expect pretty much free flow with little slowing in the left lane and very few autos caught in the right lane. We can say for sure we want no stoppages in the left lane. With our knowledge of traffic flow, what conditions bring this about?

Effects with Trucks in Right Lane Only

Since we are considering freeways, we assume good alignment and that slow-moving vehicles can be seen by an auto driver in the right lane for some distance before he would have to slow appreciably. This auto driver, on seeing the truck, would then simply merge into the left lane. He would have as much time to do this as he would normally have to merge into a freeway at any high-standard on-ramp.

We know from observation that the maximum auto rate-of-flow that can pass a single truck or series of trucks in the right lane varies from 1,800 to about 2,400 vph. And for smooth flow with no queuing upstream, the maximum passing rate is limited to 2,000 vph. As an example, if a flow rate greater than 2,000 vph is going by a truck and if a car caught behind the truck tries to merge into this stream, there is a high probability of breakdown in the left lane. Of course, there is a chance that this car would not try to pass but if there are several cars caught and the grade is of significant length, it is certain that some will attempt to pass, causing severe congestion.

When merging volumes at a good ramp are below 1,500 vph, there is little or no congestion and no ramp vehicles in queue. On a grade with trucks in the right lane, the same holds true and if total auto flow rate is less than 1,500 vph, there will be no [illegible] right lane. (If a lane is completely blocked and the public is aware of it, as they are aware of grades, 1,500 autos per hour can be handled in a two-lane to one-lane merge with little trouble or speed reduction.)

Based on this discussion, Figure 4 represents what we consider a rural design level of service with trucks in the right lane only. With no trucks, design volume is 2,000 vph (in two lanes). This is considered to be an acceptable volume giving free flow with little restrictions to desired speeds. With the introduction of a few trucks, the acceptable number of autos decreases, but fairly gradually. With just several trucks per hour, acceptable auto volume is still 1,900. Though flow around these trucks will involve some localized turbulence, this high auto flow is still considered acceptable because much of the auto traffic will not encounter the truck and the flow rate in the left lane passing the truck will take place over a very short space. In other words, the high flow rate passing the truck will be only for about ¼ mi. As the frequency of trucks increases, most traffic will encounter trucks and acceptable passing auto rates are reduced to 1,500 vph. Flow rates of 1,500 in the left lane still take place over a relatively short space, as many autos move back to the right lane on passing a truck. As truck flow rates increase even more, very few autos will go back to the right lane and most autos will be in the left lane for the entire length of grade. In this case, 1,500 vph in one lane is considered too high to maintain rural free-flow conditions and, therefore, acceptable auto flow rate decreases to 1,200 vph. (In actual practice, when trucks are this numerous, they will not stay in the right lane and acceptable auto rates are even lower.)

As long as trucks stay in the right lane, the length of grade is not a controlling factor. For example, even with a great number of trucks, if they are all in the right lane, a flow rate of 1,200 autos per hour in the left lane can be handled reasonably well for any length.

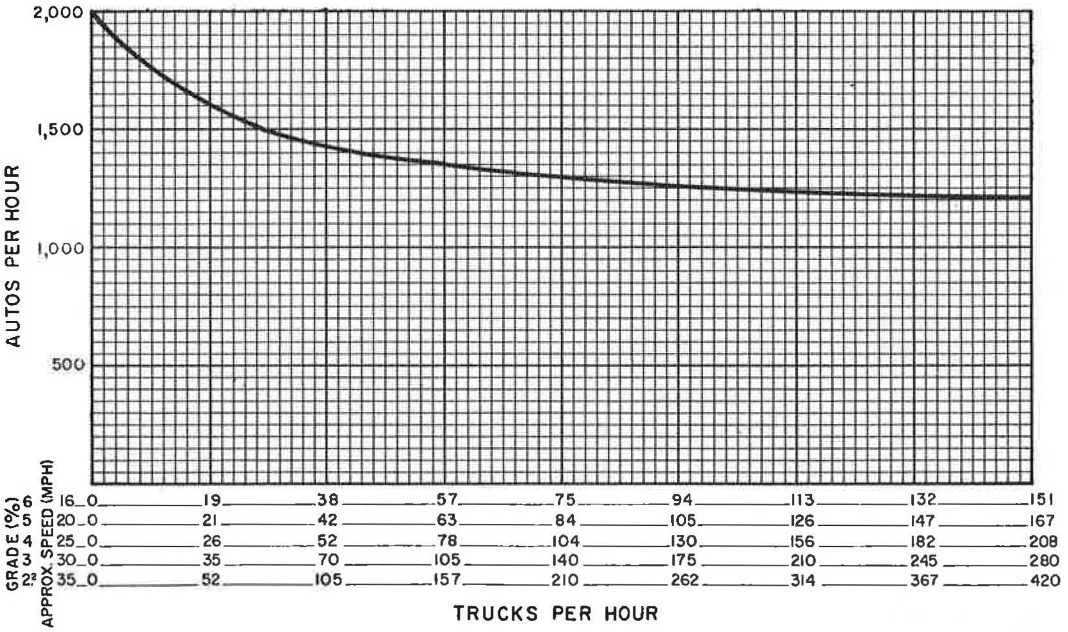


Figure 4. Effect of trucks on auto volume at given level of service if trucks remain in right lane.

The effects of different steepness of grade are indicated in Figure 4 by the different scales for each grade. Although the exact weighting of various grades was determined in connection with truck passing effects, it also represents fairly closely the degree of interference even with all trucks in the right lane. In other words, a flow rate of 1,200 autos per hour would encounter roughly the same number of slow vehicles in the right lane with 75 trucks per hour on a 6 percent grade, 84 trucks per hour on a 5 percent grade, 104 trucks per hour on a 4 percent grade, etc.

As an example of the use of this chart, assume a location with a predicted one-way peak volume of 1,300 vph including 10 percent trucks and the section contains a sustained 4 percent grade, 8,000 ft long. (A sustained grade is considered one where average truck speed is reduced to 35 mph or less.) Since the 130 trucks and 1,170 autos on a 4 percent grade plot below the capacity line, operation would generally be acceptable and a climbing lane would not be required.

Effects with Trucks in Left Lane

As the number of trucks and/or the length of grade and/or the steepness of grade increase, the chances of trucks overtaking and passing other trucks increase greatly. This becomes a much more important limitation on capacity. The number and duration of passing maneuvers can be estimated from the speed distribution of trucks, the number of trucks, and the length of grade. This number varies with the length of grade and with the square of the number of trucks and is related to the speed distribution of the trucks. (See Appendix A and B for a more detailed explanation.)

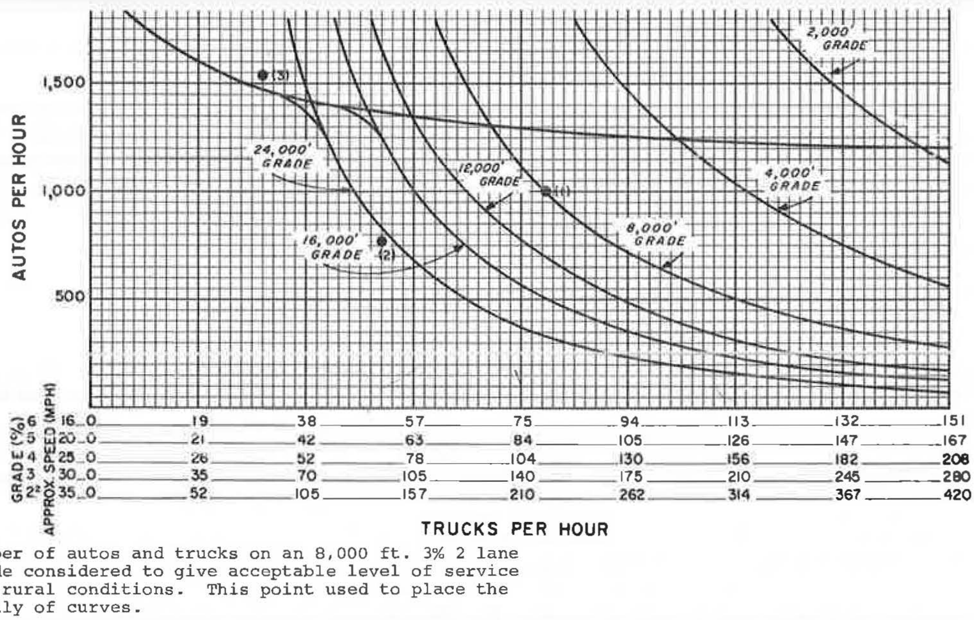
In other words for a given number and speed of trucks if the grade is twice as long, there will be twice as many passing maneuvers. For a given length of grade and speed of trucks, twice as many trucks will result in 4 times as many passings. For a given number of trucks and length of grade, the duration of passing maneuvers is related to the average speed of trucks as described in Appendix B; i.e., if trucks average 20 mph, slow trucks are in the left lane 1.55 times longer than if the trucks average 25 mph.

Figure 5 was drawn using these relationships such that any point on these curves will result in an equal amount of total delay to auto traffic, and level of operation is, therefore, assumed to be equivalent. In other words, on a given grade with 1,000 autos and 75 trucks, there will be a certain number of autos delayed by truck passing maneuvers. If there are 150 trucks, there will be 4 times as many passings, and if there are 250 autos in the stream, then the same number of autos will be delayed by trucks in the left lane. Therefore, 75 trucks and 1,000 autos are equivalent to 150 trucks and 250 autos.

The level of operation or position of the curves on the graph was determined subjectively by observing several grades. It was determined, for example, that on an 8,000-ft 3 percent grade with 150 trucks per hour, acceptable operation would result if auto flow did not exceed 1,000 per hour (Fig. 5). The curves, therefore, represent equal operation for any combination. Two other combinations of conditions subjectively determined to be acceptable also are shown and fit the curves fairly well.

This level of service is not exactly the same as with 2,000 vph on a level section without trucks. As long as there is even one truck in the left lane, there is a probability that some autos will be delayed. There is almost no probability of an auto being delayed this amount on a level section without trucks. It would be economically foolhardy to try to provide this service on a grade only several thousand feet long, and drivers do not require or expect it. The type of service provided in this chart is still high level and very few autos would be delayed. Some will be caught, but there will be no long queues. The percentage of autos delayed is not constant, though the amount of delay and absolute number of autos delayed is relatively constant.

Using the same example as previously to illustrate use of the chart, the 1,170 autos and 130 trucks would plot above the acceptable operation curve for an 8,000-ft grade and, therefore, operation would be less than desirable. A climbing lane should then be



(1)-Number of autos and trucks on an 8,000 ft. 3% 2 lane grade considered to give acceptable level of service for rural conditions. This point used to place the family of curves.

Independent subjective determination of acceptable conditions.
(2)-"Grapevine" 2 lane, 6%, 30,000', 50T, 750A.
(3)-"Waldo" 2 lane, 6%, 10,000', 30T, 1550A.

Figure 5. Effect of trucks on auto volume at given level of service when truck-passing-truck maneuvers control (superimposed on Figure 4 curve).

provided, although the chart also indicates that if 4,000 ft of passing lane were provided, operation on the remaining 4,000-ft section would be acceptable.

In effect, then, for a given combination of truck and grade conditions, we evaluate them using Figures 4 and 5, taking the lower of the two acceptable volume levels. Therefore, Figure 5 combines the two graphs. For convenience and practicality in constructing the final design chart, we round the corners where the truck passing control intersects nontruck passing control. Figure 6 shows the final chart.

USE OF DESIGN CHART FOR RURAL CONDITIONS

In using this chart, it should be recognized that the curves represent average conditions and there can be a great amount of variability. In other words, if trucks on a particular grade are faster than the normal speed distribution used in developing the chart, operation will not be as critical for a certain combination of autos, trucks and grade. Conversely, if the trucks are slower than normal, operation will be more critical.

Volumes indicated on the chart are full hour volumes. Therefore, this also introduces variability since volume rates can fluctuate significantly within an hour. For example, if an hour flow is 1,500 veh, there is a chance that the flow rate, when there are several trucks on the grade, could be 1,800 vph.

However, the curves in general take care of normal variability and if the conditions of the chart are satisfied, only in rare cases would severe congestion result. Actual capacity would never be exceeded.

The design chart is for two lanes only. When more lanes are available, we assume that trucks will still only use the right two lanes or could be required to by law, so that the additional lanes could handle, based on rural conditions, 1,000 autos per hour per lane. Therefore, assuming conditions of a grade were such that acceptable hour flow were 1,200 veh including 100 trucks in the two lanes, if three lanes were available, acceptable flow would be 2,200 veh including 100 trucks. If four lanes were available, the figure would be 3,200 with 100 trucks.

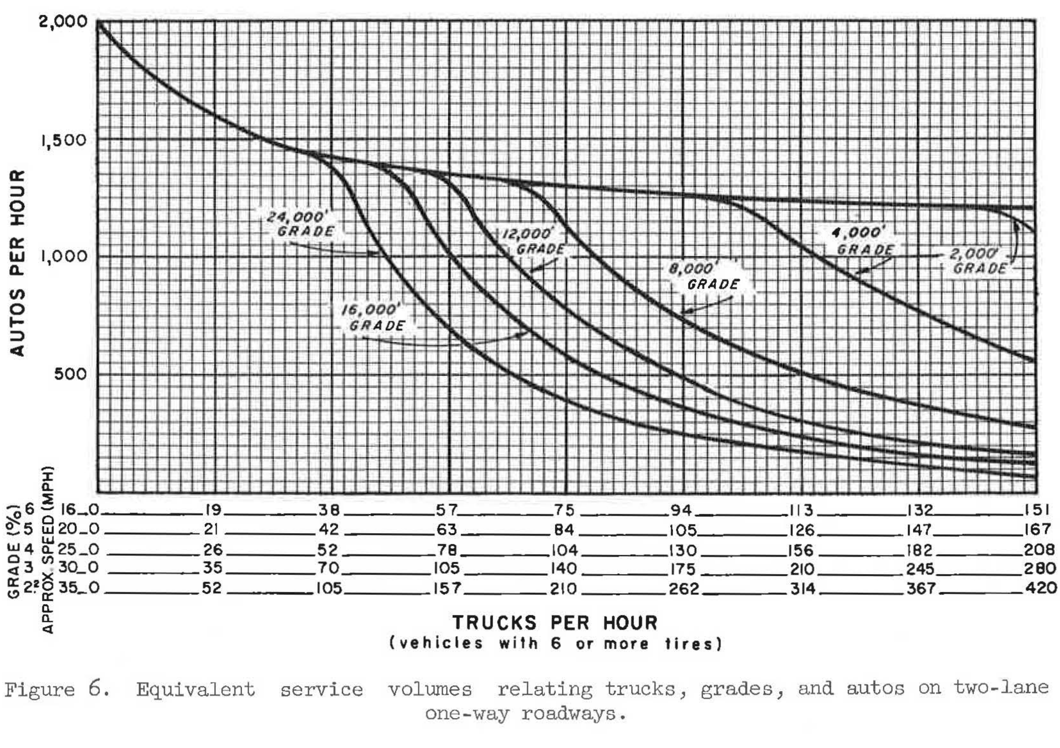


Figure 6. Equivalent service volumes relating trucks, grades, and autos on two-lane one-way roadways.

On grades less than 2 percent where truck speeds are not reduced to 35 mph, effects of trucks are generally based on the greater space occupied by trucks and slightly lower speeds maintained. In this case, reductions in free-flow volume of 2,000 autos per hour without trucks can be based on an equivalency of one truck equaling two autos.

One other point should be noted concerning the lengths of grade indicated in the chart. They represent sustained grades which are grades long enough to reduce average truck speed to 35 mph or less. The vertical curve or length of grade necessary to slow the trucks from 50 to 35 mph does not have to be considered. The length indicated on the chart is the length the trucks are at 35 mph or less. It takes about 2,000 ft for trucks to slow to 35 and then accelerate from 35 to 50 mph on a 6 percent grade compared to about 3,400 ft on a 3 percent grade (2). If we want to be more precise and were evaluating a 6 percent grade against a 3 percent grade, then the 8,000-ft grade curve, for example, would actually represent about 10,000 ft of 6 percent grade but 11,400 ft of 3 percent grade.

In summary, if predicted traffic conditions and grade conditions plot above the acceptable volume curves, an additional lane should be provided if flow on the grade is to be maintained at a quality roughly equivalent to the same flow on level or rolling terrain.

The chart, however, indicates that for some conditions, a climbing lane would not have to be added the full length of a grade to bring operating conditions to an acceptable level. For example, assume a 24,000-ft 6 percent grade with predicted traffic of 1,000 autos and 75 trucks. This plots well above the curve of acceptable operation for a 24,000-ft grade, and a climbing lane should be provided. But if only 16,000 ft of climbing lane were provided leaving 8,000 ft of two-lane 6 percent grade, then flow conditions would be acceptable since the traffic and grade conditions plot below the acceptable operation curve for an 8,000-ft grade. The physical picture this represents is that with 24,000 ft of two-lane grade, trucks will be in the left lane frequently enough to cause numerous delays to the auto stream. If the length of two-lane grade is only 8,000 ft, there is much less chance that trucks will catch up with each other; consequently, there will be fewer trucks in the left lane and infrequent delays to the auto

In general, if a truck lane is not provided the whole length of a grade, it should be used on the upper part of the grade. This will not require merging of slow trucks on the grade. It is important that a climbing lane be carried far enough so that trucks can accelerate to about 40 mph before entering the main lanes. Also, trucks that do not decelerate as slowly as normal do not need a truck lane at the beginning of the grade.

However, it is much safer to provide a truck lane the whole length. In the example, 16,000 ft of climbing lane increased auto design volume by 700. But 24,000 ft of climbing lane increases auto design volume by at least 1,000 and virtually insures that the grade will have as much capacity as the approach section.

URBAN FLOW CONDITIONS

The discussion and design chart presented deal strictly with rural conditions where most trips are long and essentially free-flow conditions are desired. Although we have not yet developed charts in this detail for urban conditions, some discussion is in order. In any section of freeway, the most important thing is to insure that enough lanes are provided so that demand will not exceed the capacity of the section. Capacity of a rural freeway is no different from that of an urban freeway.

Sustained grades of any slope (defined as a grade long enough and steep enough to reduce truck speeds to 35 mph or less) greatly reduce capacity and since most urban freeways will operate near capacity, as a general rule an added lane should be provided on all sustained grades in urban areas in which design hour volumes are within 1,000 vph of normal capacity on a level section.

For example, on a four-lane section (one-way) with estimated peak hour traffic of 5,000 veh, an extra lane should be provided on any sustained grade. Similarly, an extra lane should be provided for 3,500 veh on three lanes and 2,000 vehicles on two lanes. Even if truck volumes are relatively light, these lanes should still be provided.

Design hour volumes of 1,500 veh/lane cannot be handled without severe congestion on any sustained grade when there is a truck on this grade at 35 mph or less. And once a stoppage occurs, conditions become even worse because trucks which ordinarily would traverse the grade at a reasonable speed cannot, once they have been stopped, accelerate to this speed.

Several grades have been observed at maximum capacity. One four-lane one-way 5.5 percent grade, only 4,600 ft long, with 150 trucks and buses, had a maximum capacity of about 6,000 autos per hour. This is about 1,500 vph less than capacity of a level four-lane section. Another grade, 10,000 ft long, three lanes wide at 6 percent, with only 100 trucks and buses, had a maximum capacity of about 3,800 autos, about 1,700 to 1,800 vph less than capacity on a level three-lane section.

REFERENCES

1. U.S. Bureau of Public Roads. Highway Capacity Manual. U.S. Govt. Print. Office, 1950.
2. Calif. Div. of Highways Traffic Bull. No. 2.

Appendix A

DATA ON TRUCK CHARACTERISTICS AND EFFECTS OBTAINED AT CORDELIA GRADE (I-80)

To obtain more information on the behavior and effect of trucks, a study was made of operation on a grade located about midway between Sacramento and San Francisco

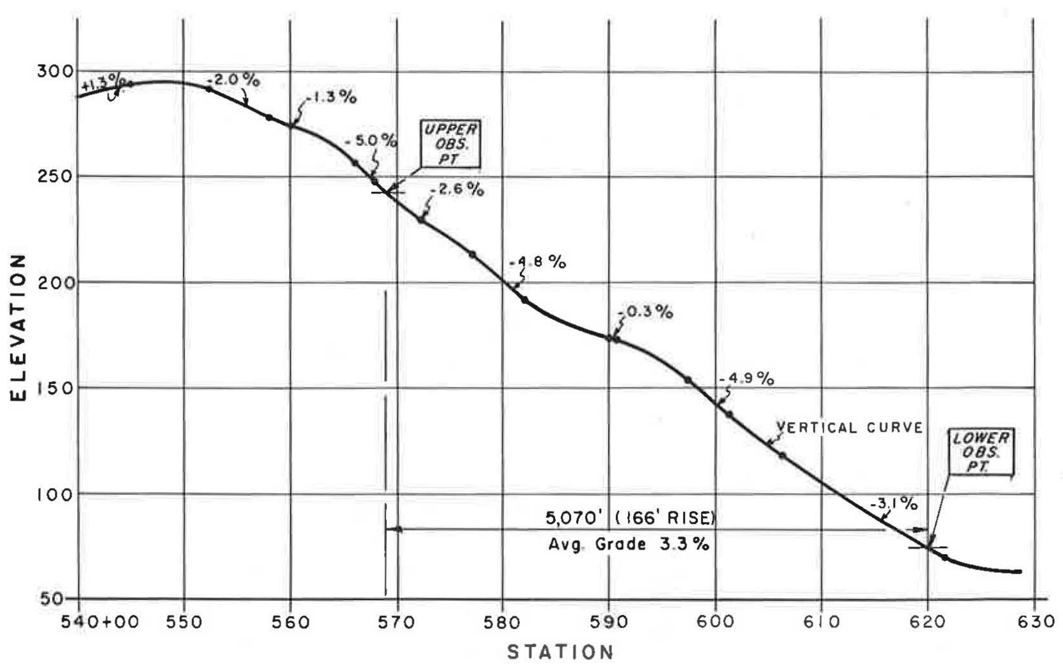


Figure 7. Cordelia grade, westbound lanes, I-80.

on the two westbound lanes of I-80. The location is rural and the nature of the traffic tends to make it high speed. Through vehicles have been on 50 mi of continuous freeway and expressway, although they passed through several construction zones. Figure 7 shows the profile of the grade and the observation points. The 1962 two-way ADT at this location was 26,000 veh with a seasonal peak of 33,500.

Data were gathered by time-lapse photography taken simultaneously at two points: (a) near the bottom of the grade, and (b) 5,000 ft up the grade. Photography was in color and a clock was framed in the picture at each camera. Thus, it was relatively simple to match vehicles and determine elapsed time between the two observation points. This was done for 100 percent of the trucks and about 60 percent of the autos. Matching of all trucks also made it possible to determine truck-passing-truck maneuvers. All analysis was made from the films.

Speeds shown or noted were always based on travel time between the two observation points because we believed effects on auto travel could best be determined in this manner. Spot speeds would not reflect delays at points not observed.

