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Freeway Design and Operations

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Contents

Capacity Study of Signalized Diamond Interchanges

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This paper presents a portion of the results from a research project on freeway ramps and interchanges which is presently being conducted by the Texas Transportation Institute in cooperation with the Texas Highway Department. This study was designed to obtain traffic performance data which would have useful application in evaluating the capacity of signalized diamond interchanges. Library research and evaluation of existing design requirements indicated a great demand and need for this type of data.

The field data were gathered through the use of time-motion pictures which furnished a complete and simultaneous record of the traffic operations occurring in the intersections that were studied. The operational characteristics of over 4,000 vehicles were recorded on 16-mm film at two conventional-type diamond interchanges on the Gulf Freeway in Houston, Texas. These data were collected during the peak periods on approaches that were fully loaded.

This research was conducted to investigate the factors which could be used in developing criteria for evaluating the capacity of the various movements encountered in diamond interchange operation. Data on vehicle starting delays and time-headways at both interchanges were accurately measured to develop a basic approach to the determination of lane capacity. Special emphasis was placed on determining the capacity of a single-lane turning movement and a two-lane, or two-abreast-type turning movement.

The analysis of the study produced some significant results and provides the designer with current operational characteristics of vehicles at signalized diamond interchanges. Design procedures for diamond interchanges, based on lane capacity and signalization requirements, were developed and are presented in this report.

●IN URBAN AREAS traffic interchange between major arterials and freeways is frequently accomplished by the use of conventional-type diamond interchanges (Figs. 1 and 2). The simplicity of design, minimum right-of-way requirements, and economy of this type of interchange have greatly encouraged the use of diamond interchanges.

In contrast to simplicity of design the operation of a diamond interchange often becomes very complex. High volumes of cross-town major street traffic in combination with traffic interchanging to and from the freeway creates the need for a multiphase signal system to separate conflicting traffic streams. The signalization is further complicated by the proximity of the two at-grade intersections. These signalized intersections exert a capacity limitation which often results in the diamond interchange being incapable of handling the traffic demand.

Although the diamond interchange has many efficient applications, the operational

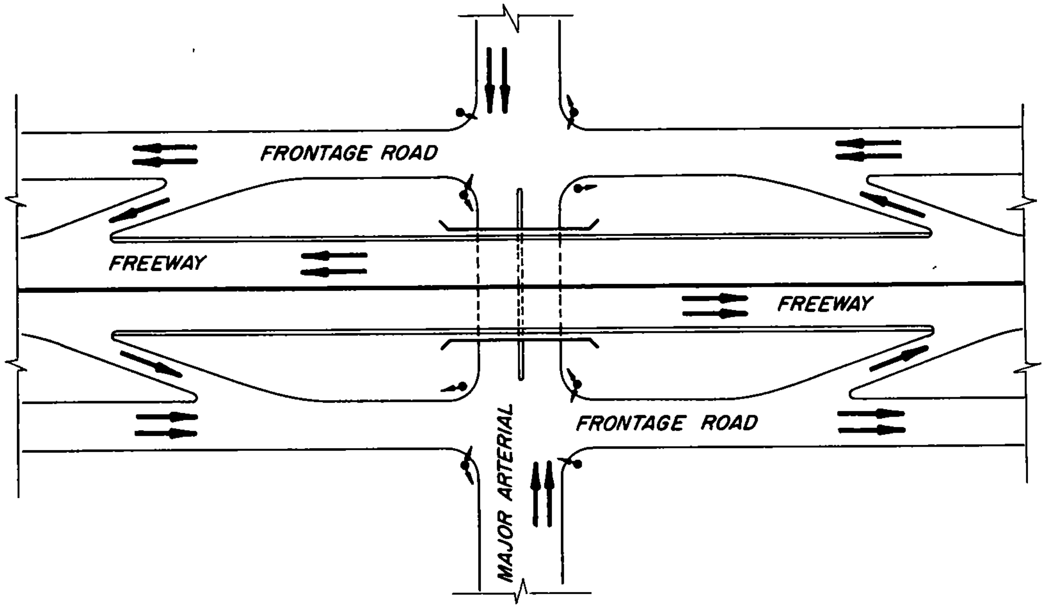


Figure 1. Diamond interchange conventional arrangement.

problems and capacity limitations of such an interchange must be recognized. The diamond interchange has been in existence for several years, but very little factual data on the operational characteristics and capacity of this type of interchange are available. The purpose of this study was to record data on the operational aspects of the diamond and to develop criteria for determining capacity, design, and signalization.

PROCEDURE

Sites

The Wayside Drive and Cullen Boulevard interchanges on the Gulf Freeway in Houston were selected for study sites. Both of these interchanges were conventional-type diamond interchanges and were known to carry large volumes of traffic (Fig. 3).

The Wayside interchange and the Cullen interchange (Figs. 4 and 5) had similar geometrics (Fig. 6). The major street approaches to both interchanges provided for two lanes of through traffic and a free right-turn lane. Each interchange was served by parallel, continuous one-way frontage roads which were 32 ft in width and operated with 3-lane flow during peak periods.