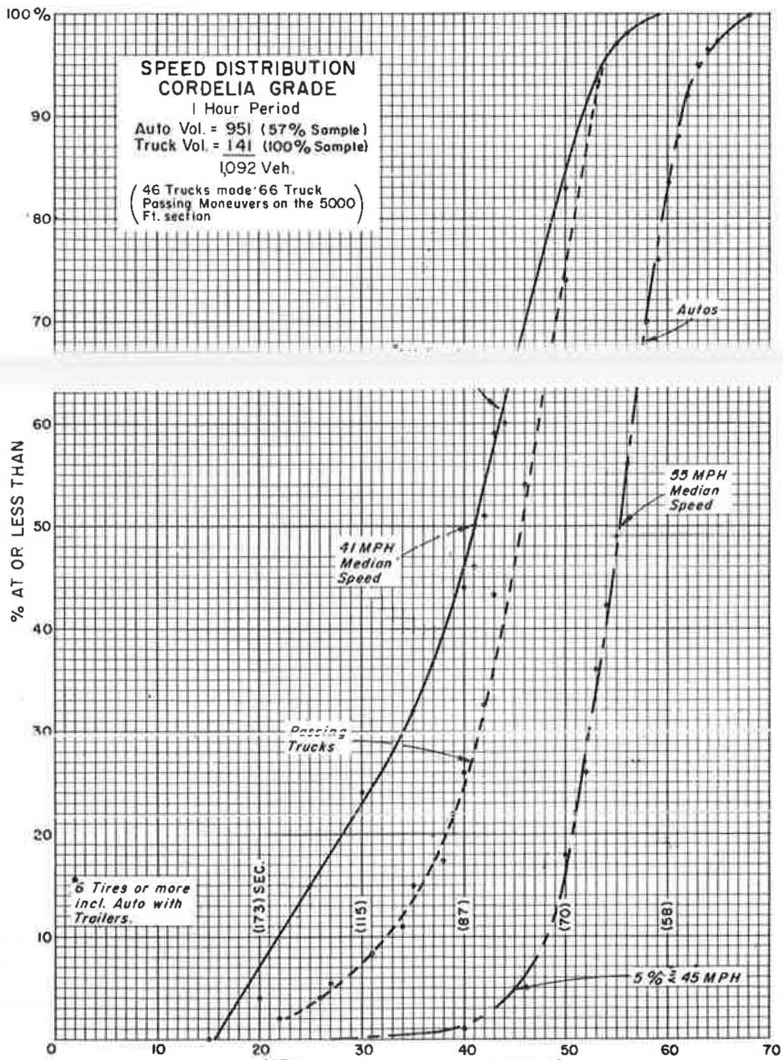


Figure 8.

The study was made in August 1963 for a 2-hr period. For the period studied, 5-min total flow rate varied from 875 to 1,580 vph. Truck rates (including buses and autos with trailers) varied from 85 to 225 trucks per hour.

Figure 8 shows that for a complete hour with 140 trucks (median speed of 41 mph with 66 truck-passing-truck maneuvers), 950 autos traveled through the section at a median speed of 55 mph, and 5 percent were at 45 mph or less. Figure 9 shows that for a separate 50-min period with a truck flow rate of 140 trucks per hour (median speed of 35 mph with 70 truck-passing-truck maneuvers per hour), 1,150 autos per hour traveled through the section at a median speed of 51 mph; 15 percent traveled at 45 mph or less.

For a 5-min period (Fig. 10) with 130 trucks per hour and 1,030 autos per hour, median auto speed was 57 mph with none less than 48 mph. There was only one truck-passing-truck maneuver during the 5-min period and average truck speed was 46 mph. Therefore, at this volume of trucks and autos, there was little or no delay to autos.

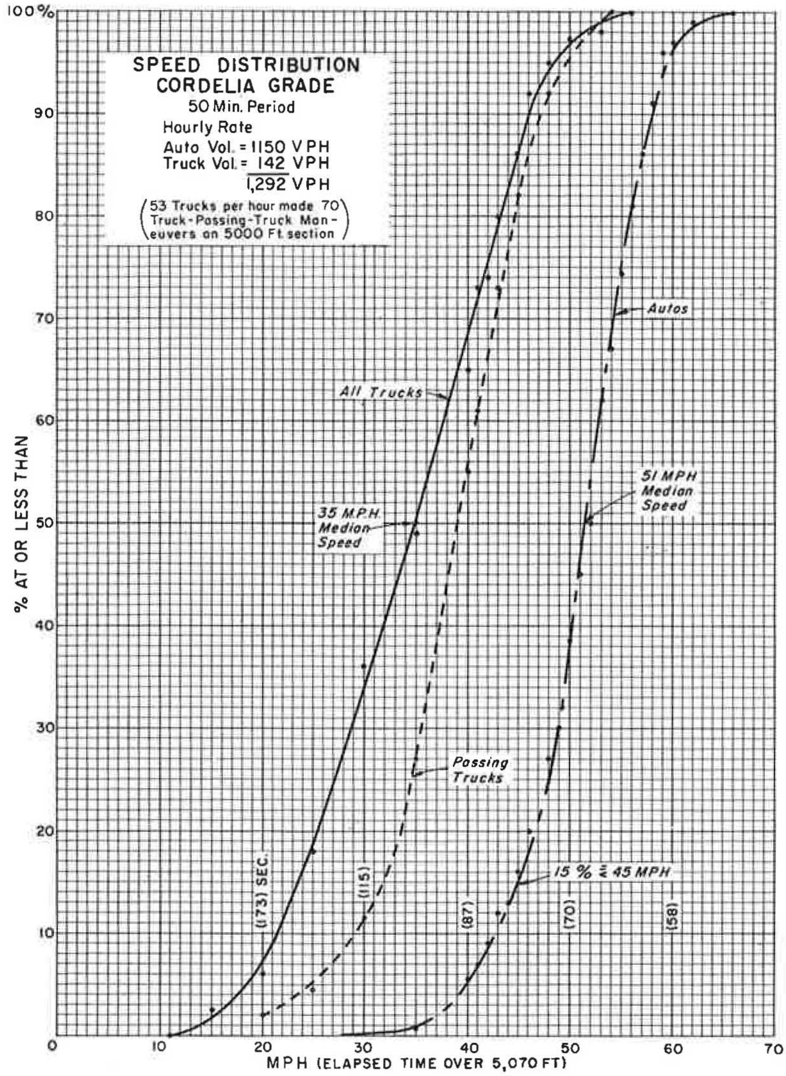


Figure 9.

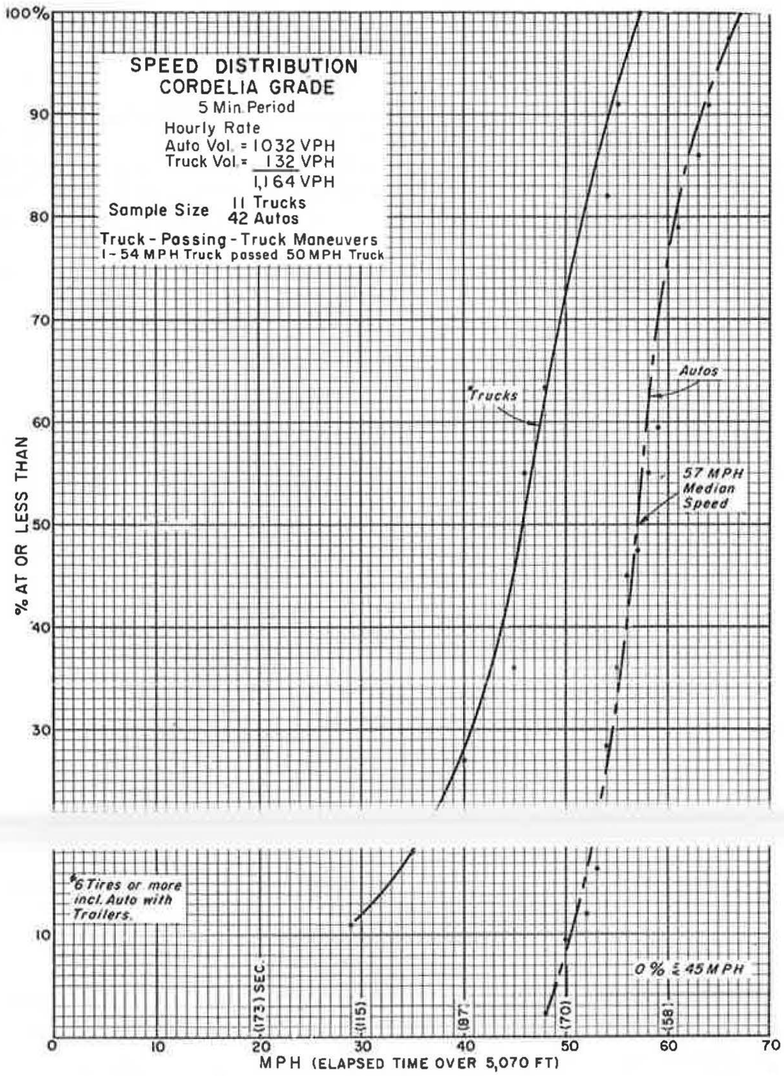


Figure 10.

Figure 11 shows that the average free-flow auto speed on all California freeways in 1962 was 56.7 mph.

In another 5-min period (Fig. 12) with 190 trucks per hour and 1,380 autos, median auto speed was again 57 mph. This also reflects little delay to autos in spite of a large number of trucks with a median speed of 34 mph and numerous truck-passing-truck maneuvers. However, on examining the truck passing maneuvers, it can be seen that the passing trucks were reasonably fast and the passed trucks were slow. Under these conditions, the duration of a passing maneuver tends to be minimized. This figure points out that, despite a large number of trucks, as long as the left lane is not seriously blocked there is little delay to autos when the auto flow rate is 1,400 vph.

During another 5-min period (Fig. 13) with 170 trucks per hour and 1,420 autos per hour, median auto speed was only 46 mph. Median truck speed was 32 mph. Although truck passings were not as numerous as in the period illustrated in Figure 12, the passing trucks were considerably slower. The effect with 1,420 autos per hour as reflected by the auto speeds is severe.

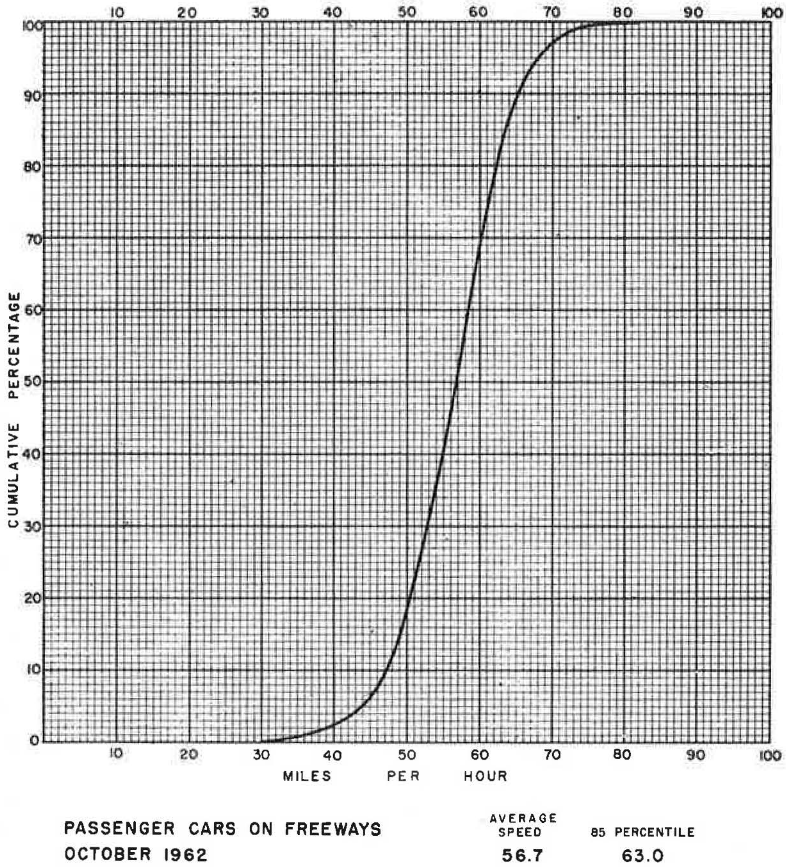


Figure 11. Free-flow speed curves for all California freeways, prepared by California Division of Highways, Traffic Department.

We can assume that the desired speed is about 57 mph or about 60 sec to traverse the study section. In this instance, the average car took 75 sec or a delay of 15 sec per car. Twenty-five percent of the cars took 87 sec or longer. These are serious delays and reflect undesirable operation with large speed differentials between autos even in the left lane. Actually when an average auto speed is, for example, 40 mph over the 5,000-ft section, the spot speed on the section could vary from zero to 60 mph. There were, in fact, complete stoppages in the left lane at times.

It can be seen that operation at these auto volume levels is not as dependent on total number of trucks as it is on the number and type in the left lane.

Figure 14 plots the number of truck-passing-truck maneuvers against total number of trucks. Theory of probability indicates that the number of trucks catching up with other trucks varies with the square of total number of trucks. Figure 14 indicates that passing maneuvers follow this same pattern. Other studies have shown this same trend.

In summary, study of this grade indicates the validity of the previous discussion that as long as there are no trucks (or no slow ones) in the left lane of a two-lane section, there will be little or no delay as long as auto volume is 1,500 vph or less. However, with an appreciable number of trucks, there will be enough truck-passing-truck maneuvers to lower auto capacity below 1,500 vph.

On this particular grade with an hour volume of 140 trucks, buses and trailers and 1,150 autos (Fig. 9), operation is less than acceptable for design purposes. With these

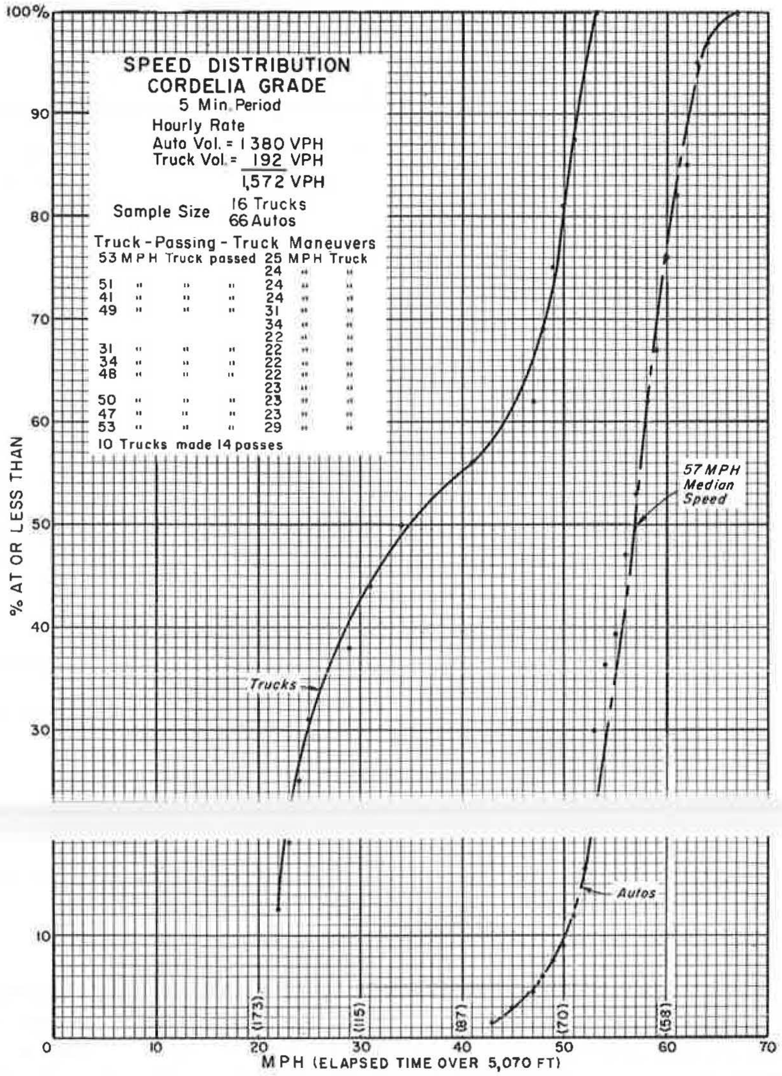


Figure 12.

conditions, median auto speed (over a 5,000-ft section of the grade) was 51 mph with 15 percent at or less than 45 mph. At times there were actual stoppages.

For the same number of trucks and 950 autos (Fig. 8), median auto speed was 55 mph, slightly less than the unrestricted speed of 57 mph. Only 5 percent were at or less than 45 mph. In this case this would probably be considered acceptable for design purposes under rural conditions.

Appendix B

DEVELOPMENT OF DESIGN CAPACITY CHART

This section describes the development of the relationships between truck frequency, length and steepness of grade shown in Figure 5 indicating limits of free-flow volumes for two lanes in one direction when truck-passing-truck maneuvers are the control.

To illustrate how truck passing maneuvers affect operation, we first stipulate that as soon as a truck catches another he will pass. (Our studies indicate this is generally true regardless of auto volumes.) We assume that a truck going 19 mph passing one going 10 mph would encroach on the left lane long enough to gain about 300 ft on the passed truck. This would take about 26 sec and he would travel about 680 ft in the left lane. (At this point it might be well to point out that though absolute numbers and arbitrary rules, such as gaining 300 ft, are used, they are used only in the sense of developing relationships. The numbers are not used to determine amounts of delay,

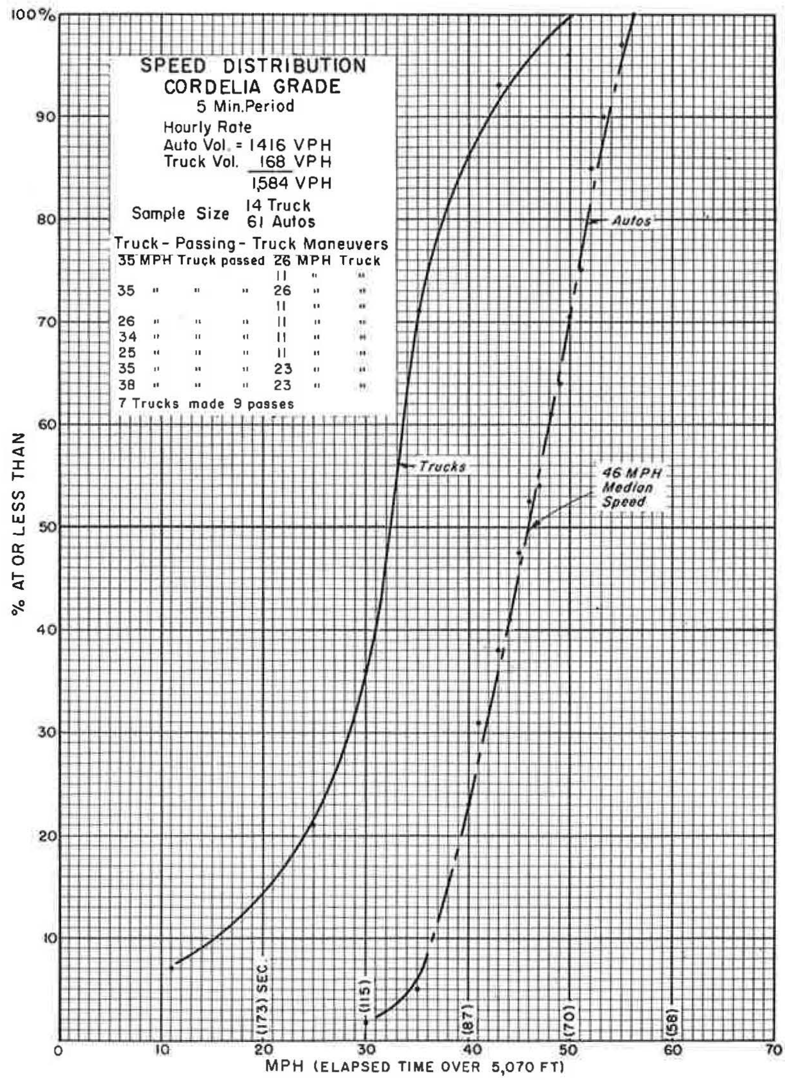


Figure 13.

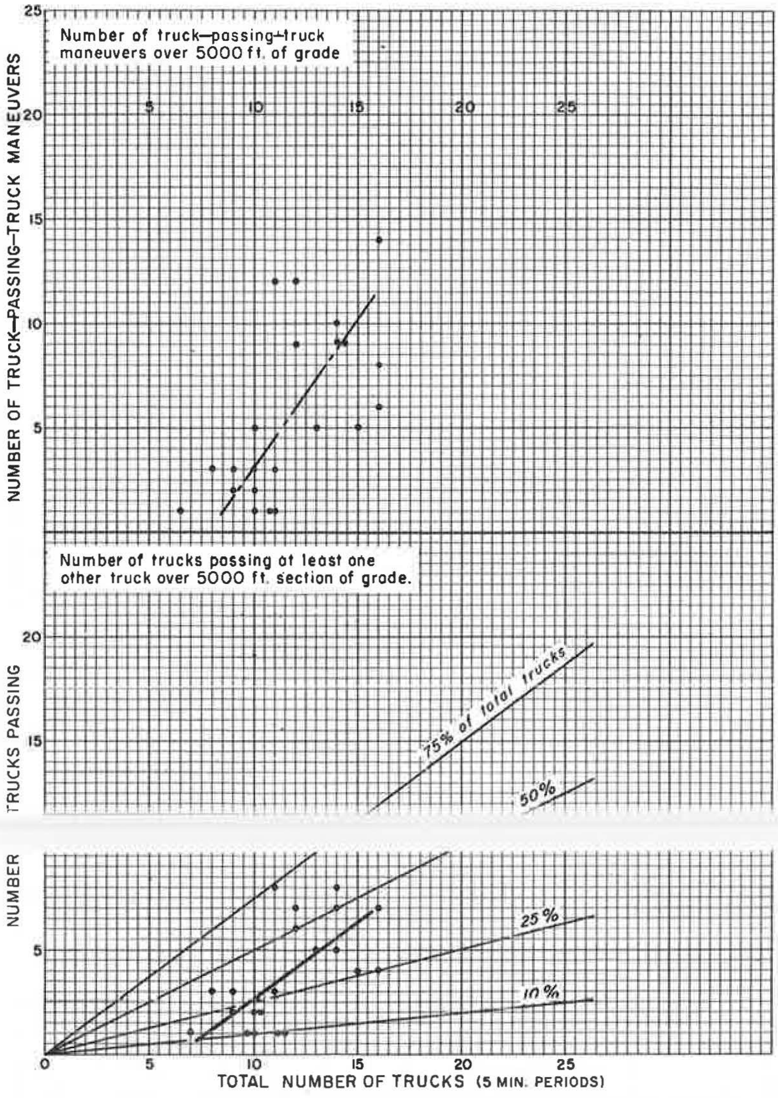


Figure 14.

etc., but only to obtain a relative effect. Changing the values would not change the relative effects significantly.) It would take a car about 9 sec to travel the same distance (680 ft). Thus, when a truck started the passing maneuver, a car would have to be 17 sec back in order to just catch the truck as he was getting back into the right lane. Five seconds are added so that the car will still be behind the truck a certain distance when the truck moves back to the right lane so that the auto will not slow in anticipation that the truck will not clear the left lane. This means that if no slowdown is to occur in the left lane, no car can arrive within 22 sec after the truck starts his passing maneuver. If the auto flow rate is 1,500 per hour, there is zero probability that no car will arrive at a point in a 22-sec period. There is a 42 percent chance that 10 cars or more will arrive. It is obvious then that if free flow is desired in the left lane, acceptable flow rates will have to be less than 1,500 vph if there are many truck-passing-truck maneuvers.