The traffic movements at each interchange were controlled by a fixed-time, multiphase signal system. The signal system used a three-dial operation with a separate dial provided to control the morning, the off-peak, and the evening peak periods of traffic flow.

At both interchanges, traffic operations at the two closely spaced intersections (300 ft center-to-center) were controlled by separate controllers which were interconnected for coordinated movement. Due to the lack of vehicle storage space between the intersections, the signal phasing (Fig. 7), as designed by engineers of the Department of Traffic and Transportation of the City of Houston, permitted vehicles to move through both intersections on receiving a green indication. With this phasing the only vehicles required to store between the intersections were a small percent of U-turning vehicles. The signal phasing also provided time separation for conflicting movements.

Figure 2. Gulf Freeway—Houston.

Collection of Field Data

Several methods of collecting data were evaluated to determine the best approach to the problem of obtaining a complete and simultaneous record of the traffic events occurring in the interchange area. The use of a 20-pen graphic recorder and manual counting methods were given consideration. However, in the final evaluation, the motion picture method was chosen as the best approach. This method required a minimum of field personnel and allowed the flexibility of enabling one to view and re-create all traffic events.

All of the traffic operational data were collected by filming traffic operations at each of the study intersections with a 16-mm motion picture camera. The filming was performed from a vantage point provided by a hydraulic platform truck similar to the one shown in Figure 8. The platform on this truck extended to a height of 35 ft and additional elevation was gained by taking advantage of the terrain. The truck was located in an inconspicious area and it is felt that the presence of the truck and photographer had little or no effect on the behavior of traffic in the intersection being filmed.

The movies at each study site were taken at a camera speed of ten frames per second which permitted accurate determination of vehicle time-headways and delay. Because it was desired to obtain data on possible capacity, the studies were conducted during both the morning and evening periods of peak flow on an average weekday.

As an aid to the determination of vehicle delays and time-headways from the motion pictures, reference lines were placed perpendicular to traffic lanes at each intersection approach. The purpose of these lines was twofold: (a) to regulate and fix the region where approaching vehicles would stop when waiting for a green indication; and (b) to aid in determining when each vehicle entered the intersection area.

Data Tabulation

Data on traffic operations were extracted from the film through the use of a specially constructed projector (Fig. 9). A special control attached to the projector permitted the film to be advanced or reversed in single-frame increments and an interconnected frame counter allowed the operator to determine the number of frames between specific events on the film. By using the constant camera speed, elapsed time between events could be determined.