Knowing the speed distribution of trucks, number of trucks, and length of grade, the number of expected passing can be determined. Using this relationship, curves in Figure 5 were developed such that total delay for a given number of autos at a point on any curve will be equal. This equivalence is in relative terms and a particular amount of delay cannot be inferred from the curves.

Truck speed distributions obtained for the development of the truck acceleration curves in Traffic Bulletin No. 2 (2) were used as basic data. Figure 15 reproduces from this bulletin the average crawl speed on various grades of loaded as well as unloaded trucks.

Truck speeds were grouped as shown in Figure 16. For example, if the average speed of all trucks on a particular grade is 20 mph, 30 percent will be 15 mph or less. These groupings are used to calculate the relative number of truck passing maneuvers for different grades. These are given in Table 1 and are based strictly on the probability of trucks at various speeds overtaking trucks at the lesser speeds indicated. This was done by first assuming 100 trucks per hour at an average speed of 20 mph on a section of grade, 2,000 ft long. Thirty of the trucks will have an average speed of 10 mph (5 to 15 mph) and 22 an average of 18 mph (16 to 20 mph). It would take a 10-mph truck 137 sec to traverse the 2,000 ft. It would take an 18-mph truck 76 sec to traverse the same distance. Therefore, if the 18-mph truck arrives at the beginning of the 2,000-ft section within 137 - 76 or 61 sec after the 10-mph truck arrives, it will catch up. Since there are 22 trucks at 18 mph on the average they will cross a given point every 164 sec. Therefore, when a 10-mph truck passes the start of the section on the average (over many trials), 61/164 or 0.37 18-mph trucks will arrive soon enough to catch the 10-mph truck within 2,000 ft. Since there are 30 10-mph trucks, on the average 0.37 × 30 or 11 18-mph trucks will catch up with the 10-mph trucks at some point on the 2,000-ft section of grade during the hour.

It follows that if the section is twice as long, there will be twice as many 18-mph trucks catching 10-mph trucks. If the section is 4,000 ft long, the 10-mph truck takes 274 sec to traverse the grade and the 18-mph truck 152 sec. Therefore, any 18-mph truck arriving within 122 sec after the 10-mph truck will catch up. With 22 trucks per hour at 18 mph, each time a 10-mph truck arrives at the beginning of the section, on the average 0.74 18-mph will arrive. If there are 30 10-mph trucks, 22 18-mph trucks will catch 10-mph trucks on the 4,000-ft section or twice the number on the 2,000-ft

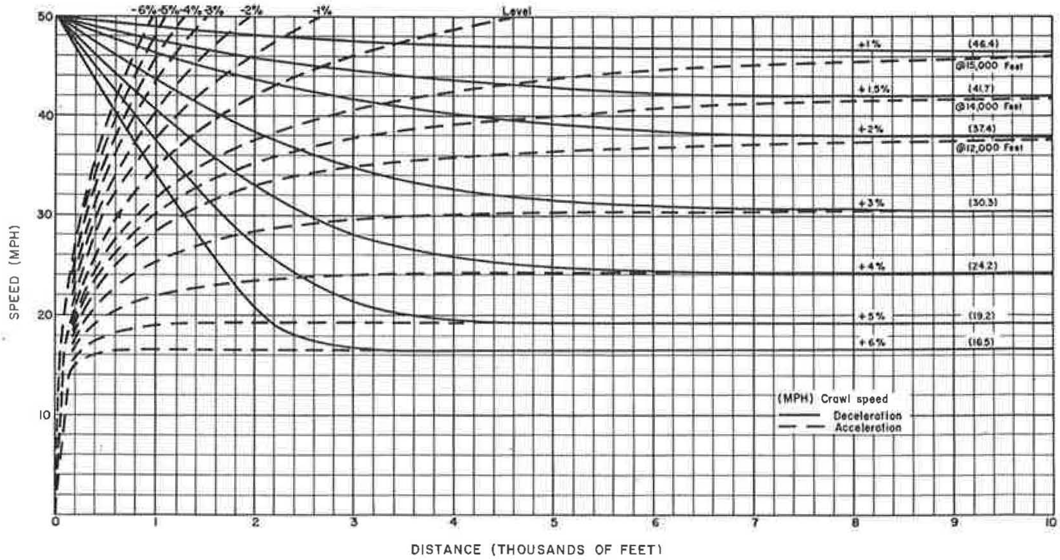
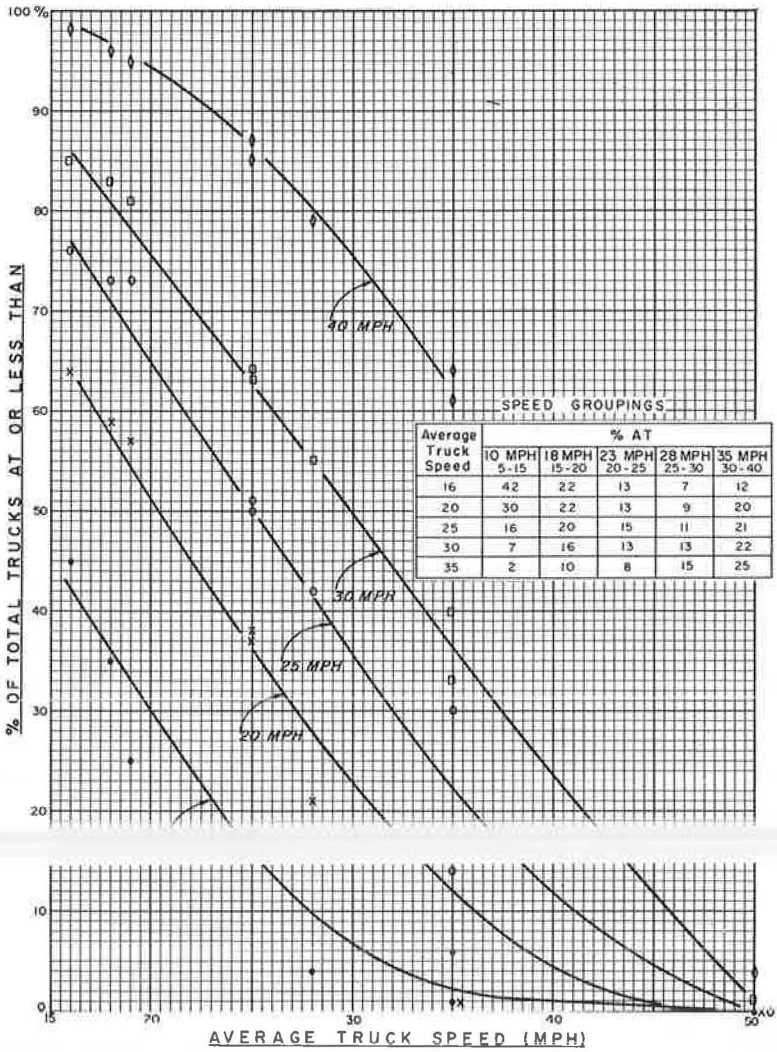


Figure 15. Truck acceleration and deceleration curves (2).



Average Truck Speed	% AT				
	10 MPH 5-15	18 MPH 15-20	23 MPH 20-25	28 MPH 25-30	35 MPH 30-40
16	42	22	13	7	12
20	30	22	13	9	20
25	16	20	15	11	21
30	7	16	13	13	22
35	2	10	8	15	25

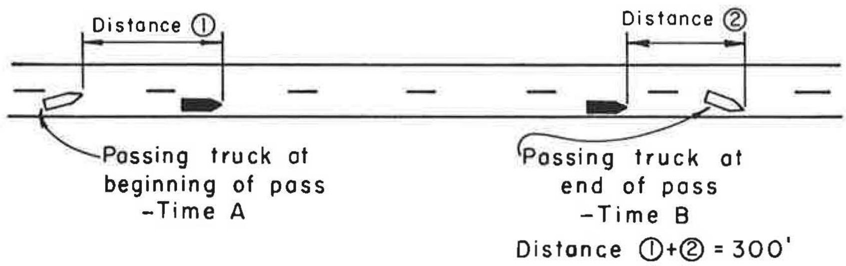
Figure 16. Truck speed distributions.

section. It also follows that if there are twice as many trucks there will be 4 times as many overtakings.

Development of Relative Scale for Different Truck Speeds

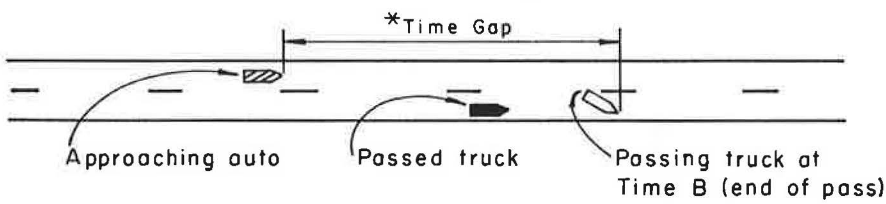
In calculating the number of seconds an auto must be behind a truck when the truck starts his passing maneuver so that the auto will not be delayed by the passing maneuver, the following procedure is used:

1. A basic stipulation is that the passing truck on the average will encroach on the left lane long enough to gain about 300 ft on the passed truck:



2. When the passing truck is completing his pass any approaching auto must still be a certain number of seconds behind the truck so that he will not have braked appreciably in anticipation that the truck will not clear the left lane (reaction time):

*Assume that at time B approaching
auto, in order not to be delayed, must be -
5 seconds behind truck if truck is going 20mph
4 " " " " " " " 25mph
3 " " " " " " " 30mph
2 " " " " " " " 35mph



3. Using this criteria, Table 2 was prepared which gives the necessary time gap an auto must be behind various speed trucks when the truck starts its pass, if the auto is not to be delayed. The necessary gap also indicates in a relative way the amount of

TABLE 1

EXPECTED NUMBER OF TRUCKS OVERTAKING OTHER TRUCKS[a]

Type of Maneuver		Number Overtaking at Avg. Truck Speed				
		16 Mph	20 Mph	25 Mph	30 Mph	35 Mph
18-mph tr. catching	10-mph tr.	16	11	5	2	0.3
23-mph tr. catching	10-mph tr.	12	8	6	2	0.3
28-mph tr. catching	10-mph tr.	7	7	4	2	0.7
35-mph tr. catching	10-mph tr.	14	16	9	4	1.3
Total		49	42	24	10	3.0
23-mph tr. catching	18-mph tr.	1	1	1	1	0.5
28-mph tr. catching	18-mph tr.	1	1.5	2	1.5	1.0
35-mph tr. catching	18-mph tr.	3	4.5	4	3.5	2.5
Total		5	7.0	7	6.0	4.0
28-mph tr. catching	23-mph tr.	0.5	0.4	0.6	0.6	0.4
35-mph tr. catching	23-mph tr.	1	1.5	1.8	1.6	1.1
Total		1.5	1.9	2.4	2.2	1.5
35-mph tr. catching	28-mph tr.	0.2	0.4	0.5	0.6	0.8
Overall total		56	51	34	19	9

[a]Per 100 trucks per hour on 2,000-ft section of grade, based on grouping shown in Figure 16; number of overtakings proportional to length of grade and square of number of trucks.

TABLE 2

Type of Maneuver		Time for Pass. Truck to Gain 300 Ft (sec)	Dist. Traveled (ft)	Time for 55-Mph Auto to Travel Dist. (sec)	Reac. Time (sec)	Necessary Gap (sec)[a]
18-mph tr. passing	10-mph tr.	25.6	680	8.5	5	22
23-mph tr. passing	10-mph tr.	15.8	530	6.5	4	13
23-mph tr. passing	18-mph tr.	41.1	1,380	17.5	4	28
28-mph tr. passing	10-mph tr.	11.4	470	6.0	3	8
28-mph tr. passing	18-mph tr.	20.4	840	10.5	3	13
28-mph tr. passing	23-mph tr.	40.5	1,670	21.0	3	23
35-mph tr. passing	10-mph tr.	8.2	420	5.0	2	5
35-mph tr. passing	18-mph tr.	12.0	620	8.0	2	6
35-mph tr. passing	23-mph tr.	17.0	880	11.0	2	8
35-mph tr. passing	28-mph tr.	28.4	1,510	19.0	2	11

[a]Time for passing truck to gain 300 ft minus time for 55-mph to travel distance plus reaction time.

total delay time. For example, if an 18-mph truck passed a 10-mph truck and pulled out directly in front of an auto, that auto would be delayed approximately 22 sec. If a 28-mph truck passed a 23-mph truck directly in front of an auto, that auto would be delayed about 23 sec or about the same total amount. What this table does not indicate is that even though for this example the delay is the same, subjectively perhaps the delay behind the 10-mph truck is worse than delay behind the 28-mph truck.

4. From Table 1 and Step 3, Table 3 is derived which indicates the total time that no auto may arrive so that no delay will occur for a case with 100 trucks per hour and a 2,000-ft length. Although this table is not used directly the sums are used to weight relative effects of different grades when other conditions are equal.

This essentially relates the amount of delay which can be attributed to 100 trucks at various average speeds due to passing maneuvers. Since the number of passings is proportional to the square of the total number of trucks, if 100 trucks at 20 mph have

$$\left(\frac{x}{100}\right)^2 \times 716 = 582 = 100\sqrt{0.81} = \text{90 trucks at 16 mph} = \text{100 trucks at 20 mph}$$

124 trucks at 25 mph = 100 trucks at 20 mph
167 trucks at 30 mph = 100 trucks at 20 mph
250 trucks at 35 mph = 100 trucks at 20 mph

This gives the relative truck scale used at the bottom of Figure 6 in the text.

In summary, when truck passings control, equivalent service volume is related to the square of the number of trucks, proportional to length of grade, and is related to speeds of trucks as follows: 16 mph = 0.9; 20 mph = 1.00; 25 mph = 1.24; 30 mph = 1.67; 35 mph = 2.50. Therefore, if we say from subjective observation that on an 8,000-ft grade with 150 trucks per hour with an average speed of 30 mph, a volume of 1,000 autos per hour will result in good operation without undue delays to autos. We

TABLE 3
TOTAL TIME WHEN NO AUTO MAY ARRIVE[a]

Passing Maneuver		At Average Truck Speed														
		16 Mph			20 Mph			25 Mph			30 Mph			35 Mph		
		No. Passings	Requir. Gap	Time (sec)	No. Passings	Requir. Gap	Time (sec)	No. Passings	Requir. Gap	Time (sec)	No. Passings	Requir. Gap	Time (sec)	No. Passings	Requir. Gap	Time (sec)
18-mph tr. passing	10-mph tr.	16	22	352	11	22	242	5	22	110	2	22	44	0.3	22	7
23-mph tr. passing	10-mph tr.	12	13	156	8	13	104	6	13	78	2	13	26	0.3	13	4
23-mph tr. passing	18-mph tr.	1	28	28	1	28	28	1	28	28	1	28	28	0.5	28	14
28-mph tr. passing	10-mph tr.	7	8	56	7	8	56	4	8	32	2	8	16	0.7	8	6
28-mph tr. passing	18-mph tr.	1	13	13	1.5	13	20	2	13	26	1.5	13	20	1.0	13	13
28-mph tr. passing	23-mph tr.	0.5	23	13	0.4	23	9	0.6	23	13	0.6	23	13	0.4	23	9
35-mph tr. passing	10-mph tr.	14	5	70	16	5	80	9	5	45	4	5	20	1.3	5	7
35-mph tr. passing	18-mph tr.	3	6	18	4.5	6	27	4	6	24	3.5	6	21	2.5	6	15
35-mph tr. passing	23-mph tr.	1	8	8	1.5	8	12	1.8	8	14	1.6	8	13	1.1	8	9
35-mph tr. passing	28-mph tr.	0.2	11	2	0.4	11	4	0.5	11	6	0.6	11	7	0.8	11	9
Total				716			582			376			208			93

[a]So that no delay will occur for case with 100 trucks per hour and 2,000-ft length; total time = No. passings x required gap for each pass.

can then relate any combination of trucks, grade, and speed to this same level of operation or same amount of delay.

Example: Given conditions of 100 trucks per hour, a 12,000-ft grade, and speed of 25 mph, what is the auto capacity?

$$\text{Equiv. No. of trucks at 30 mph} = \frac{1.67}{1.24} \times 100 = 135$$

$$\text{Adjustment for No.} = \left(\frac{150}{135}\right)^2 \times 1,000 = 1,235 \text{ autos}$$

$$\text{Adjustment for length of grade} = \frac{8,000}{12,000} \times 1,235 = 825 \text{ autos per hour}$$

Study of Operational Characteristics of Left-Hand Entrance and Exit Ramps on Urban Freeways

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The paper is essentially a companion to an earlier report dealing with left-hand exit ramps for freeways and is divided into three main sections: (a) a study of the general operating characteristics of left- and right-hand entrance ramps on urban freeways; (b) an analysis of traffic behavior along a 2-mi section of urban freeway containing two internal diamond interchanges; and (c) a comparative study of the reported accident rates at a sample of right- and left-hand entrance and exit ramps on urban freeways in the Chicago area.

Brief descriptions are given of study locations and study techniques, together with a discussion of major results. Conclusions are drawn concerning the operational efficiency, relative safety and general suitability of left-hand entrance and exit ramps for urban freeways under the type of site conditions existing in the Chicago area.

•THIS PAPER is essentially a companion to an earlier report by Berry, Ross and [illegible] ramps on urban freeways. The present paper develops some of those earlier analyses somewhat further, discusses a complementary study of left-hand entrance ramps and presents the results of a study of reported accident rates at a sample of left- and right-hand entrance and exit ramps in the Chicago area.

OPERATIONAL STUDIES—LEFT- AND RIGHT-HAND ENTRANCE RAMPS

The operational problems posed by a left-hand entrance ramp on a high-volume urban freeway differ considerably from those posed by a left-hand exit ramp. In the case of a left-hand exit, the major operational problems are generally associated with increases in the incidence of weaving and hazardous maneuvers immediately upstream of the ramp nose (1). In the case of a left-hand entrance, however, the major problems arise from the fact that ramp vehicles are forced to merge with through traffic traveling in the high-speed, high-volume left lane of the freeway, rather than in the lower speed, lower volume right-hand lane. The studies described in this first section of the paper assess some of the operational problems which are likely to occur at left-hand entrance ramps by comparing the operation of a series of left-hand ramps located in the Chicago area with an equivalent sample of "typical" right-hand entrance ramps.

Field studies were conducted at four left-hand and two right-hand entrance ramps on the Eisenhower and Kennedy Expressways in Chicago (Table 1). The studies encompassed a total of 24 hr of observations, during typical morning and afternoon weekday traffic conditions. They included a period of over 2 hr in the early morning, during which severe congestion backed up into the vicinity of one of the left-hand ramps

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Paper sponsored by Committee on Freeway Operations.

TABLE 1

CHARACTERISTICS OF ENTRANCE RAMP STUDY LOCATIONS

Location of Entrance Ramp	Freeway Characteristics		Ramp Characteristics			Volume Characteristics (AWDT)	
	No. Lanes	Alignment	Accel. Lane Length (ft)	Width (ft)	Grade (%)	Freeway Vol. Upstream of Nose	Ramp Vol. At Nose
Left-hand entrance ramps							
Harlem Ave. EB, Eisenhower Expwy.	3	Depressed fwy., tangent and level.	1,075 (parallel)	16	-3	53,300	10,700
Harlem Ave. WB, Eisenhower Expwy.	3	Depressed fwy., tangent and level.	800 (dir. taper)	16	-3	53,300	7,600
Austin Blvd. WB, Eisenhower Expwy.	3	Depressed fwy., tangent and level.	1,100 (parallel)	16	-3	59,500	4,500
Diversey Ave. SB, Kennedy Expwy.	4	Embanked fwy., slight curve left, level.	900 (parallel)	16	+3	72,000	3,500
Right-hand entrance ramps							
First Ave. WB, Eisenhower Expwy.	3	Depressed fwy., tangent and level.	350 (dir. taper)	16	-2	52,500	4,500
Sayre Ave. SB, Kennedy Expwy.	3	Depressed fwy., tangent and level.	800 (parallel)	16	-2	52,700	1,300

studied (Harlem Ave. eastbound (EB) entrance ramp, Eisenhower Expressway) from a point more than 2 mi downstream. Field data were collected at all locations by means of time-lapse movie photography, supplemented by "direct observation" measurements.

With one exception (Diversey Ave. southbound (SB) left-hand entrance ramp, Kennedy Expressway), all of the locations studied were situated on three-lane level sections of six-lane depressed freeway. The Austin Blvd. and Harlem Ave. left-hand entrance ramps were elements of two internal diamond interchanges, located $1\frac{3}{4}$ mi apart on the Eisenhower Expressway in west suburban Chicago. The First Ave. and Sayre Ave. right-hand entrance ramps were both elements of conventional external diamond interchanges, also situated in suburban areas on the Eisenhower and Kennedy Expressways, respectively. The Diversey Ave. left-hand entrance ramp entered into a four-lane elevated section of the Kennedy Expressway at a point some 6 mi north of the city center. It was the only ramp studied whose approach to the freeway lay on an upgrade and which did not form part of a diamond interchange.

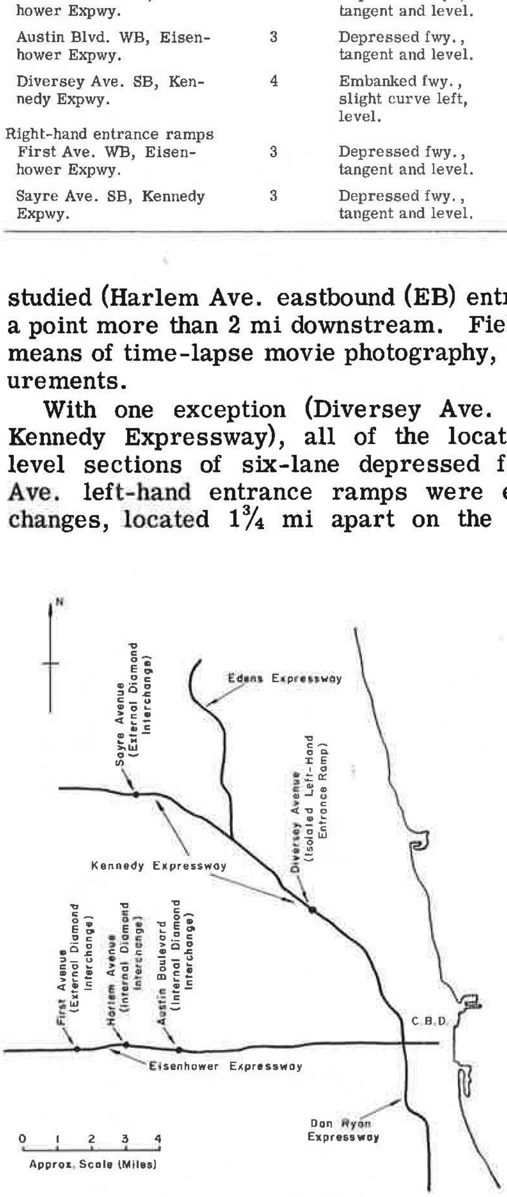


Figure 1. Operational studies—location of interchanges studied in first section of report.

Figure 1 shows the location of each of the ramps studied within the Chicago area expressway system. Figures 2 and 3 are aerial views of the Harlem Ave. and First Ave. interchanges and Figures 4 through 9 are ground-level photographs of the six entrance ramps studied.

Volume Studies

Figures 10, 11 and 12 illustrate the volume distributions by lane in the vicinity of the Harlem Ave. EB left-hand entrance

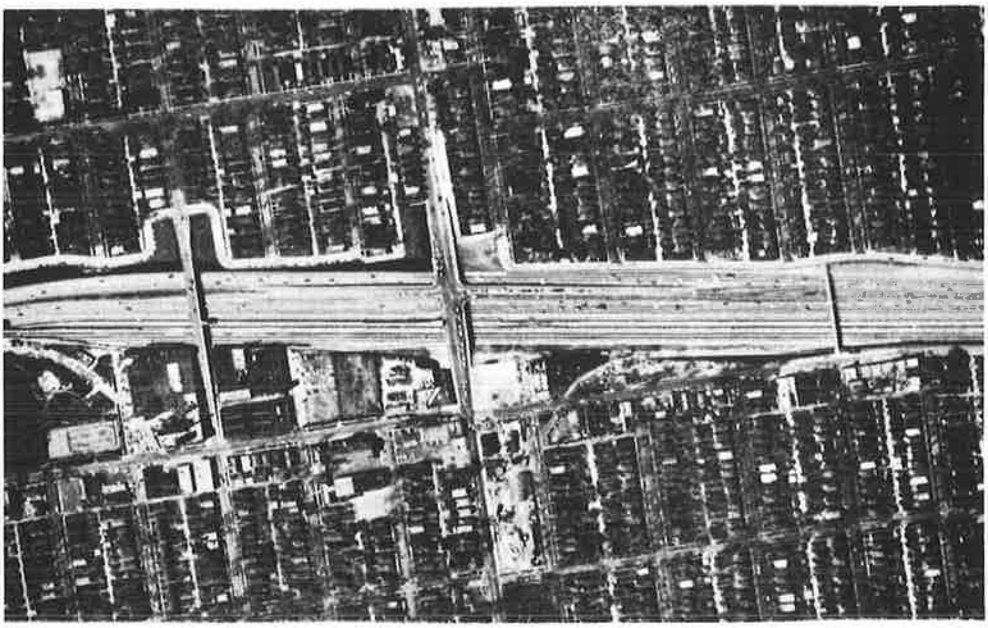


Figure 2. Aerial view of Harlem Ave. internal diamond interchange, Eisenhower Expressway, Chicago.

Figure 3. Aerial view of First Ave. external diamond interchange, Eisenhower Expressway, Chicago.

Figure 4. Harlem Ave. EB left-hand entrance ramp, Eisenhower Expressway, Chicago.

Figure 5. First Ave. WB right-hand entrance ramp, Eisenhower Expressway, Chicago.

Figure 6. Austin Blvd. WB left-hand entrance ramp, Eisenhower Expressway, Chicago.

Figure 7. Harlem Ave. WB left-hand entrance ramp, Eisenhower Expressway, Chicago.

Figure 8. Sayre Ave. SB right-hand entrance ramp, Kennedy Expressway, Chicago.

Figure 9. Diversey Ave. SB isolated left-hand entrance ramp, Kennedy Expressway, Chicago.

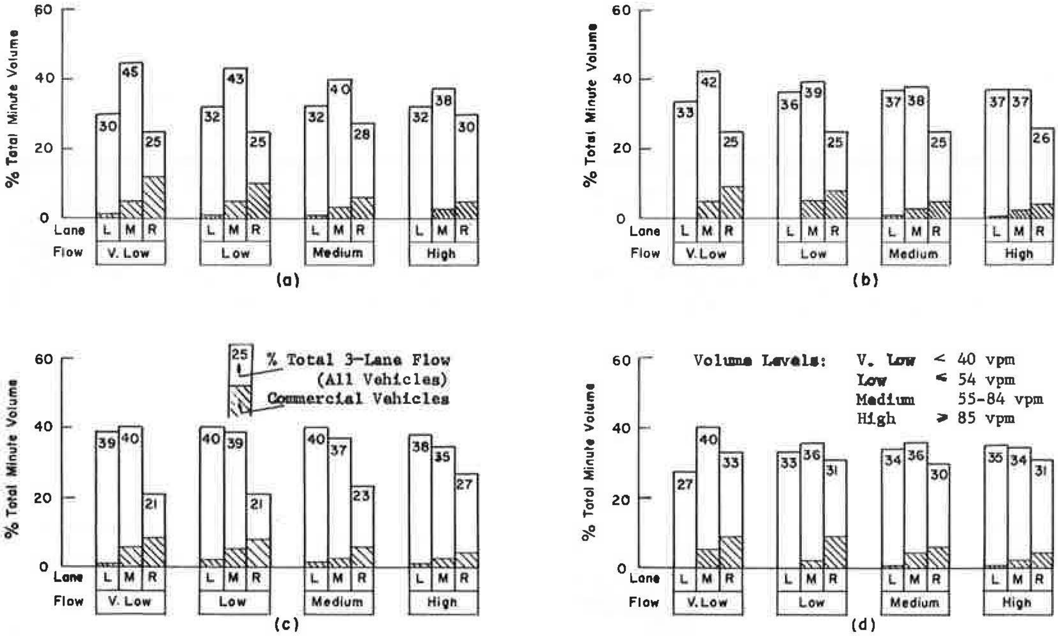


Figure 10. Volume distributions by lane: (a) Harlem Ave. left-hand entrance ramp, at nose; (b) First Ave. right-hand entrance ramp, at nose; (c) Harlem Ave. left-hand entrance ramp, at end of acceleration lane; and (d) First Ave. right-hand entrance ramp, at end of acceleration lane.

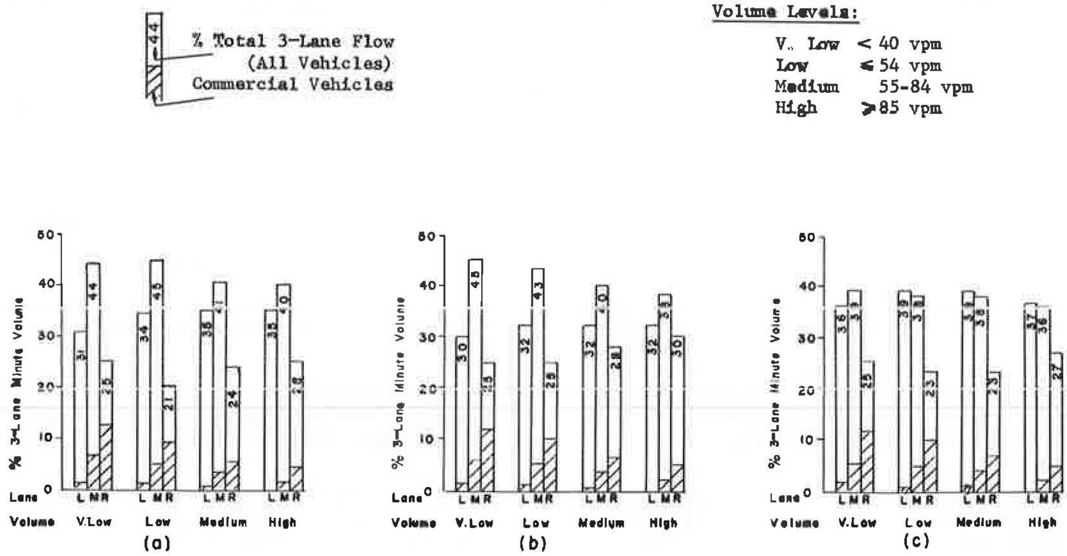


Figure 11. Volume distributions by lane in vicinity of Harlem Ave. EB left-hand entrance ramp, Eisenhower Expressway, Chicago: (a) at nose of left-hand exit ramp, 1,700 ft upstream; (b) at nose; and (c) at point 3,300 ft downstream.

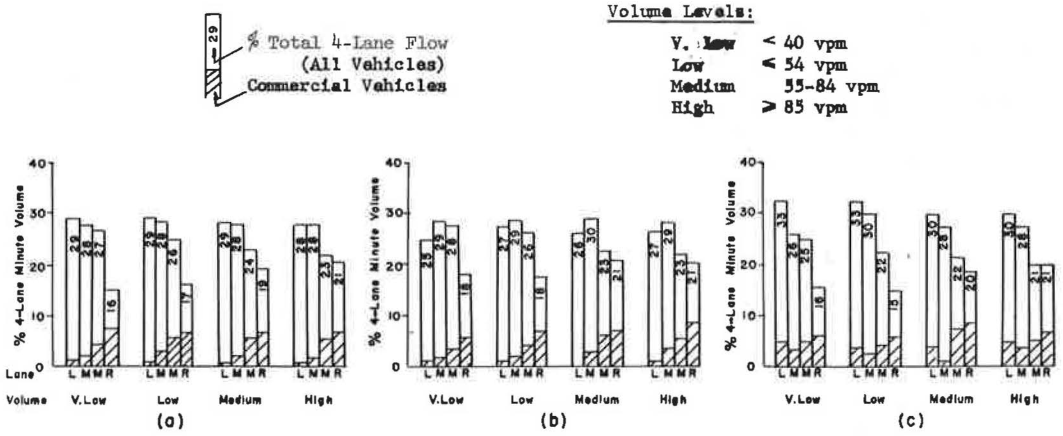


Figure 12. Volume distribution by lane in vicinity of Diversey Ave. SB isolated left-hand entrance ramp, Kennedy Expressway, Chicago: (a) "average" lane distribution on eight-lane freeway; (b) at nose; and (c) at end of acceleration lane.

ramp, the Diversey Ave. SB left-hand entrance ramp and the First Ave. westbound (WB) right-hand entrance ramp.

Figure 10 compares the lane distributions at the nose and at the end of the acceleration lane of the Harlem Ave. EB left-hand ramp and First Ave. WB right-hand ramp for four different levels of mainstream freeway volume. These volume levels (i.e., v. low < 40 vpm, low ≤ 54 vpm, medium 55 to 84 vpm, and high ≥ 85 vpm), measured at a point immediately upstream of the ramp nose, correspond to the levels used in the exit-ramp study referred to earlier (1). From this figure it may be concluded that:

1. The left-lane volumes at the nose of the Harlem Ave. left-hand entrance ramp were consistently lower and the center- and right-lane volumes consistently higher than the comparable volumes at the nose of the First Ave. right-hand on-ramp. The proportion of traffic traveling in the extreme left lane on the approach to the Harlem Ave. left-hand entrance ramp was considerably lower at all volume levels than that using the adjacent center lane, but still higher than the proportion traveling in the right lane.

2. Immediately downstream from the left-hand entrance, the left lane carried from 1 to 3 percent more traffic than the center lane, and 11 to 19 percent more than the right lane. Downstream from the right-hand entrance, the distribution of traffic was much more uniform; the maximum difference between individual lane proportions in this case was only 6 percent (as opposed to 19 percent downstream of the left-hand on-ramp). There was a tendency in both cases for the lane distributions to even out at higher volume levels.

3. At both locations, the distribution of trucks in the freeway lanes opposite the ramp nose was roughly 60 percent in the right lane, 36 percent in the center lane and only 4 percent in the left lane, reflecting the presence of "Trucks Use Two Right Lanes" signs along the expressway. Immediately downstream of the left-hand ramp, however, the left lane carried over 35 percent of the total commercial traffic, due to the large number of trucks and buses entering from the left-hand on-ramp.

In interpreting the data of Figure 10, it is important to note that the Harlem Ave. left-hand entrance ramp was located only some 1,700 ft downstream of a left-hand exit ramp which carried an ADT of 7,600 veh. The pronounced effect of this off-ramp on the volume distribution in the vicinity of the on-ramp is illustrated in Figure 11. Clearly, the 6 to 9 percent difference between the left- and center-lane volumes observed at the nose of the left-hand entrance ramp may be largely attributed to the effect of the left-hand exit ramp immediately upstream. There was apparently no tendency for the left lane to "fill up" over the 1,700 ft between the left-hand exit and entrance ramps. This point is discussed in more detail later. Figure 11c illustrates the distribution of main line traffic at a point 3,300 ft downstream of the Harlem Ave. left-hand entrance. At this point, approximately halfway between the Harlem Ave. and Austin Blvd. interchanges, the volume distribution appears to have stabilized, with the left and center lanes carrying approximately the same proportions of traffic at all flow levels.

As might be expected, a single isolated low-volume left-hand entrance ramp had considerably less effect than an internal diamond interchange on the mainline volume distribution. In the case of such an isolated ramp, the volume distributions in the vicinity of the entrance ramp differed very little from the distributions determined on an "average" four-lane section of freeway (average distributions computed from data collected on the Eisenhower Expressway in Chicago), though there was again a tendency for the left-hand lane to carry a slightly lower volume than "normal" on the approach to the on-ramp and a higher volume at the end of the acceleration lane (Fig. 12).

Speed Studies

Observations of average-minute-lane-speeds (average of the individual speeds of a series of vehicles passing a given point in a given lane of a freeway within 1 min) adjacent to the Harlem Ave. left-hand and First Ave. right-hand entrance ramps were classified according to the following general conditions of flow:

1. Off-peak—three-lane density < 35 vpm/lane (free flow);
2. Peak uncongested—three-lane density between 35 and 60 vpm/lane (no complete stoppages); and
3. Peak congested—three-lane density > 60 vpm/lane (regular stoppages in all lanes).

For each of these flow conditions, cumulative distributions of average-minute-lane-

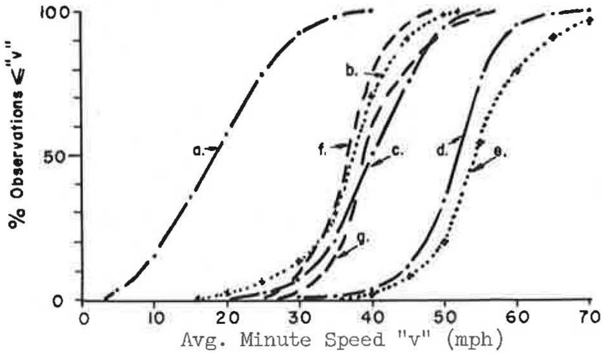
determined for the speeds of entering vehicles, measured in this case over a 300-ft trap length located on the ramp proper immediately upstream of the ramp nose.

These ramp speeds are also plotted on Figure 13. The distributions of peak-congested and peak-uncongested ramp speeds were not significantly different at the Harlem Ave. left-hand entrance ramp, and no peak congested flow condition was observed at the First Ave. right-hand entrance ramp; consequently, no curves are plotted in Figure 13 for the Harlem Ave. ramp peak congested or the First Ave. peak congested conditions.

Off-Peak Speeds.—During off-peak periods, the distribution of average-minute-speeds in the left lane adjacent to the left-hand on-ramp was significantly lower (at the 5 percent level of significance) and that of right-lane speeds was significantly higher (at the 5 percent level of significance) than in the comparable lanes adjacent to the right-hand entrance. There was no significant difference, however, in the distributions of the center-lane speeds.

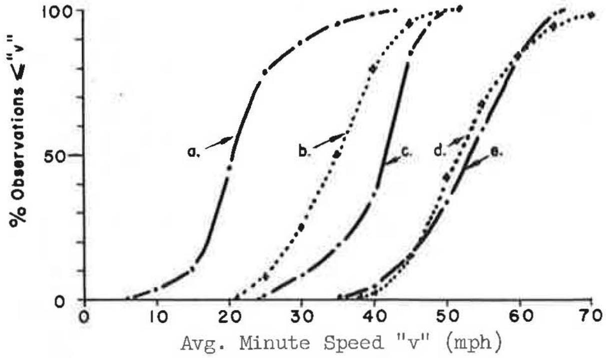
The average speeds of ramp vehicles approaching the nose of the left-hand entrance were about 4 to 6 mph higher than the equivalent speeds measured at the right-hand ramp. This difference was significant at the 10 percent level. The average speed of entering trucks was approximately 4 mph lower than that of entering automobiles at the left-hand ramp, and 3 mph lower at the right-hand ramp. These differences were not statistically significant.

Comparison of the average speed differentials between entering vehicles and vehicles traveling in the adjacent through lane at each ramp indicated that this differential varied from 8 to 15 mph at the left-hand ramp to 6 to 12 mph at the right-hand ramp. The distributions of average speed differentials at the two locations were not, however, significantly different at the 10 percent level. There was a tendency at both locations for entering speeds to be slightly lower during periods of very heavy flow.



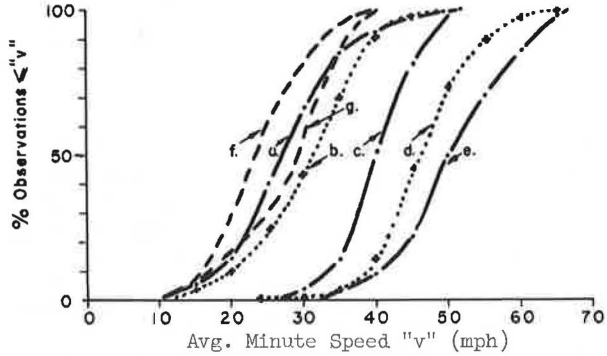
a) RAMP & LEFT-LANE SPEEDS

a Harlem, L. Lane, Peak Cong.
b First, L.Lane, Peak Uncong.
c Harlem, L.Lane, Peak Uncong.
d Harlem, L.Lane, Off-Peak
e First, L.Lane, Off-Peak
f Harlem, Ramp, Peak Uncong.
g Harlem, Ramp, Off-Peak



b) CENTER-LANE SPEEDS

a Harlem, Peak Congested
b First, Peak Uncongested
c Harlem, Peak Uncongested
d First, Off-Peak
e Harlem, Off-Peak



c) RAMP & RIGHT-LANE SPEEDS

a Harlem, R.Lane, Peak Cong.
b First, R.Lane, Peak Uncong.
c Harlem, R.Lane, Peak Uncong.
d First, R. Lane, Off-Peak
e Harlem, R.Lane, Off-Peak
f First, Ramp, Peak Uncong.
g First, Ramp, Off-Peak

Figure 13. Distributions of average-minute-lane-speeds and average-minute-ramp-speeds at Harlem Ave. EB left-hand entrance ramp and First Ave. WB right-hand entrance ramp, Eisenhower Expressway, Chicago.

The average speeds in the vicinity of the Diversey Ave. left-hand entrance ramp (not illustrated in Figure 13) followed the same general pattern, with ramp vehicle speeds very slightly lower and truck speeds about 5 mph lower than automobile speeds in the main through lanes.

Peak Uncongested Speeds. —The average peak uncongested speeds in all three lanes adjacent to the left-hand entrance ramp were significantly higher (at the 5 percent level of significance) than the comparable speeds adjacent to the right-hand ramp over the whole volume range studied. Similarly, the average speeds of entering vehicles were significantly higher at all volume levels (again at the 5 percent level) at the left-hand than at the right-hand on-ramp. There was again no significant difference between the speeds of entering trucks and entering automobiles.

As the total volume increased adjacent to the left-hand entrance ramp, the average speed of through traffic in the left lane dropped below that for the center lane. The left-lane speed again remained consistently higher, however, than the right-lane speed.

Peak Congested Speeds.—During the peak congested period, there was a considerable backup of traffic from downstream of the Harlem Ave. interchange. This backup greatly reduced average speeds in all lanes near the left-hand entrance ramps. However, the mean speed in the left lane during that period was somewhat lower than that in the center lane. Congestion caused by high-density merging maneuvers at the left-hand entrance undoubtedly contributed to the lowering of left-lane speeds.

Headways and Gap Availability

Figure 14 summarizes the relative availability of time gaps in the adjacent lane at the nose of the Harlem Ave. EB left-hand entrance ramp and the First Ave. WB right-hand entrance ramp on the Eisenhower Expressway. The time-gap distributions are presented in terms of the percentage of time expended in gaps greater than or equal to a given value. They refer only to gaps in the lane adjacent to the acceleration lane at each ramp (i.e., in the extreme left lane at Harlem Ave. and in the extreme right lane at First Ave.). Distinction is again drawn between the distributions obtained at four different levels of mainstream volume (< 40, ≤ 54, 55-84, ≥ 85 vpm). A total of 2,067 and 1,810 gaps were observed at the left- and right-hand ramps, respectively.

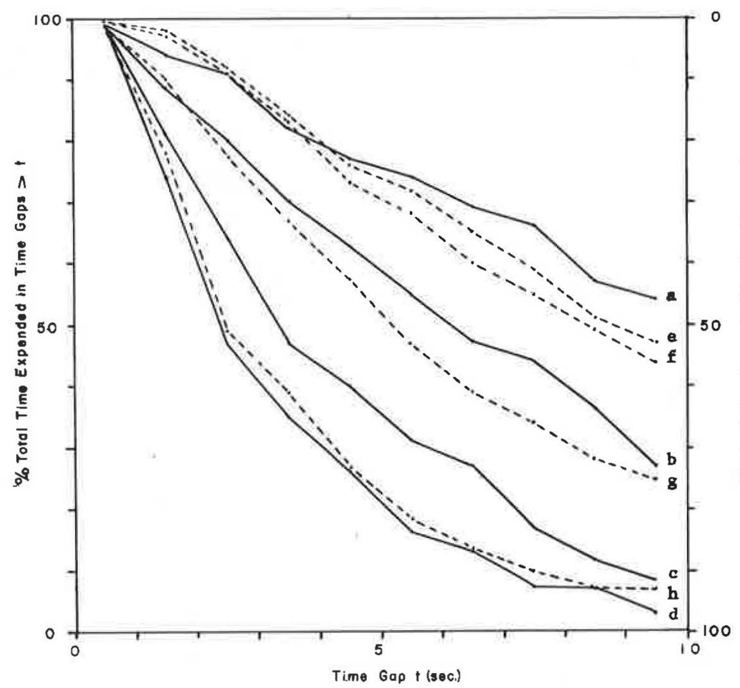
Before discussing the results of this study, it should be noted that the analysis in its present form is somewhat limited in scope. At each study location, observations were restricted solely to the ramp nose. No attempt was made to study the variation in the size of individual time gaps over the length of the acceleration lanes. Similarly, all observations were made during periods of free flow on the expressway. No analyses were made of gap availability during periods of high-density forced flow,

ability at the entrance ramp nose.

With these qualifications accepted, the following statements may be made concerning the distribution of time headways and the availability of time gaps in the lanes adjacent to the two ramps studied:

1. The modal time-gap size (at all volume levels) in the left lane adjacent to the Harlem Ave. left-hand entrance ramp was 1 to 2 sec. This mode became more exaggerated as the total three-lane volume increased. The total amount of time expended in such gaps, however, was very small, less than 5 percent of the total study time at all volume levels. By comparison, the modal value for time gaps in the adjacent right lane at the First Ave. right-hand entrance ramp varied from 3 to 4 sec at low total volumes to 1 to 2 sec at high volumes. Again, the total amount of time expended in such gaps was very small.

2. At all except high total volume levels, there was a significant difference (at the 5 percent level) between the total amount of time expended in time gaps less than t in the adjacent lanes at the two locations. During periods of very low flow (< 40 vpm), a significantly larger proportion of time was expended in gaps of less than 5 sec in the lane adjacent to the left-hand entrance ramp than in the comparable lane at the right-hand entrance ramp. During periods of low (≤ 54 vpm) and medium (55-84 vpm) total flow, however, the position was reversed and a significantly higher proportion of time was expended in time gaps of less than 5 sec at the right-hand entrance ramp than at the left-hand entrance ramp (Fig. 14). If a value of approximately 5 sec is assumed for the minimum acceptable gap size at each location, the results indicate that the probability of an entering driver encountering an acceptable gap was higher at the right-hand ramp during periods of low and medium flow and at the left-hand ramp during periods of very low flow. At high total flows (> 84 vpm), there was no significant



a Harlem Ave. LH On-Ramp, V. Low Flow (<40 vpm)
b Harlem Ave. LH On-Ramp, Low Flow (≤54 vpm)
c Harlem Ave. LH On-Ramp, Medium Flow (55-84 vpm)
d Harlem Ave. LH On-Ramp, High Flow (≥85 vpm)

e First Ave. RH On-Ramp, V. Low Flow (<40 vpm)
f First Ave. RH On-Ramp, Low Flow (≤54 vpm)
g First Ave. RH On-Ramp, Medium Flow (55-84 vpm)
h First Ave. RH On-Ramp, High Flow (≥85 vpm)

Figure 14. Time-gap distributions in adjacent lane at Harlem Ave. EB left-hand entrance ramp and First Ave. WB right-hand entrance ramp, Eisenhower Expressway, Chicago.

difference between the time-expended distributions at the two locations (Fig. 14). During this flow condition, therefore, the probability of an entering driver finding an acceptable gap was approximately the same at each location.

3. At the right- and left-hand ramps studied, the proportion of acceptable gaps (i.e., gaps ≥ 5 sec in the adjacent lane) decreased with increase in total three-lane volume.

Lane Changing

The intensity of lane changing in the vicinity of the left- and right-hand entrance ramps studied is summarized in Table 2. On the basis of these data it would appear that:

1. Immediately upstream of the nose of the entrance ramps, the intensity of lane changing (measured in terms of the total number of lane changes between all lanes per 1,000 ft/min) was 1.6 times higher approaching the left-hand entrance ramp than on the approach to the right-hand entrance ramp. Values of the ratio:

$$\frac{\text{Avg. No. lane changes per 1,000 ft/min upstream of left-hand entrance}}{\text{Avg. No. lane changes per 1,000 ft/min upstream of right-hand entrance}}$$

for different interlane movements were as follows:

TABLE 2

LANE CHANGES IN VICINITY OF HARLEM AVE. EB LEFT-HAND ENTRANCE RAMP AND FIRST AVE. WB RIGHT-HAND ENTRANCE RAMP, EISENHOWER EXPRESSWAY, CHICAGO, AS FUNCTION OF RAMP AND MAINSTREAM MINUTE VOLUME LEVELS

Ramp Type	Ramp Vol.[a]	Lane Changes (No./1,000 ft/min)											
		Low Fwy. Vol.[b]				Med. Fwy. Vol.[b]				High Fwy. Vol.[b]			
		L-C	C-R	R-C	C-L	L-C	C-R	R-C	C-L	L-C	C-R	R-C	C-L
		(a) Within 1,000-ft section of freeway immediately upstream											
RH	Low	1.10	0.59	0.98	0.82	0.65	0.42	0.91	0.85	0.47	0.41	0.62	0.76
LH		1.37	1.20	0.96	1.13	1.70	1.48	0.75	0.86	1.35	1.28	0.68	0.39
RH	Medium	0.91	0.68	1.21	0.97	0.62	0.39	0.99	0.48	0.69	0.67	0.76	0.88
LH		1.43	1.38	1.19	0.93	1.76	1.66	1.22	1.33	1.44	1.33	1.03	1.33
RH	High	0.82	0.41	1.15	0.81	0.65	0.51	1.04	0.51	0.52	0.46	0.83	0.61
LH		1.38	0.83	1.11	0.55	1.55	0.83	0.42	0.37	1.27	0.61	0.42	0.37
		(b) Adjacent to acceleration lanes											
RH	Low	1.00	0.80	1.16	1.05	0.82	0.73	0.99	0.91	0.68	0.67	0.79	0.71
LH		3.00	1.16	0.79	1.04	1.43	1.30	1.56	1.17	1.29	0.92	0.68	1.01
RH	Medium	0.75	0.90	1.27	0.98	1.01	1.12	2.13	1.07	0.71	0.89	1.21	0.90
LH		3.78	1.21	0.68	1.53	2.91	2.89	1.14	0.86	1.29	0.81	0.62	0.73
RH	High	0.69	0.91	1.72	1.21	0.63	1.15	2.78	1.42	0.61	1.07	2.30	1.14
LH		5.91	1.21	0.45	0.76	4.80	1.14	0.45	1.60	3.75	0.52	0.38	1.35

[a]Low, ≤ 5 vpm; medium, 6-15 vpm; and high, ≥16 vpm.
[b]Low, ≤54 vpm; medium, 55-84 vpm; and high, ≥85 vpm.

From center to right lane = 2.3/1,
From right to center lane = 0.8/1, and
From center to left lane = 1.2/1,

for an overall average of 1.6/1.

2. The predominate lane-changing movement upstream of the left-hand entrance was to the right (i.e., from the left lane to the center lane and from the center lane to the right lane) at all volume levels. Upstream of the right-hand ramp, there was a more even amount of lane changing between all lanes, though again there was a slight predominance of movements away from the ramp. At both locations, the intensity of lane changing first increased and then decreased with continuing increases in freeway flow. Individual maneuver lengths (not illustrated here) varied from 70 to 500 ft and tended to be shorter at higher total flows.

3. There were 1.8 times as many lane changes per 1,000 ft/min adjacent to the acceleration lane of the left-hand ramp as there were adjacent to the acceleration lane of the right-hand ramp. Values of the ratio:

$$\frac{\text{Avg. No. lane changes per 1,000 ft/min adjacent to left-hand ramp}}{\text{Avg. No. lane changes per 1,000 ft/min adjacent to right-hand ramp}}$$

for different interlane movements in this case were:

From left to center lane = 4.3/1,
From center to right lane = 1.3/1,
From right to center lane = 0.6/1, and
From center to left lane = 1.1/1,

for an overall average of 1.8/1. The predominate movements at both locations were again away from the entrance ramps. Again the intensity of lane changing increased and then decreased with increase in freeway volume. Movements away from both ramps increased directly as ramp volumes increased. During peak volumes, a separate analysis indicated that there was a net lane changing over the length of the left-hand ramp acceleration lane of 12.8 lane changes per 100 ft/hr, compared with a net figure of 2.3 lane changes per 100 ft/hr adjacent to the right-hand ramp. At lower volumes these figures increased to 16.2 and 12.3 lane changes per 100 ft/hr, respectively. (These figures include movements between all mainstream lanes.)

4. Table 3 summarizes the intensity of lane changing within the 1,700 ft of roadway separating the left-hand exit and entrance ramps at the Harlem Ave. EB interchange. It is apparent that there was no tendency for time gaps created in the left-lane traffic stream due to vehicles exiting at the left-hand exit ramp to fill up between the nose of the exit ramp and the nose of the entrance ramp 1,700 ft downstream. This conclusion is based on a total sample of 4 hr of lane-changing data.

Hazardous Maneuvers

A qualitative analysis of the incidence of hazardous maneuvers was performed at each of the ramps studied. For the purposes of the analysis a hazardous maneuver is defined as a maneuver by an entering vehicle which caused the driver of a following through vehicle to change his speed or direction violently. Although no attempt was made to develop a precise quantitative definition of a hazardous maneuver, errors of definition were kept to a minimum by insuring that the same individual performed all of the analyses.

There was very little difference in the overall incidence of hazardous maneuvers at the left- and right-hand ramps studied. No significant relationship could be developed between the incidence of hazardous maneuvers and traffic volume for the range of daytime volumes studied. At the four left-hand ramps, an average of 18.7 hazardous entries per 1,000 ramp vehicles were observed. At the two right-hand ramps, the comparable average figure was 17.6/1,000 ramp vehicles.

At the four left-hand entrances, an average of three hazardous maneuvers per ramp per hour were observed in which through vehicles made use of the acceleration lane to pass other through vehicles. At the right-hand entrance ramps, only one such maneuver was observed during a period of 8 hr. A relatively large percentage of the hazardous direct entries (i.e., entering maneuvers in which the entering vehicle cut directly across one or more lanes of traffic) observed at all of the left-hand entrance ramps studied involved trucks. This was due primarily to a regulation requiring trucks to use the two extreme right-hand lanes on the freeway. No such tendency was observed at the right-hand ramps.

TABLE 3

LANE CHANGES WITHIN 1,700-FT SECTION OF FREEWAY[a]

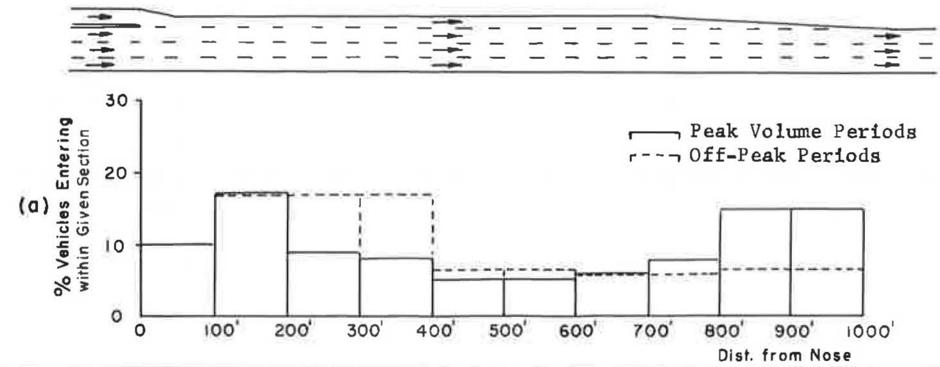
Fwy. Vol.[b]	Lane Change Direction	Lane Changes (No./min)
Low	L-C	2.37
	C-R	2.80
	R-C	2.18
	C-L	2.11
Medium	L-C	3.20
	C-R	2.92
	R-C	2.71
	C-L	2.88
High	L-C	2.96
	C-R	2.66
	R-C	2.16
	C-L	2.79

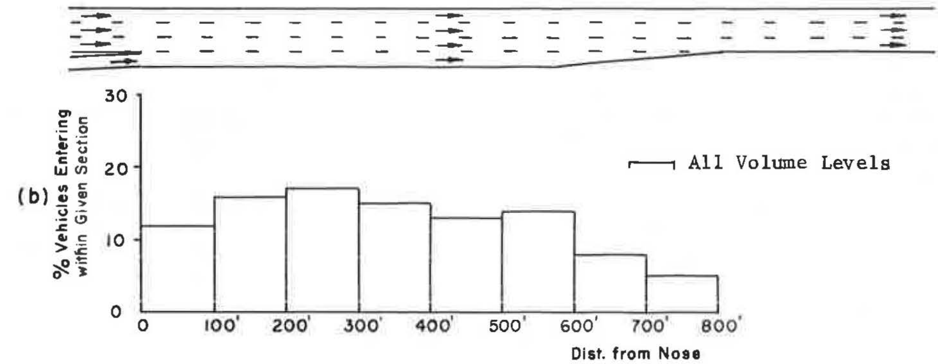
[a]Between noses of Harlem Ave. EB left-hand exit and entrance ramps, Eisenhower Expressway, Chicago.
[b]Low, ≤ 54 vpm; medium, 55-84 vpm; and high ≥ 85 vpm.

Zone of Entry Onto Through Lanes

Figure 15 illustrates the variation of point of entry onto the main freeway lanes (the point at which a vehicle's front near-side wheel finally crosses from the acceleration lane onto the adjacent through lane) for the Harlem Ave. EB left-hand entrance ramp, the First Ave. WB right-hand entrance ramp and the Sayre Ave. SB right-hand entrance ramp. In each case, the data are subdivided into conditions of peak and off-peak flow in the mainstream. Congested and uncongested observations were combined in the case of the peak flow condition.

At the Harlem Ave. left-hand entrance ramp, the zones of entry were concentrated mainly in the first 400 ft during off-peak





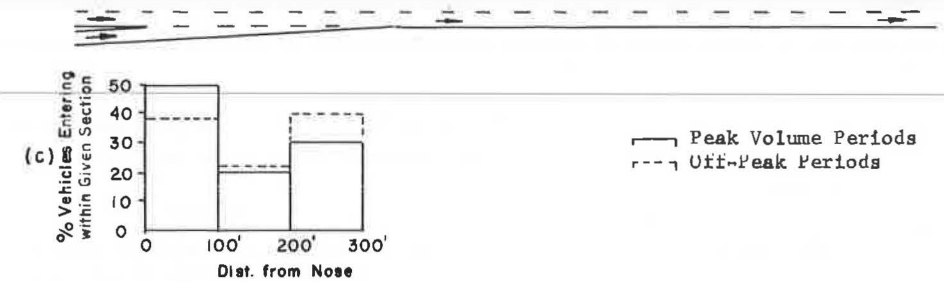


Figure 15. Zone of entry distributions: (a) Harlem Ave. EB left-hand entrance ramp, Eisenhower Expressway; (b) Sayre Ave. SB right-hand entrance ramp, Kennedy Expressway; and (c) First Ave. WB right-hand entrance ramp, Eisenhower Expressway.

periods and in the first 200 and last 350 ft during the peak periods. The equivalent distributions at the First Ave. right-hand entrance ramp, which has a very short acceleration lane, were more uniform for both volume conditions. The absence of an adequate acceleration lane, however, encouraged a number of drivers to stop at the nose of the ramp to wait for an acceptable gap, thereby creating an increased number of early entries. The Sayre Ave. entrance ramp observations, for a right-hand ramp provided with an 800-ft long acceleration lane, showed an even distribution of points of entry at all volume levels.

The results of the Harlem Ave. zone-of-entry studies, when considered in isolation, indicate that the use of a long acceleration lane by no means guarantees satisfactory operation of a left-hand entrance ramp. At high total volumes, a large number of drivers were forced to the end of the acceleration lane without finding an acceptable gap and were then brought to a complete halt. Obviously, such a procedure reduces the efficiency of ramp operation. It should be noted, however, that the acceleration lane was sufficiently long to prevent backups due to such stoppages from extending up the ramp proper.

During off-peak periods, relatively few entering vehicles made full use of the extra length of acceleration lane provided for their benefit, whereas a number of through vehicles utilized the speed-change lane as a fourth through lane to pass other vehicles. This latter practice clearly constitutes a hazardous maneuver that might be dealt with by reducing the length of the acceleration lane. Such a proposition, however, is in conflict with the conclusions of the preceding paragraph. A more satisfactory solution might be to extend the ramp nose by some 200 ft beyond its present position.

SYSTEM STUDY—EISENHOWER EXPRESSWAY EB

Figure 16 illustrates a 2-mi section of the eastbound Eisenhower Expressway in west suburban Chicago. This section of freeway contains two internal diamond interchanges, at Harlem Ave. and Austin Blvd., spaced about 7,300 ft apart.

Traffic operations within this section of freeway were studied on four separate occasions in the spring and summer of 1964, during the evening inbound peak period (between 4:00 and 6:30 p.m.). The condition during this period was studied to avoid congestion backing up into the study area from downstream, as occurred regularly in the morning peak period.

Data were collected by means of coordinated time-lapse movie photography. Films were taken simultaneously from each of eight locations illustrated in Figure 16. Each series of films was synchronized by stopwatch timing supplemented by a series of timed runs through the study section in a marked vehicle. All cameras were equipped with synchronous electric motors connected to the main freeway lighting circuit, giving a constant film speed in each case of 60 ft/min. Shoulder markings 25 ft apart were laid down at each study location to provide a distance scale for the analysis.

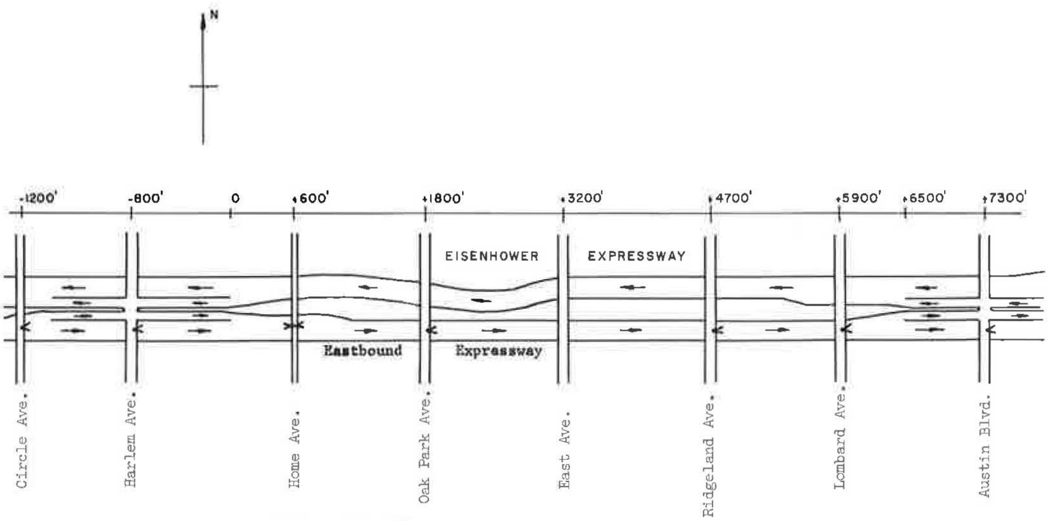


Figure 16. Eisenhower Expressway EB system study—map of study section showing eight camera locations.

Entrance Ramp Merge Rates and Development of Congestion in Adjacent Freeway Section

At each camera location a speed/volume profile was prepared for each lane and ramp, showing the variation of average-minute-volumes and average-minute-speeds with time throughout the study period. Figure 17 reproduces a section of one of these diagrams, covering a period of 40 min during which a shock wave was propagated in the vicinity of the Ridgeland Ave. overpass and reflected back along the expressway beyond Harlem Ave. Figures 18 and 19 illustrate the variation of speeds upstream of the same ramp for two separate periods of 50 min during which high average merge rates were sustained at the left-hand entrance ramp.

On the basis of these diagrams, it may be concluded that:

1. Extremely high merge rates (i.e., ramp volume plus through volume in left lane) were maintained throughout the study period at the Harlem Ave. entrance ramp. These rates ranged from a 2-hr average flow rate of 1,968 vph to an average 50-min rate of 2,034 vph to sporadic peaks maintained for only a few minutes in excess of

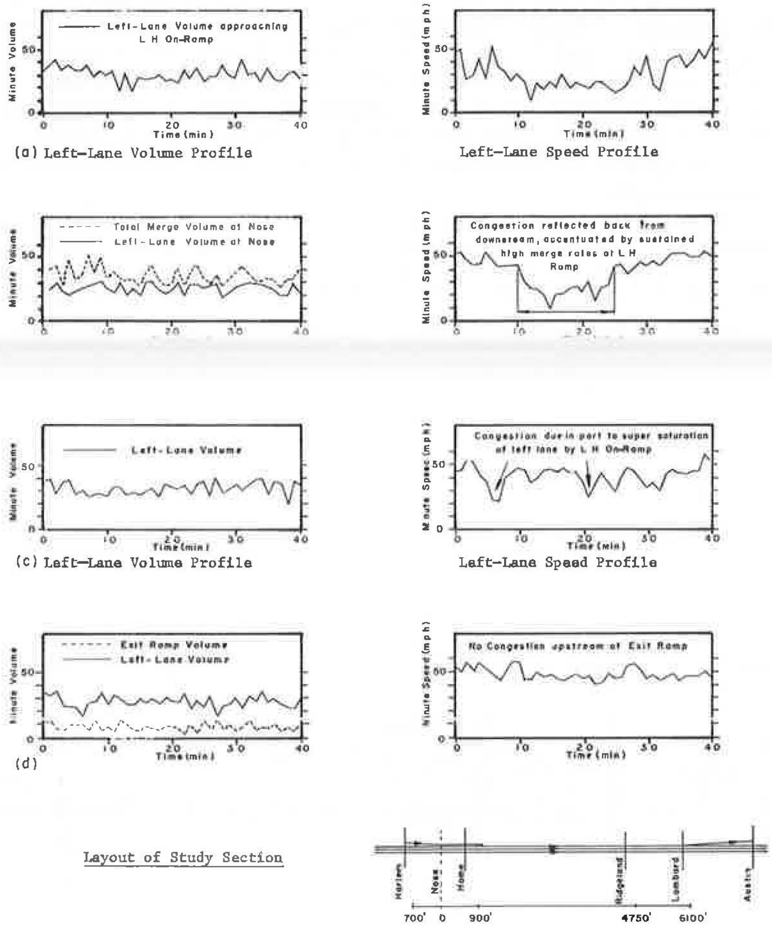


Figure 17. Volume/speed profiles within study section—left lane only: (a) Harlem Ave. overpass, 700 ft upstream of nose of left-hand entrance ramp; (b) Home Ave. overpass, 600 ft downstream of nose of left-hand entrance ramp; (c) Ridgeland Ave. overpass, 4,750 ft downstream of left-hand entrance ramp; and (d) Lombard Ave. overpass, 6,100 ft downstream of left-hand entrance ramp, 600 ft upstream of nose of left-hand exit ramp.

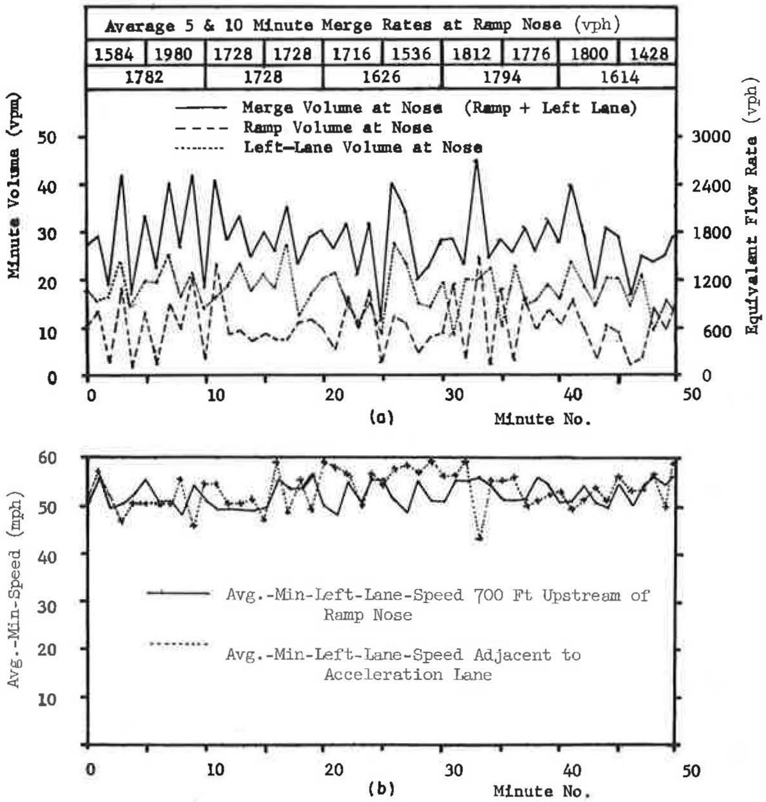


Figure 18. Entrance ramp merge rates (uncongested condition) at nose of Harlem Ave. left-hand entrance ramp, Eisenhower Expressway, Chicago: (a) minute merge volume profiles; and (b) minute speed profiles for left lane in vicinity of ramp.

2,200 vph. Average-minute-speeds in the left lane at a point 700 ft upstream of the ramp nose varied from 10 to 58 mph and, in the same lane at a point immediately downstream of the merge area, from 12 to 52 mph. At no time during the entire study period did the sustained high merge rates result directly in a total breakdown of flow upstream of the ramp. The 10-mph average speeds mentioned and shown in Figures 17 and 19 resulted not from congestion backing up from the merge area, but from a shock wave propagated at a point some 6,000 ft downstream of the ramp (see paragraph 3).

2. For a period of over 50 min, an average merge rate in excess 1,700 vph was maintained at the left-hand entrance ramp without average-minute-speeds in the left lane upstream of the ramp ever falling below 45 mph (Fig. 18). During this period, there was one 5-min merge rate of 1,920 vph and a series of intermittent 1-min merge rates of over 2,000 vph. Throughout the 50 min, the average rate of flow in the left lane approaching the ramp nose was 1,100 vph, with a minimum minute flow rate of 660 vph and a maximum minute flow rate of 1,680 vph.

3. During a separate period of 50 min, an average merge rate of 2,034 vph was observed at the Harlem Ave. entrance ramp (Fig. 17). This period included one period of 10 min during which the average merge rate was 2,360 vph, and four separate 5-min periods during which the average merge rate exceeded 2,000 vph. At no time during the 50 min did the 5-min merge rate drop below 1,872 vph. The average left-lane flow rate approaching the ramp throughout the period was 1,469 vph. During this same period, average-minute-speeds in the left lane upstream of the entrance

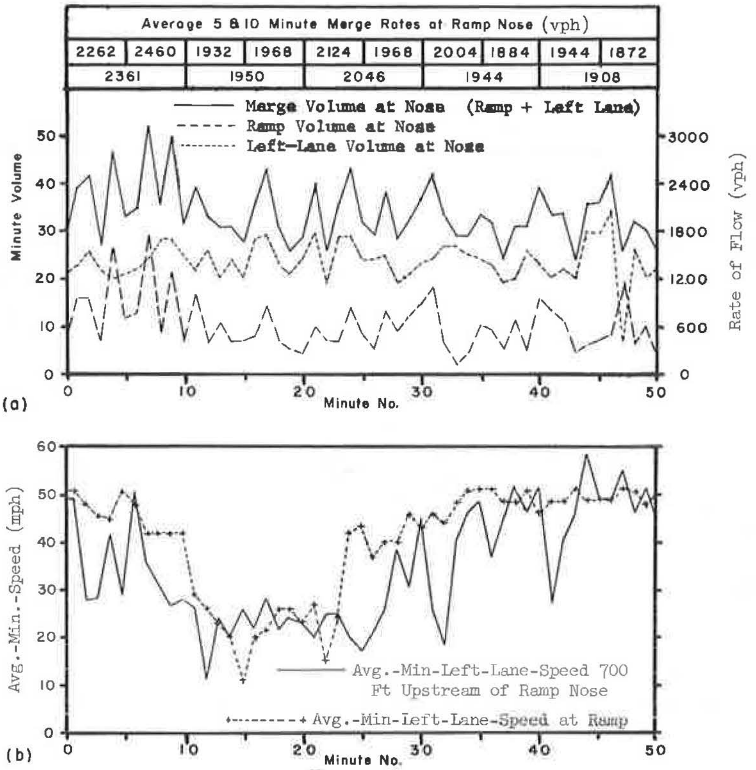


Figure 19. Entrance ramp merge rates (congested condition) at nose of Harlem Ave. EB left-hand entrance ramp, Eisenhower Expressway, Chicago: (a) minute merge volume pro-

ramp dropped to a minimum value of 10 mph (Fig. 19). This maximum slowdown was not, however, created solely by the queueing of freeway vehicles in the left lane upstream of the ramp, but was attributable in large part to the effects of a shock wave reflected back into the vicinity of the ramp from a point 6,000 ft downstream (Fig. 17).

4. During the 50-min period of high sustained merge rates previously mentioned, a condition of extreme forced flow or supersaturation was created in the left lane of the freeway downstream from the entrance ramp. In this condition, the flow in the left lane was extremely sensitive to even relatively minor disturbances. The shock wave illustrated in Figure 17 was propagated in the vicinity of Ridgeland Ave. by a series of abrupt lane changes in and out of the left lane which caused vehicles in that lane to decelerate sharply. This shock wave was reflected back along the left lane of the freeway and its effect was magnified by the sustained high merge rates at the Harlem Ave. entrance ramp, producing a 17-min period of congestion (average-minute-left-lane-speed $\leq$ 35 mph) adjacent to the ramp's acceleration lane and a 23-min congested period immediately upstream of the ramp nose. The total effects of this shock wave and the accompanying slowdowns caused by the sustained high merge rates at the left-hand entrance were totally dissipated and average-minute-left-lane-speeds returned to their original 45- to 50-mph level upstream of the ramp within a period of 25 min without the average 10-min merge rate ever falling below 1,900 vph.

5. Figures 18 and 19 illustrate a number of extremely high, but short-lived, merge rates in excess of 2,000 vph. Almost without exception, such a merge rate, if sustained for more than 1 min, resulted in a significant drop in the average left-lane speed upstream of the ramp nose.

6. Lane changing adjacent to the acceleration lane of the left-hand entrance ramp has already been discussed. It should be noted here, however, that throughout the study period an average net volume of approximately 125 to 150 vph moved out of the left lane over the length of the merge area. This net lane changing reduced the maximum sustained 50-min merge rate of 2,034 vph to a left-lane flow downstream of the ramp of 1,904 vph.

7. There was no indication (Fig. 17) that the Austin Blvd. left-hand exit ramp caused any congestion in the adjacent freeway section.

8. The analyses previously described represent the initial stages of a more detailed series of investigations which are currently in progress at Northwestern University. These investigations include analyses of gap acceptance on both a static and dynamic, basis, the study of maximal permissible merge rates and ramp capacity under all conditions of flow, and an analysis of the entire 2-mi section of freeway considered as a system.

Lane Distribution of Ramp Vehicles Upstream and Downstream of Left-Hand Exit and Entrance Ramps

Using two of the four sets of films described, an analysis was made of the lane distributions of ramp vehicles at varying distances upstream and downstream of the pair of left-hand entrance and exit ramps. The films yielded approximately 2 hr of data. The first hour encompassed primarily free-flow operations on the freeway, and the second hour primarily forced-flow or near-capacity conditions.

Entrance ramp vehicles, entering from the Harlem Ave. EB left-hand entrance ramp, were traced through the study section by means of a master chart on which were recorded the classification, make, year, color and other distinguishing characteristics of each ramp vehicle. From this chart, the number of entrance ramp vehicles traveling

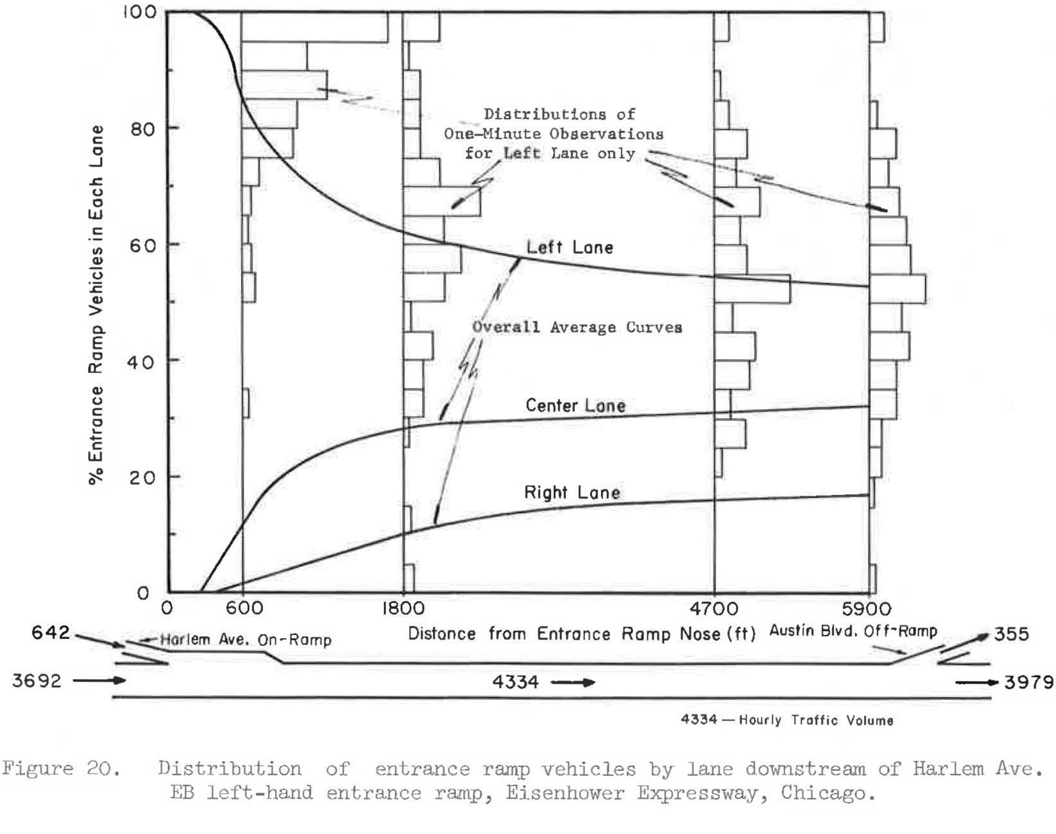


Figure 20. Distribution of entrance ramp vehicles by lane downstream of Harlem Ave. EB left-hand entrance ramp, Eisenhower Expressway, Chicago.

in each lane of the freeway was determined, on a minute-by-minute basis, at four different locations downstream of the entrance ramp nose (Fig. 20). Concurrent with these observations, classified counts of minute-lane-volumes were also made at each location. A similar analysis technique was adopted for vehicles exiting via the Austin Blvd. EB left-hand exit ramp. The average proportion of commercial vehicles within the section throughout the entire study period was 11 percent.

Figure 20 summarizes the results of the entrance ramp studies in general terms. The curves represent the overall average lane distributions of entrance ramp vehicles at varying distances downstream of the entrance ramp nose for the entire study period (i.e., the curves connect the percentage lane distributions computed at each study location on the basis, not of the average of a series of successive minute observations, but of a single aggregate observation period of approximately 2 hr). The extreme variability of the individual minute observations is indicated in Table 4. Because of this variability, it was considered more meaningful to plot a single aggregate curve for the entire study period than an average minute observation curve.

Despite the acknowledged variability of the data, it is clear from Figure 20 that the proportion of ramp vehicles remaining in the left-lane downstream of the entrance ramp fell off rapidly as the distance from the ramp nose increased. It also appears (Table 4) that the frequency distributions of the sets of minute observations became more uniform as distance from the ramp nose increased.

In an effort to develop an understanding of the variables influencing the behavior of entrance ramp vehicles, their lane distribution at varying distances from the ramp nose was analyzed as a function of total mainstream volume. In this connection, it is interesting to note that the mere separation of the second hour of data (mainstream flow rate of 5,050 vph at Oak Park, 60 min of observations) from the first hour (mainstream flow rate of 3,607 vph, 57 min of observations) revealed notable differences in lane distribution percentages.

Total three-lane volume was broken down again into the general categories of high (> 84 vpm), medium (55-84 vpm), and low (< 55 vpm) flow rates. Figure 21 illustrates the variation in the average proportion of entrance ramp vehicles remaining in the left lane, at varying distances downstream of the ramp nose, for each different level

in form to those illustrated for the left lane. The curves for the right lane (also not illustrated) indicate virtually no variation with total volume level.

In Figure 21, the curves for high and medium total volume levels almost coincide, both tending towards an asymptotic value of approximately 50 percent at a distance of 6,100 ft downstream of the ramp nose. The low-volume curve indicates a more rapid transition to this asymptote, at a point in this case some 2,200 ft downstream of the ramp nose. The subsequent increase in the proportion of ramp vehicles traveling in the left lane is simply a reflection of normal weaving and lane-changing maneuvers on a six-lane freeway. Throughout the study period, less than 5 percent of the traffic entering the freeway via the Harlem Ave. left-hand entrance ramp was destined for the Austin Blvd. left-hand exit ramp. All of this traffic remained in the left lane throughout the study section.

TABLE 4

LANE DISTRIBUTION OF LEFT-HAND ENTRANCE RAMP VEHICLES[a]

Location	Dist. Downstream of Ramp Nose (ft)	Sample Size (min)	% Veh in Lane[b]			Std. Dev.[c] (%)			Range[c] (%)		
			Left	Center	Right	Left	Center	Right	Left	Center	Right
Home Ave.	600	117	86	12	2	13	12	4	33-100	0-67	0-17
Oak Park Ave.	1,800	118	62	28	10	20	20	12	0-100	0-100	0-67
Ridgeland Ave.	4,700	112	54	30	16	18	16	12	20-100	0-80	0-50
Lombard Ave.	5,900	102	52	31	17	18	16	12	0-100	0-80	0-50

[a]At selected points downstream of Harlem Ave. EB left-hand entrance ramp, Eisenhower Expressway.
[b]Mean of minute sample observations.
[c]Of minute observation percentages.

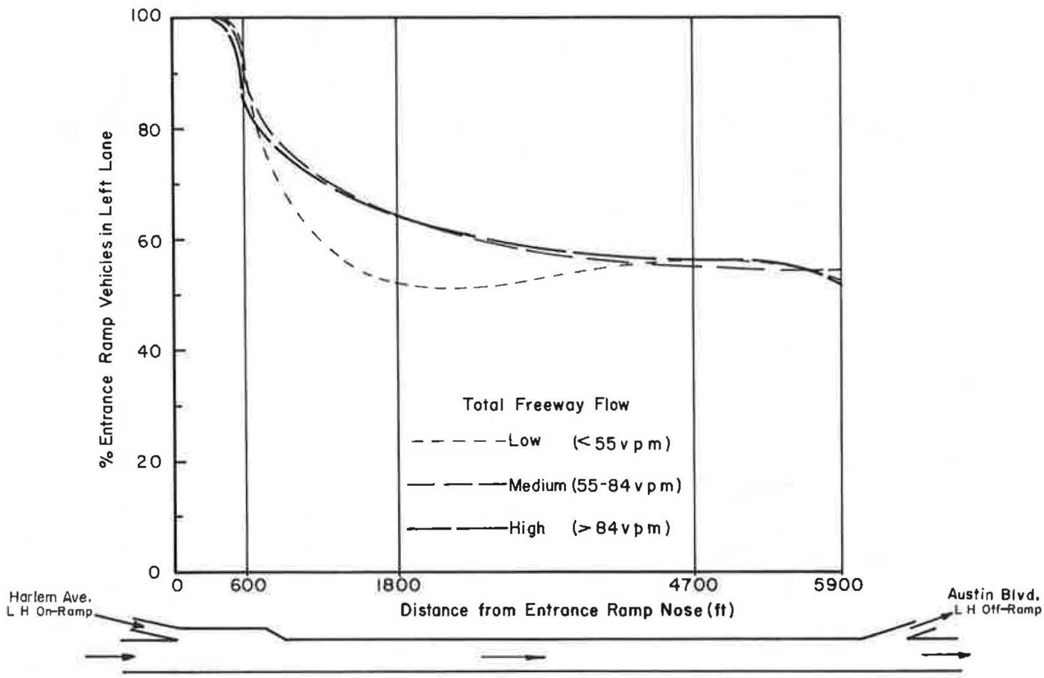


Figure 21. Distribution of entrance ramp vehicles remaining in left lane of expressway downstream of Harlem Ave. EB left-hand entrance ramp, Eisenhower Expressway, Chicago, as function of total volume.

An analysis of the effects of ramp volume and mainstream volume level on the behavior of entering vehicles yielded only inconclusive results. There was a general tendency for the proportion of vehicles remaining in the left lane to increase at all points within the study section with increase in total volume. The behavior of ramp vehicles under conditions of forced flow in the mainstream also appeared to differ considerably from their behavior under conditions of free flow. The scatter of points was too wide, however, and the sample of data too small to permit any detailed conclusions to be drawn.

The results of the companion exit ramp studies are summarized in Table 5 and Figure 22. Figure 22 also indicates the location of directional signs at distances ½, 1 and 2 miles upstream of the exit ramp nose. All of these signs emphasized the fact that the approaching ramp was located on the left rather than the right side of the traveled way.

TABLE 5

LANE DISTRIBUTION OF LEFT-HAND EXIT RAMP VEHICLES[a]

Location	Dist. Upstream of Ramp Nose (ft)	Sample Size (min)	% Veh in Lane[b]			Std. Dev.[c] (%)			Range[c] (%)		
			Left	Center	Right	Left	Center	Right	Left	Center	Right
Harlem Ave.	7,300	104	74	21	5	20	18	11	0-100	0-67	0-50
Home Ave.	5,900	105	81	16	3	19	18	9	0-100	0-100	0-50
Oak Park Ave.	4,700	108	91	8	1	14	12	7	33-100	0-50	0-50
Ridgeland Ave.	1,800	105	97	2	1	11	9	6	50-100	0-50	0-50

[a]At selected points upstream of Austin Blvd. EB left-hand exit ramp, Eisenhower Expressway.
[b]Mean of minute sample observations.
[c]Of minute observation percentages.

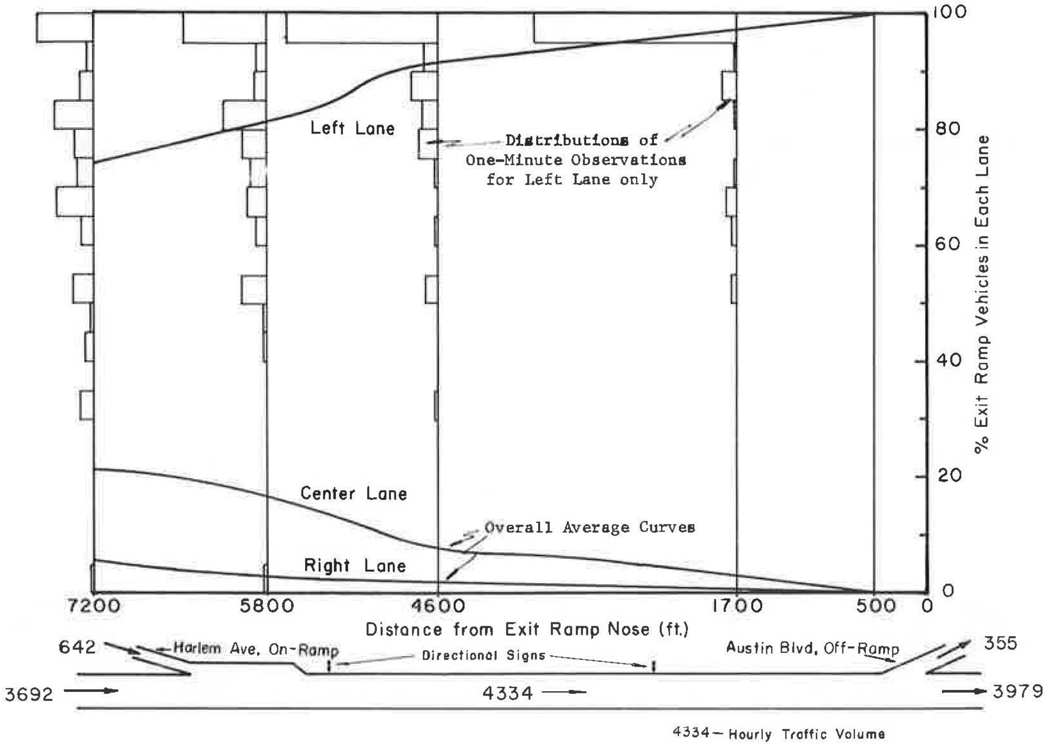


Figure 22. Distribution of exit ramp vehicles by lane upstream of Austin Blvd. EB left-hand exit ramp, Eisenhower Expressway, Chicago.

As in the case of the entrance ramp data, the distributions were again highly dispersed, due largely to the effects of a number of extremely low minute ramp volume observations. In this case, however, the arrival pattern of ramp vehicles at the exit ramp nose was relatively uniform over the entire hour. Minute volumes were low merely because the overall demand for the ramp was low.

Whereas the entrance ramp relationships tended to stable asymptotes within the study section, the exit ramp curves failed to do so. Undoubtedly, a substantial movement of exit ramp vehicles into the left lane occurred upstream of the study section, between the Harlem Ave. camera location and the first directional sign located approximately 1 mi farther upstream. The curves indicate clearly that drivers wishing to use the left-hand exit tended to move into the left lane far in advance of the ramp nose, perhaps as a result of the directional signing or perhaps for fear of missing the ramp completely. Over 70 percent of all exiting vehicles were positioned in the left lane of the freeway at a point 7,250 ft upstream of the left-hand exit ramp. Total freeway volume had no apparent effect on the lateral placement of exit ramp vehicles at any point in the section. These results all tend to confirm those of a previous more limited study (1).

ACCIDENT STUDIES

A number of previous studies (2, 3, 4, 5) have indicated that the average accident rate at left-hand ramps is generally higher, and frequently more severe, than that observed at comparable right-hand terminals. None of these studies, however, was based on more than a very limited sample of study locations, and none of them included any detailed statistical comparison of the two design types.

The final section of this paper describes a series of comparative accident studies, conducted over a 2-yr period at 75 entrance and exit ramps (20 left-hand and 55 right-hand ramps) on urban expressways in the Chicago area. The studies include analyses of the simple frequency of occurrence of ramp accidents at left- and right-hand ramps, of the characteristics of these accidents, of the factors contributing to them and of the effect of different ramp configurations on the pattern of mainstream accidents.

Selection and Description of Study Locations and Sources of Data

A total of 20 left-hand (16 entrances and 4 exits) and 55 right-hand ramps (26 entrances and 29 exits) were selected for study (Fig. 23). To reduce errors resulting from differential levels of accident reporting, the selection of study locations was restricted to three intensively patrolled urban freeways in the Chicago area.

Of the 20 left-hand ramps selected for study, eight (four exits and four entrances) were elements of two internal diamond interchanges, located $1\frac{1}{2}$ mi apart on a six-lane section of the Eisenhower Expressway in west suburban Chicago. Another isolated left-hand entrance ramp was located on the eight-lane section of the Kennedy Expressway, some 8 mi northwest of the Loop. This was the only left-hand entrance

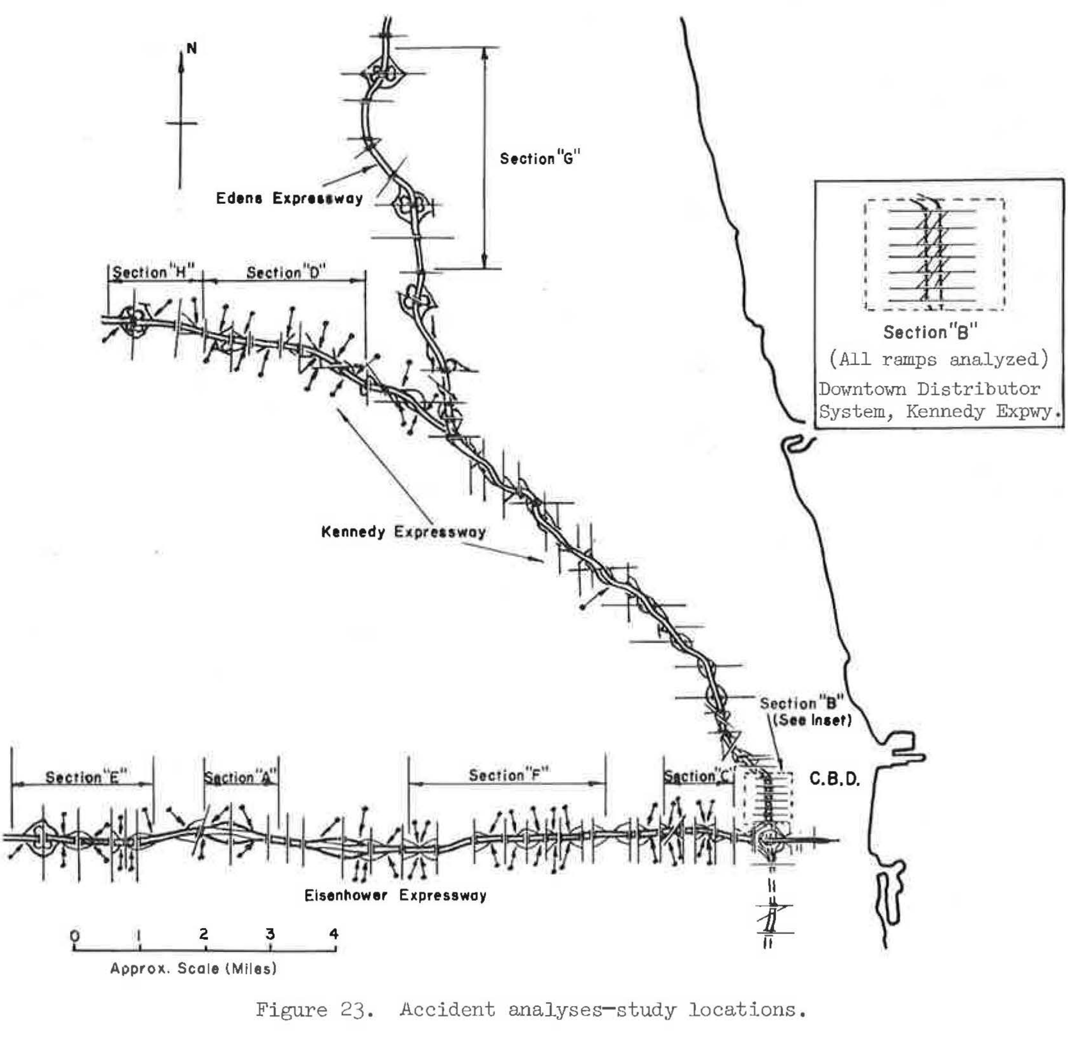


Figure 23. Accident analyses—study locations.

studied whose approach to the freeway was located on an upgrade. The remaining 11 left-hand ramps (all entrance ramps) composed part of a compact downtown distributor system leading from the Kennedy Expressway immediately to the west of the Loop. These 11 entrance ramps, together with a parallel set of 11 right-hand exits, provide the main distributory system linking the expressway with the downtown street network. The entire system occupies slightly less than 0.8 mi of expressway.

The general design and volume characteristics of the 20 left-hand ramps differed considerably. The two internal diamond interchanges and the single isolated left-hand entrance ramp were all designed to high standards (Table 1). The minimum acceleration lane length was 800 ft and minimum deceleration length 450 ft. Ramp grades were approximately ±3 percent and ramp widths 16 to 18 ft. Adequate directional signing was provided at all locations. The 11 downtown left-hand entrance ramps, in contrast, were substandard in design. They were provided with very short acceleration lanes (350 ft), had extremely poor sight lines and, as already mentioned, were "nested" together into a compact ramp system that paralleled a similar compact sequence of right-hand exits. All of the 11 ramps were single-lane designs with grades of -3 percent. Average weekday daily traffic (AWDT) volumes at the various locations varied from 1,200 to 10,700 veh on the entrance ramps, and from 4,300 to 10,700 veh on the exit ramps. The adjacent freeway sections carried between 48,500 and 54,800 AWDT (three-lane sections) and between 63,600 and 91,600 AWDT (four-lane sections).

In selecting the sample of right-hand ramps for comparison, it was not possible to choose locations which reproduced exactly the characteristics of the left-hand ramps described. Instead, a sample of right-hand ramps was selected that encompassed, as far as possible, the entire range of volume and design characteristics to be found at the left-hand ramps.

As in the case of the left-hand ramps, the sample of right-hand locations ranged from high standard designs with 1,000-ft speed-change lanes to substandard facilities spaced very closely together. Of the 26 entrance ramps studied, nine had parallel acceleration lanes, eight had direct taper designs and nine had acceleration lanes which were continuous with the deceleration lane of an adjacent off-ramp. Of the 29 exit ramps, 21 had direct taper deceleration lanes and eight were continuous lane designs [illegible] and from 300 to 850 ft for deceleration lanes. Ramp gradients varied from +3 to -3 percent (all but one of the entrance ramps were either level or on a downgrade and all but two of the exit ramps were on an upgrade). Ramp widths varied from 14 to 18 ft; all the ramps studied were again single-lane designs. Approximately 50 percent of both entrance and exit ramps were located on eight-lane sections of freeway, and 50 percent on six-lane sections. All but three of the 55 ramps were elements of diamond interchanges. Traffic volumes again varied considerably: from 1,300 to 12,000 AWDT on the entrance ramps, from 1,100 to 13,800 AWDT on the exit ramps, and from 53,600 to 60,900 (three-lane) and 54,800 to 84,400 (four-lane) on the adjacent freeway sections.

Data for the accident analyses were obtained from copies of original police accident reports, filed by the City of Chicago Police Department and the Illinois State Highway Patrol for the 2-yr period from January 1962 to December 1963. Further general data were obtained from a series of punched card summaries of individual freeway accidents, prepared for Illinois by the Chicago Area Transportation Study. Twenty-four-hour volume data for 1961 and 1963 were provided for each location by the Illinois Division of Highways. These data were corrected to yield average, 24-hr weekday volume figures for 1962 and 1963 for each ramp and for each adjacent freeway section. Approximate annual volumes were then computed by multiplying these 24-hr figures by 340 to allow for weekends and holidays (7).

Analysis Techniques

The annual number of accidents occurring both on the ramp itself and also within the ¼-mi section of freeway immediately upstream of and adjacent to the ramp's speed-change lane were computed separately for each study location for 1962 and

1963. These computations yielded a total of 32 "ramp-years" of data for the left-hand entrance ramps and 8 "ramp-years" for left-hand exit ramps. Ramp accidents were defined for the purposes of this study as all those accidents which occurred either on the ramp itself, on the speed-change lane, or that involved vehicles in the act of either entering or leaving a speed-change lane. They were further subclassified according to whether they occurred at the upper or lower terminal of the ramp or on the body of the ramp proper.

A final series of analyses were made to determine the annual number of accidents occurring within each of the eight "sections" of freeway defined in Figure 23.

Each set of data was broken down according to the severity of the individual accidents, the type of collisions, and the class and number of vehicles involved. Data on the major factors which contributed to the cause of each accident (such as lane changing, failure to yield right-of-way, and speed) were unfortunately not reported in sufficient detail to warrant any form of rigorous analysis.

Annual ramp and freeway accident rates were computed for each location, using six different "exposure indices" as denominators:

$$\text{Exposure index 1} = \frac{\text{Avg. annual ramp volume}}{\text{Avg. annual freeway volume}};$$

$$\text{Exposure index 2} = \frac{\text{Avg. annual ramp volume}}{\text{Avg. annual lane volume on freeway}};$$

$$\text{Exposure index 3} = \frac{\text{Avg. annual ramp volume}}{\text{Avg. annual merge volume}};$$

$$\text{Exposure index 4} = \frac{\text{Avg. annual ramp volume} \times \text{avg. annual freeway volume}}{\text{No. of freeway lanes}};$$

$$\text{Exposure index 5} = \frac{\text{Avg. annual freeway volume}}{\text{No. of freeway lanes}} + \frac{1}{\text{freeway volume}}; \text{ and}$$

$$\text{Exposure index 6} = \frac{\text{Peak hour ramp volume}}{\text{Avg. peak hour lane volume on freeway}}.$$

None of these relatively complicated exposure indices were any more significantly related to accident occurrence than were simple stratified combinations of average daily ramp volume and average daily freeway volume. For this reason, the following simple volume classification system was adopted for the purposes of statistical analysis:

Freeway Volume—Low, 45,000 AWDT; medium, 45,000-70,000 AWDT; and high, 70,000 AWDT.

Ramp Volume—Low, 4,000 AWDT; medium, 4,000-8,000 AWDT; and high, 8,000 AWDT.

Detailed ramp accident rates were not computed for the 11 left-hand entrance ramps on Kennedy Expressway for 1962 because of the considerable fluctuations which occurred during that period in individual ramp volumes.

A number of serious problems arose in the analysis of the accident data because of inaccuracies or incompleteness in the original accident reports. Of these, by far the most serious was that of accurately locating the point of occurrence of an accident on the freeway. Very few of the original reports specified locations to an accuracy of more than half a block length (i.e., approximately 100 to 125 yd) and most were considerably less accurate. It was possible, however, to determine from the information in the original accident report whether a ramp accident occurred at the upper or lower terminal of the ramp or on the body of the ramp itself. In the case of freeway accidents, such a procedure was not feasible and, therefore, the analysis was restricted to a general study of the number and general characteristics of all accidents occurring over an approximate two-block length (i.e., approximately ¼ mi) upstream of each ramp.

Detailed Ramp Accident Studies. —Average corrected annual ramp accident rates for the left-hand and right-hand ramps studied are given in Table 6, classified according to the three levels of ramp and freeway volume defined previously:

Corrected annual ramp accident rate = Annual No. ramp accidents per millions of ramp vehicles per year

In every case (for both entrance and exit ramps) the average corrected accident rate at the left-hand ramps exceeded that at the right-hand ramps carrying equivalent traffic volumes. Averaged over all volume levels, the respective rates were 1.55 for left-hand vs 0.97 for right-hand entrance ramps (60 percent higher) and 1.52 vs 0.80 for left-hand and right-hand exits (90 percent higher). For each ramp volume/freeway volume combination for which data on both left- and right-hand ramps were available, a student's "t" test was run to determine whether any significant difference in average corrected accident rates existed between left- and right-hand designs. Table 6 summarizes the results of these statistical tests.

Of the 17 combinations for which comparisons were possible, seven showed differences that were significant at the 20 percent level or higher (five significant at the 10 percent level and three at the 5 percent level). The remaining ten combinations showed no significant difference in average accident rates, although, as previously noted, the rates at the left-hand ramps were consistently higher than those at the comparable right-hand ramps. It is also interesting to note that entrance-ramp accident rates, though by no means consistently higher, tended to be slightly greater than comparable exit-ramp accident rates. This latter statement applies equally to both left- and right-hand ramps. The incompleteness of the matrix in Table 6 precluded the performance of any more rigorous statistical tests.

Figure 24 illustrates some of the major characteristics of ramp accidents observed at the sample of right- and left-hand ramps. In terms of accident severity, it would appear that apart from a slightly higher proportion of personal injury accidents at left-hand exits, there was virtually no difference between the left- and right-hand entrance ramps studied. This increase in personal injury accidents at left-hand exits was an increased proportion of sideswipe accidents (39 percent vs 18 percent for entrance ramps, 24 percent vs 11 percent for exit ramps). Only in the case of the entrance ramps, however, was this difference statistically significant (at the 10 percent

TABLE 6

AVERAGE ANNUAL RAMP ACCIDENT RATES PER MILLION RAMP VEHICLES FOR RIGHT- AND LEFT-HAND ENTRANCE AND EXIT RAMPS

Ramp Vol. (AWDT)	Accident Data	Accident Rate															
		<45,000 AWDT				45,000-70,000 AWDT				>70,000 AWDT				All Volumes			
		On-Ramps		Off-Ramps		On-Ramps		Off-Ramps		On-Ramps		Off-Ramps		On-Ramps		Off-Ramps	
		Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
<4,000	Accident rate	—	0.84	—	0	1.32	1.11	—	1.22	2.13	1.09	—	0	1.85	1.05	—	0.94
	sample, RH vs LH	0	4	0	4	3	14	0	17	6	4	0	1	9	22	0	22
						No sig. diff.				Diff. sig. at 20% level				No sig. diff.			
4,000-8,000	Accident rate	—	—	—	0.60	2.33	0.59	0.97	0.74	0.93	0.86		0.60	1.43	0.77	0.07	0.64
	sample, RH vs LH	0	0	0	5	3	12	4	10	5	6	0	7	8	18	4	22
						Diff. sig. at 5% level		No sig. diff.		No sig. diff.				Diff. sig. at 10% level		No sig. diff.	
>8,000	Accident rate					1.34	1.19	2.17	0.85	—	1.15	—	1.04	1.34	1.17	2.17	0.92
	sample, RH vs LH	0	0	0	0	4	8	4	9	0	4	0	5	4	12	4	14
						No sig. diff.		Diff. sig. at 5% level						No sig. diff.		Diff. sig. at 5% level	
All Volumes	Accident rate	—	0.84	—	0.33	1.60	0.97	1.52	1.00	1.59	1.01	—	0.72	1.55	0.97	1.52	0.80
	sample, RH vs LH	0	4	0	9	10	34	8	36	11	14	0	13	21	52	8	58
						Diff. sig. at 10% level		No sig. diff.		No sig. diff.				Diff. sig. at 20% level		No sig. diff.	

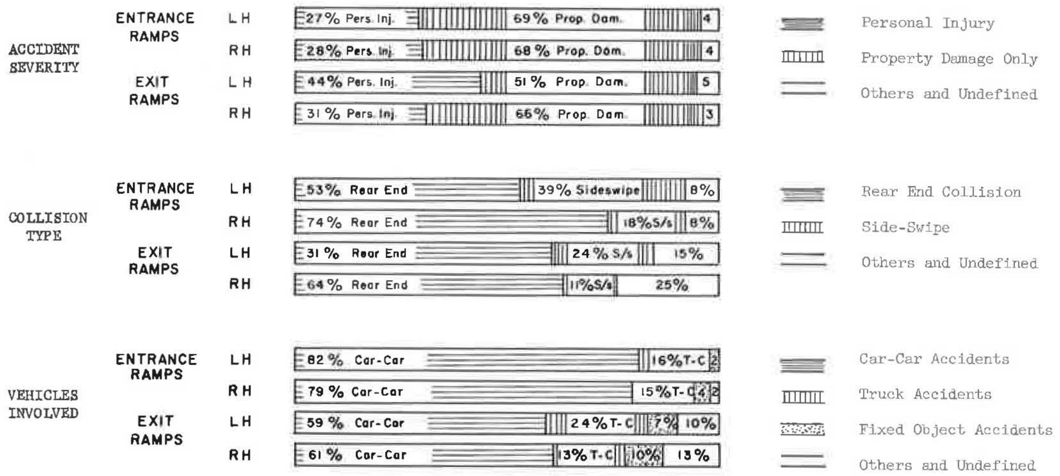


Figure 24. Characteristics of accidents at left- and right-hand ramps in Chicago area.

level). At all ramps, the major proportion of ramp accidents involved car-car collisions, with a slightly higher proportion of truck accidents at left-hand exit ramps and a higher percent of fixed object accidents at all exit ramps. Neither of these differences was significant at the 10 percent level.

A comparison of the factors contributing to the occurrence of accidents yielded a wide dispersion of results. At all ramps, whether left- or right-hand, it would appear that such features as acceleration or deceleration lane length, ramp width, ramp grade, ramp and freeway alignment and sight distance had a considerable effect on the occurrence of accidents. In no case, however, could a significant relationship be developed between any one of these variables and accident rate. A simple multiple correlation analysis indicated that, apart from volume, no one individual characteristic had a predominant effect on accident rate, and even the influence of volume was apparently subordinate to the combined effects of the other characteristics. No significant relationships could be developed between accident rate and either freeway or ramp volume level. It should be borne in mind, however, that the sample of ramps studied was somewhat small for this type of analysis and that this fact may account partially for the lack of significant results.

Studies of Mainstream Accidents Upstream of Left- and Right-Hand Entrance and Exit Ramps. —Studies also were made of the mainstream accidents occurring in the ¼-mi section of freeway immediately upstream of each entrance and exit ramp. In the case of entrance ramps, the ¼-mi section was measured from the end of the acceleration lane, and in the case of exit ramps from the ramp gore. As in the case of the ramp accident analyses, there was again no significant relationship between ramp volume, freeway volume or merge volume and accident rate. A similar series of statistical tests to those carried out on the ramp accident data indicated that for all combinations of ramp and freeway volume, there was no significant difference between the average corrected upstream accident rates at right- and left-hand terminals. There were, however, slight increases in the average severity, frequency of sideswipes and number of weaving accidents upstream of left-hand entrances and exits, as compared to right-hand entrances and exits. None of these differences was statistically significant at the 20 percent level.

In most cases it was apparent that a high upstream accident rate was dependent not as much on the type of ramp as on the alignment and profile of the through lanes. At the Austin Blvd. WB ramps, for example, the width of the freeway drops from four lanes to three, the decrease effected by the termination of the right-hand lane exactly opposite the Austin Blvd. left-hand exit. This decrease in freeway width has a far greater effect on the upstream accident rate than does the presence of the left-hand exit.

TABLE 7

SECTIONAL STUDIES—ANNUAL ACCIDENTS/MVM FOR FREEWAY SECTIONS CONTAINING DIFFERENT COMBINATIONS OF RIGHT- AND LEFT-HAND RAMPS

Section	No. Lanes	Length (mi)	No. Ramps	Ramps/ Mi	Ann. Acc./MVM	
					1962	1963
A, Eisenhower Expressway	6	1.0	2 LH on, 2 LH off, all direct ramps.	4.0 (100% left)	2.11	2.20
B, Kennedy Expressway, 400 S-299 N	8	0.9	11 LH on, 11 RH off, all direct ramps.	24.6 (50% left)	5.82	6.98
C, Eisenhower Expressway, 1100 W-1949 W	8	1.0	5 RH on, 4 RH off, all direct ramps.	9.0 (all right)	3.51	4.84
D, Kennedy Expressway, 5100 N-7500 W	6	2.2	7 RH on, 7 RH off, direct; 1 RH on, 2 RH off, loops.	6.4 (all right)	1.76	1.68
E, Eisenhower Expressway, 8200 W-9899 W	6	2.1	7 RH on, 7 RH off, direct; 1 RH on, 2 RH off, loops.	8.1 (all right)	4.10	3.21
F, Eisenhower Expressway, 2700 W-5099 W	8	2.9	8 RH on, 8 RH off, all direct ramps.	5.5 (all right)	2.70	2.15
G, Edens Expressway, 6400 N-8999 N	6	3.2	2 full clover leaves.	5.0 (all right)	1.70	2.39
H, Kennedy Expressway, 7500 W-9000 W	6	1.4	1 RH on, 1 RH off, direct; 1 full clover leaf.	7.1 (all right)	0.89	1.32

Sectional Accident Studies.—A series of eight sections of freeway, varying in length from 0.8 to 3.0 mi, were chosen (Fig. 17) on the Edens, Kennedy and Eisenhower Expressways, containing different combinations of left- and right-hand entrance and exit ramps. For each section, the total number of accidents occurring during 1962 and 1963 were computed and these totals then converted into annual accident rates per million vehicle miles (MVM). Table 7 summarizes the results of these studies; Figure 25 illustrates the monthly variation in accident rates for each of the sections between January and December 1963. From these analyses, it is apparent that the distribution of monthly accident rates along section A, containing the Harlem Ave. internal diamond interchange on the Eisenhower Expressway (but excluding the Austin Blvd. interchange where the width of the adjacent freeway section changes from four to three lanes), did not differ significantly from the average rates observed along sections D, E, F, G and H, all of which contained various combinations of right-hand ramps.

Section B, the downtown section of Kennedy Expressway containing the complex distributor system of left-hand entrances and right-hand exits, had a significantly higher distribution of accident rates than any of the other seven sections studied. However, the accident rate for section B is by no means attributable solely to the presence of the left-hand entrance ramps. The section carries consistently heavy volumes of traffic with substantial weaving and lane-changing movements and is also subjected to congestion backing up from downstream locations in both directions of travel.

The correction of the data, illustrated in Figure 21, to allow for variations in average ramp spacing within the eight sections studied resulted in a much more compact set of monthly accident distributions. In particular, the average monthly accident rate for section B dropped from 6.98 to 0.32 acc./MVM/ramp. Similar, but less

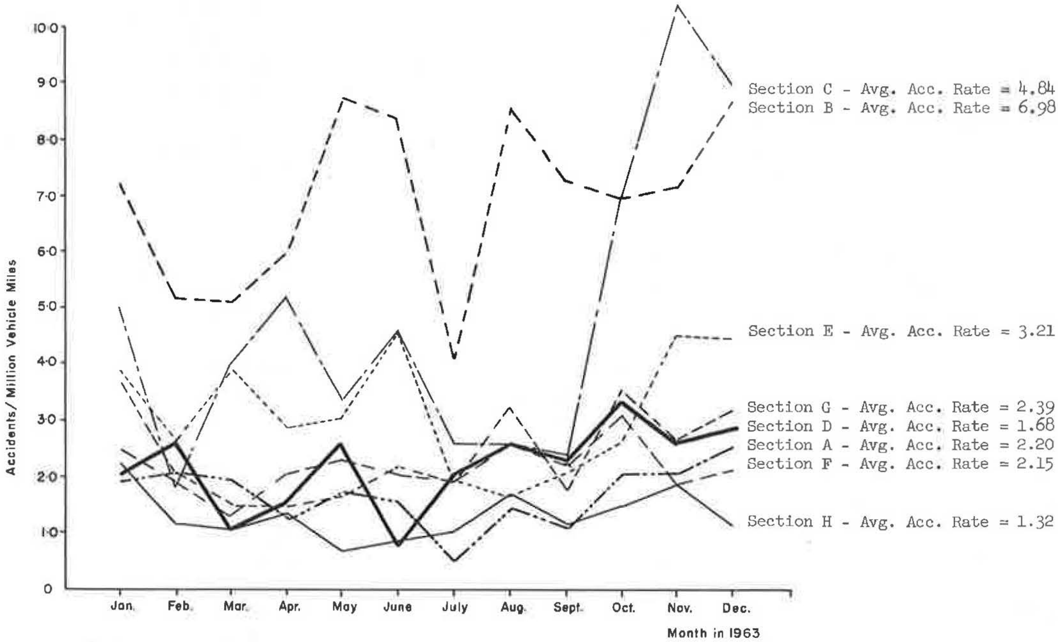


Figure 25. Monthly variations in sectional accident rates.

pronounced reductions occurred for the other seven sections. A comparison of the resultant distributions of monthly accident rates per MVM per ramp indicated that none of these corrected distributions differed significantly at the 10 percent level.

CONCLUSIONS

On the basis of the studies described in this paper, the following conclusions may be drawn concerning the operation of left-hand ramps located on six-lane tangent sections of depressed urban freeway situated in level terrain in the Chicago area. The ramps studied carried average AWDT volumes of 1,300 to 10,700 veh. (10 percent trucks) and the adjacent freeway sections AWDT volumes of 100,000 to 120,000 veh. (9 percent trucks).

1. None of the left-hand ramps studied caused any prolonged disruption of flow in the adjacent freeway lanes.
2. The distributions by lane of average-minute-volumes, average-minute-speeds, and individual time headways on the approach to a left-hand entrance forming part of an internal diamond interchange were significantly different from those observed on the approach to a comparable right-hand entrance carrying similar traffic volumes and forming part of an external diamond interchange.
3. Average merging speeds and the speed differentials between entering and through vehicles were higher at left-hand entrance ramps than at right-hand entrance ramps.
4. There was a higher incidence of mainstream lane changing in the vicinity of the left-hand entrance ramps studied than at the comparable right-hand entrances.
5. No significant difference was observed in the incidence of hazardous maneuvers at right- and left-hand entrance ramps.
6. A single low-volume left-hand entrance ramp provided with a 900-ft acceleration lane had apparently little effect on traffic behavior in the adjacent freeway section.
7. Extremely high merging rates, in excess of 1,800 vph, were observed at a left-hand entrance ramp provided with a 1,075-ft acceleration lane on a level three-lane section of depressed freeway without causing average left-lane speeds upstream of the ramp to fall below 45 mph. A 50-min average merge rate of 2,034 vph was observed

at the same entrance ramp without a prolonged drop in speeds being observed in the left lane upstream of the ramp. Periodic 1-min merge rates in excess of 2,400 vph caused brief slowdowns upstream of the ramp, but at no time caused complete breakdowns in flow. Throughout the study period the flow in the left lane approaching the ramp averaged 1,400 vph.

8. A prolonged merge rate in excess of 1,800 vph at a left-hand entrance resulted in a supersaturated flow condition in the left lane downstream of the entrance ramp. In this condition, the flow in the left lane was extremely sensitive to even small disturbances.

9. Approximately 60 percent of all vehicles entering a three-lane section of freeway via a left-hand entrance ramp were still positioned in the extreme left-hand lane at a point 2,050 ft downstream of the ramp nose. Approximately 50 percent of the vehicles still remained in the left lane at a point 6,100 ft downstream.

10. Over 70 percent of all exiting vehicles were already positioned in the left lane at a point 7,250 ft upstream of a left-hand exit ramp. The first directional sign for this ramp was located 2 mi upstream of the ramp nose.

11. Variations in total freeway volume appeared to have relatively little effect on the lateral placement of ramp vehicles upstream and downstream of left-hand entrances and exits.

12. On level sections of heavily traveled urban freeway in the Chicago area, the average reported accident rate per million ramp vehicles (MRV) was consistently higher at left-hand entrances and exits than at right-hand entrances and exits. At right-hand entrances, the average ramp accident rate was 0.97 acc./MRV, vs 1.55 at left-hand entrances; at right-hand exits the equivalent rate was 0.80 compared with 1.52 at left-hand exits. The differences in average accident rates, both overall and grouped according to ramp/freeway volume combinations, were not consistently significant at the 20 percent level.

13. At both left- and right-hand ramps, the absolute number of ramp accidents increased with increases in ramp volume. In neither case, however, could a simple relationship be found between accident rate and volume. Individual design characteristics appeared to have a more significant effect on the accident [illegible]

14. There was no significant difference in the average severity of accidents occurring at left- and right-hand entrance and exit ramps.

15. There was no significant difference in the through lane accident rates upstream of left- and right-hand entrance and exit ramps.

16. Average accident rates tended to be slightly though not consistently higher at entrance ramps than at equivalent exit ramps. This statement applies to both left- and right-hand ramps.

17. With the exception of a 0.8-mi section of downtown distributor freeway containing 11 left-hand entrance ramps and 11 right-hand exit ramps (total for both directions), there was no significant difference between the distribution of monthly accident rates along sections of freeway containing primarily left-hand ramps and along sections containing exclusively right-hand ramps. For normal spacings (i.e., less than eight ramps per mile in both directions), the average spacing of ramps within the study section did not have any significant effect on the overall accident rate.

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Freeway Pedestrian Accidents: 1958-1962

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•FREEWAYS are not designed to accommodate pedestrians and for this reason most freeway ramps are posted to inform pedestrians that they are prohibited from entering. Dismounted vehicle occupants, persons who drive onto the freeway and dismount from their vehicle for some reason, are not specifically prohibited from walking along the freeway.

In addition, all freeways are fenced to prevent entry by pedestrians, animals, and vehicles. In urban areas, a 6-ft chain link fence is placed along the right-of-way. In rural areas, a 4-ft wire fence is used. Pedestrian barriers consisting of 4- or 6-ft chain link fence are often placed in the median within interchange areas to prevent pedestrians from crossing the freeways. A cable chain link median barrier, installed on approximately 150 mi of freeway, also serves as a continuous pedestrian barrier and 50 mi of blocked-out metal beam median barrier act as a lesser deterrent.

In spite of fences, signs, and barriers, there are still approximately 130 pedestrian accidents on freeways each year. Of these, approximately 55 are fatal, comprising 13 percent of all freeway fatal accidents.

STATISTICAL BREAKDOWN OF PEDESTRIAN ACCIDENTS

Freeway fatal accidents in California between 1958 and 1962 can be classified as indicated in Table 1. The Division of Highways is doing research on both cross-median and wrong-way accidents on freeways and the California Highway Patrol is studying

this seemed to be a type of accident for which specific preventatives could be devised.

Table 2 gives the number of pedestrian and total freeway accidents by severity for the 5 yr included in this study (1958-1962) and for 1963. The 416 pedestrian accidents were widely scattered throughout the freeway system with no locations having a concentration of pedestrian accidents.

Table 3 indicates why each pedestrian was on foot on the freeway and what he was doing when struck. Other tables, made to determine whether there were differences between urban and rural pedestrian accidents, indicate that the numbers of rural and urban pedestrian accidents are almost equal; they are distributed throughout the various classifications in Table 3 in a similar manner.

Table 4 summarizes the location of each pedestrian accident, regardless of why the pedestrians were on the freeway. It is obvious that one should stand as far away from the main stream of traffic as possible. Some pedestrians, such as those working on the freeway, have little control over where they stand. However, most pedestrians do have a choice and yet some stand on the traveled way.

TABLE 1

ACCIDENTS, 1958-1962

Type	% of Total
Single vehicle	50.5
Pedestrian	12.7
Head-on (cross-median and wrong-way)	15.8
Rear-end and sideswipe	21.0

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TABLE 2

SEVERITY OF PEDESTRIAN AND ALL FREEWAY ACCIDENTS

Year	No. Pedestrian Accidents				All Freeway Accidents			
	Fatal	Injury	PDO	Total	Fatal	Injury	PDO	Total
1958	32	20[a]	0	52	170	3,339	4,913	8,422
1959	18	14[a]	0	32	215	4,172	5,623	10,010
1960	36	47	0	83	259	5,902	7,871	14,032
1961	36	83	0	119	267	7,160	9,136	16,563
1962	51	79	0	130	390	9,081	11,350	20,821
Total	173	243	0	416	1,301	29,654	38,893	69,848
1963	55	115	0	170	400	10,511	13,756	24,667

[a]Does not include urban freeways.

TABLE 3

NUMBER OF FREEWAY PEDESTRIAN ACCIDENTS, 1958-1962

What Pedestrians Were Doing When Struck	Why Pedestrians Were on Freeway							
	Disabled Veh.	Prior Acc.	Working[a]	Trying to Cross	Hitch-hiking	Reason Unknown or not Stated	Total No.	Total %
Walking parallel to centerline on:								
Traveled way	1	1	0	0	2	11	15	3.5
Shoulder	5	0	0	0	0	1	6	1.4
Median	0	0	0	0	0	1	1	0.2
Ramp	3	0	0	0	5	5	13	3.1
Total	9	1	0	0	7	18	35	8.2
Standing on:								
Traveled way	18	25	15	0	0	9	67	16.2
Shoulder	10	9	15	0	2	2	38	9.2
Median	1	4	5	0	0	0	10	2.4
Ramp	5	4	3	0	1	4	17	4.1
Total	34	42	38	0	3	15	137	31.9
Working on vehicle on:								
Traveled way	38	0	0	0	0	0	38	9.2
Shoulder	29	0	0	0	0	0	29	7.1
Median	3	0	0	0	0	0	3	0.7
Ramp	0	1	0	0	0	0	1	0.2
Total	70	1	0	0	0	0	71	17.2
Crossing freeway	13	5	0	138	11	3	170	40.8
Unknown or not stated	2	2	0	0	1	3	8	1.9
Total								
No.	128	51	38	138	22	39	416	—
%	30.8	12.4	9.1	33.2	5.2	9.3	—	100.0

[a]Breakdown of those working on freeway:

Div. of highway personnel	10
Contractor's personnel	4
Police officers	21
Tow truck operators	3
Total	38

TABLE 4

LOCATION OF PEDESTRIANS WHEN STRUCK

Location	No. Ped. Acc.	% of Total
Traveled way	290	69.7
Shoulder	73	17.6
Ramp traveled way	26	6.2
Ramp shoulder	5	1.2
Median	14	3.4
Unknown	8	1.9
Total	416	100.0

WHY PEDESTRIANS WERE ON FREEWAY

Disabled Vehicles

Persons who dismounted from a disabled vehicle accounted for 30.8 percent of all pedestrian accidents. Quite often the drivers of these disabled vehicles parked on the traveled way rather than on the shoulder or median. Some freeway sections, such as viaducts and long bridges, do not have a shoulder or median, and disabled vehicles must park on the main traveled lanes. However, some drivers let their disabled vehicles coast to a stop and make no attempt to park in a safe place.

Once the vehicles come to a stop, the drivers and passengers generally do one of three things: (a) walk off the freeway to solicit assistance; (b) work on their vehicle; or (c) stand around and wonder what to do.

Working on a vehicle on or near the main traveled lanes is, or course, very hazardous. However, over one-half of the disabled vehicle operators were doing this when struck.

Prior Accident

Persons involved in a prior accident accounted for 12.4 percent of the pedestrian accidents. These persons very seldom walk off the freeway, nor do they work on their vehicles. They usually stand and wait for a police officer and tow truck or they try to flag traffic. The ones who stand on or near the main stream of traffic are more often struck by a vehicle than those who stand as far away from traffic as they can.

Working

Persons who are working on the freeway are there legally and are usually protected by signs, barriers, flashing lights, or flags. In spite of this protection, many workers are not very careful about where they stand. For instance, although police officers can stand almost any place to issue a citation or talk to motorists, many stand on the shoulder only a few inches from the main traveled lanes.

As indicated in Table 3, police officers constitute over one-half (21 of 38) of the workers involved in pedestrian accidents. Since these officers, as well as other workers, are necessary on freeways, it is unfortunate that some lose their lives, regardless of where the fault lies.

Trying to Cross Freeway

Of all pedestrians involved in accidents, more were on the freeway for the specific purpose of crossing than for any other reason (33.2 percent). There are many structures on freeways built especially so pedestrians can cross safely (pedestrian overcrossings and undercrossings). Pedestrians can also cross safely at most structures built for vehicle crossings.

To walk onto a freeway, pedestrians must climb a wire fence or walk along a ramp past a sign which informs them that they are prohibited from entering the freeway. Most pedestrians who cross a freeway know that they are violating the law and endangering their lives, yet they do it anyway.

Hitchhiking

It long been thought that hitchhikers constituted a major portion of pedestrian victims on freeways. However, they comprise only 5.2 percent of all pedestrian victims, and half of them were crossing the freeway and were not actually in the act of hitchhiking when struck. In fact, only 3 of the 22 hitchhikers were standing along the freeway when struck. One reason that hitchhikers are not struck very often may be that they stand off of the main traveled lanes and face oncoming traffic while they are actually hitchhiking.

WHAT PEDESTRIANS WERE DOING WHEN STRUCK

Walking Parallel to Centerline

People walking along the freeway comprise only 8.2 percent of the freeway pedestrian accidents. It is hard to believe that anyone would walk on the main traveled lanes (assuming a shoulder is available), yet it is done. Transients are seen walking along freeways and other roads quite frequently. Most of them probably do not know the difference between freeways, expressways, and other multilane roads, nor do they care.

Although hundreds of vehicles are disabled on freeways in California every day and hundreds of thousands of pedestrians, including disabled vehicle operators, walk along freeways during the course of a year, only six persons were struck in 5 yr while walking on the freeway shoulders in California. This is less than 2 percent of all freeway pedestrian accidents and it implies that walking to the nearest exit for professional aid is not nearly as hazardous as previously supposed.

Standing

Persons standing within the freeway right-of-way constituted 31.9 percent of all freeway pedestrian accidents. In 114 of 132 of these accidents, the pedestrians were on the freeway because their vehicles were disabled, they were involved in a prior accident, or they were working. These people were on the freeway for reasons over which they had no control. Most of them did have some control over where they stood, yet in 67 of 132 accidents they stood on the main traveled lanes.

An edge of pavement stripe and/or diagonal shoulder striping might help pedestrians to realize where they are standing.

Working on Vehicle

In 17.2 percent of the pedestrian accidents, the victim was working on his vehicle. Apparently a large number of people will work on a vehicle when it is obviously unsafe to do so.

An 8-ft shoulder with a dike or guardrail does not provide enough room to change a tire on the left side of the vehicle without the pedestrian encroaching on the main traveled lanes. To change a tire on the right, the vehicle must encroach on the traveled lanes to allow room between the vehicle and the dike or guardrail.

Crossing Freeway

One hundred seventy pedestrians were struck while actually crossing the freeway. Of these, 138 were on the freeway for the specific purpose of crossing. The remainder were crossing to or from their vehicles or were hitchhikers crossing the freeway.

TABLE 5

ACCIDENTS OF PEDESTRIANS CROSSING FREEWAY

Location of Accident	No. Ped. Acc.	% of Total
Interchange area	63	37.1
Between interchanges	105	61.7
Unknown	2	1.2
Total	170	100.0

Table 5 shows that 37 percent of the pedestrian accidents occurred within interchanges. Approximately 40 percent of all freeway mileage is within interchanges. Of the 63 pedestrian accidents which occurred within an interchange, 10 occurred at locations with a pedestrian barrier or deterrent in the median (Table 6).

High traffic volumes seem to act as pedestrian barriers. The ADT was not tabulated at each pedestrian accident,

TABLE 6

RELATION OF PEDESTRIAN ACCIDENTS TO BARRIERS AND DETERRENTS

Type	No. Ped. Acc.
Cable chain link median barrier	5
Double blocked-out metal beam barrier	4
48-in. chain link fence	1
72-in. chain link fence	0
None	53
Total	63

TABLE 7

LIGHTING CONDITION VS FREEWAY PEDESTRIAN ACCIDENTS

Condition	% of Total Acc.	
	Ped.	All
Daylight	30.5	52.2
Dusk or dawn	2.7	2.5
Dark:	66.8	45.3
No highway illumination	40.4	25.4
Highway illumination	26.4	19.9

but it appears that the higher volume freeways have a higher proportion of dismounted motorist accidents and that the lower volume freeways have a higher proportion of pedestrians crossing the freeway. The high volumes seem to act as a pedestrian barrier, although increasing the number of dismounted motorists.

MISCELLANEOUS FACTORS

Lighting

Table 7 presents the lighting condition at the time of the accident for pedestrian and for all freeway accidents. Two-thirds of all pedestrian accidents occurred at night. It is not known how much pedestrian activity there is on freeways at night as compared to daytime.

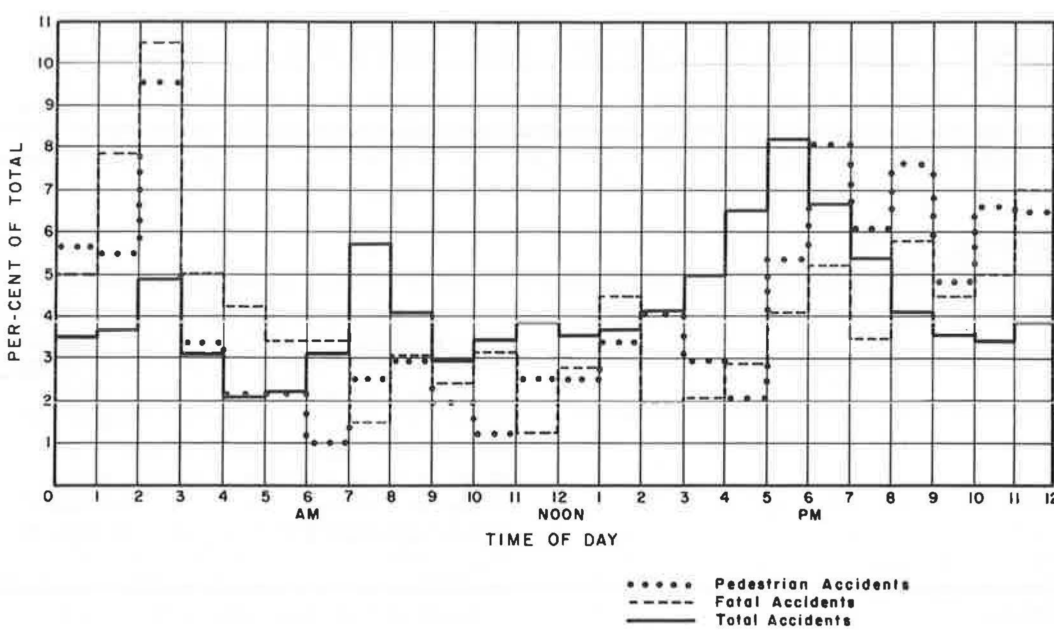

Figure 1. Hour of occurrence, pedestrian, fatal and all accidents, California freeways, 1958-1962.

TABLE 8

PEDESTRIAN ACCIDENTS ON VIADUCTS WITHOUT SHOULDERS

Freeway	No. of Ped. Acc. 1960	1961	1962	Total	MVM	Ped. Acc./100 MVM
Nimitz (Cypress St. to distr. struct.)	4	0	1	5	151	3.31
Central	0	0	0	0	106	0.00
Embarcadero	2	0	0	2	42	4.76
Total	6	0	1	7	299	2.34
Remainder of freeway system	77	119	129	325	36,069	0.90

Hour of Day

Figure 1 shows the relationship between pedestrian and all accidents during each hour of the day. Again, there is a greater frequency of pedestrian accidents at night, particularly between 6 PM and 3 AM.

Fifty-seven percent of the pedestrian accidents occurred between 8 PM and 6 AM, and 58 percent of the freeway fatal accidents occur during these hours. Therefore, pedestrian accidents do not account for the increase in freeway fatal accidents at night, but they contribute proportionally as much as the other types.

Viaducts

When a vehicle becomes disabled on a viaduct or long bridge without shoulders, the operator and the vehicle must remain on the main traveled lanes simply because there is no place else to go. Table 8 indicates the pedestrian accidents on some viaducts for a 3-yr period.

All pedestrians struck on the viaducts were disabled vehicle operators. Pedestrians rarely cross a viaduct, since they can walk underneath at almost any point. The viaducts had a higher rate of pedestrian accidents even though the only pedestrians on them were disabled vehicle occupants.

SUMMARY AND CONCLUSIONS

1. Approximately 130 pedestrian accidents occur on California freeways each year; of these 55 involved fatalities and 75 involved injuries.

2. Walking along the shoulder on a freeway is not nearly as hazardous as previously supposed. Only 6 persons were struck while walking along the freeway shoulder in 5 yr on all California freeways. During this same period, 8,813 pedestrians were struck in incroporated areas while crossing at signalized intersections with the green light, and 13,075 pedestrians were struck in incorporated areas while crossing at nonsignalized intersections.

3. Thirteen percent of all freeway fatal accidents involve a pedestrian.

4. Forty-three percent of all pedestrians struck are on the freeway because their vehicles are disabled or were involved in a prior accident.

5. Thirty-three percent of all pedestrians struck are on the freeway for the specific purpose of crossing the freeway.

6. Only 5.2 percent of the pedestrians struck are hitchhiking. It appears that present controls of hitchhiking or walking on freeways are effective and that further efforts in this regard would be relatively fruitless.

7. Of the remaining 18 percent, 9 percent were working on the freeway and it was undetermined why the other 9 percent were on the freeway.

8. Two-thirds of all pedestrian accidents on freeways occur during hours of darkness.

9. Seventy percent of the pedestrians are struck while on the main traveled lanes, 18 percent were on the shoulders, 7 percent were on ramps, 3 percent were in the median, and the position of 2 percent remains undermined.

10. Forty-two percent of freeway pedestrian accidents are fatal.