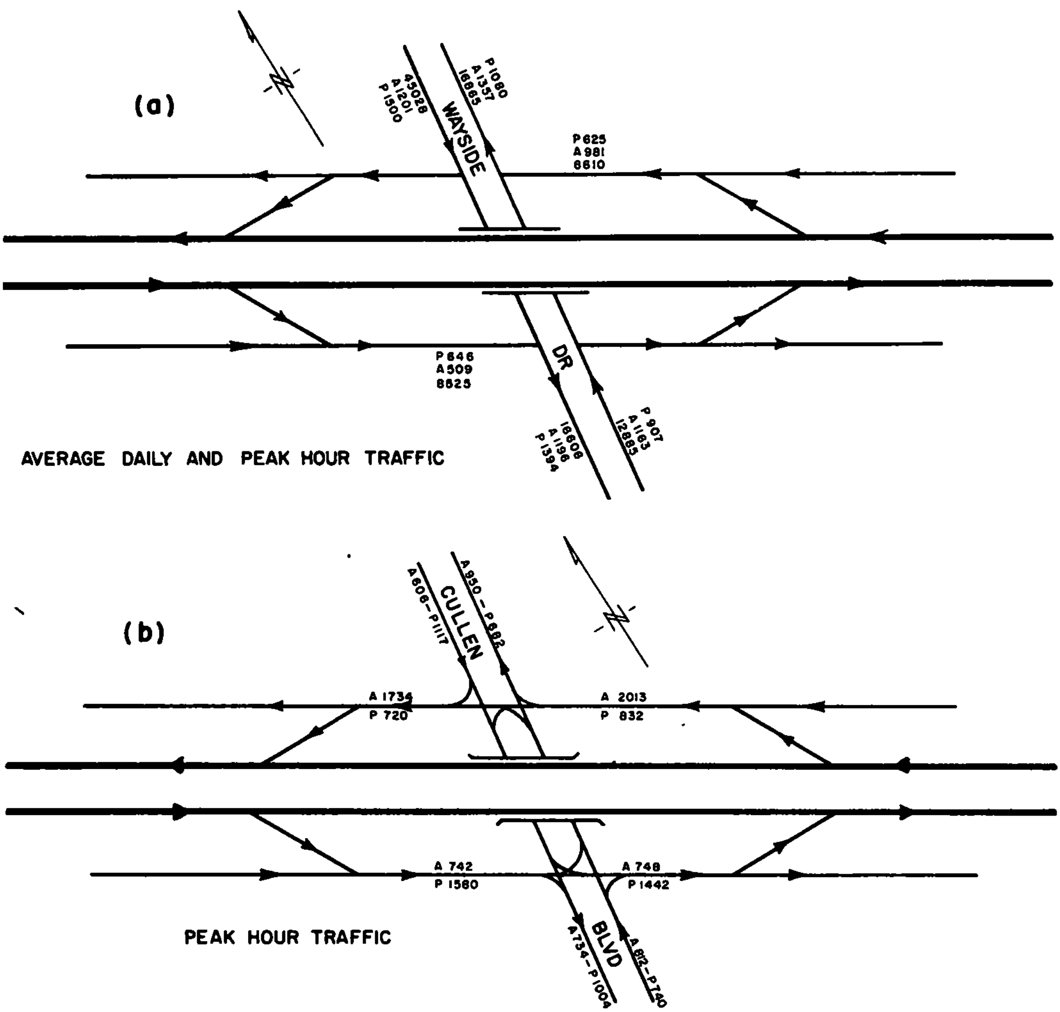


Figure 3. (a) I.H.-45 at Wayside Drive, Houston; (b) I.H.-45 at Cullen Blvd., Houston.

Figure 4. Wayside Drive interchange—Houston.

The traffic data were obtained on an individual lane basis and consisted of the following operational characteristics: (a) traffic volumes by composition and direction of movement; (b) starting delays after signal change to green by composition and direction of movement; and (c) time-headways between successive vehicles entering the intersection area by composition and direction of movement.

These data were recorded for each signal phase and notation was made to indicate if the approach lane was "loaded." An approach lane was considered loaded when the traffic demand was so great that vehicles were continuously entering the intersection throughout the green interval. The data on loaded intervals were then extracted and combined to provide information on peak flow conditions.

VEHICLE OPERATIONAL CHARACTERICTICS

In determining the number of vehicles that could clear a signalized diamond interchange in a given time period, a criterion was established by which vehicle operational

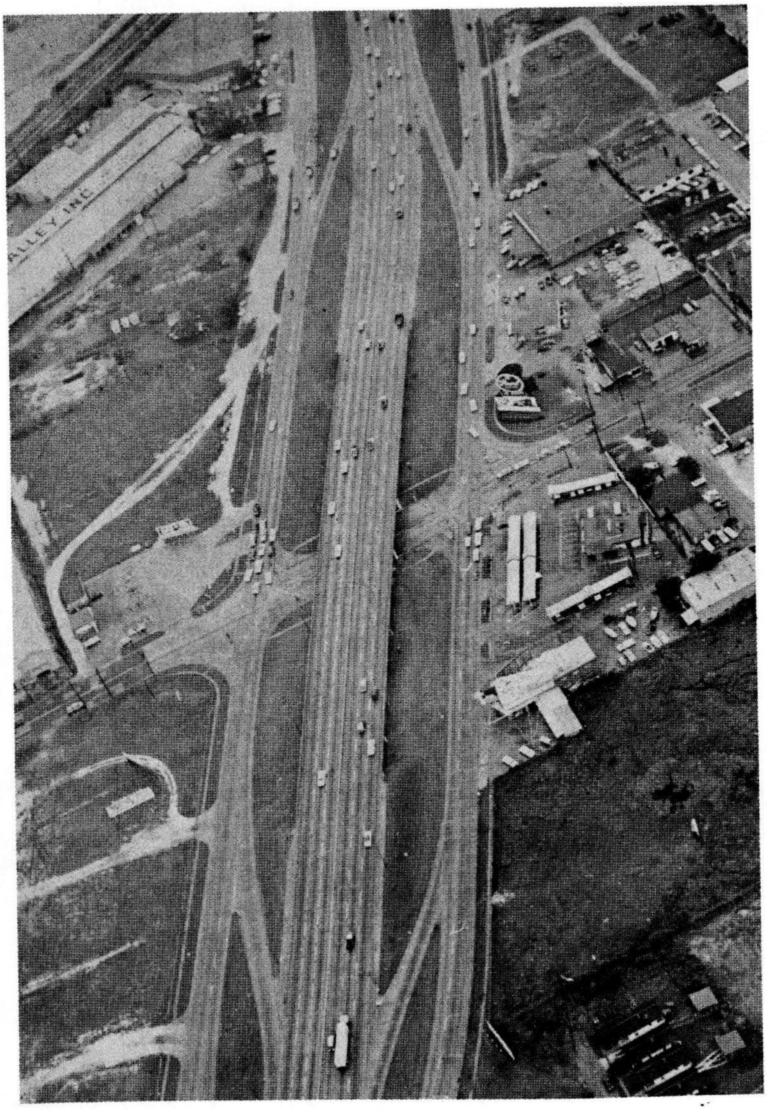


Figure 5. Cullen Blvd. interchange— Houston.

characteristics could be evaluated. This criterion involved a series of time measurements with special emphasis placed on the starting delays and time-headways of vehicles entering the interchange area. These measurements were defined as follows:

Time-Headway.—The time-headway between vehicles as they start from a stopped position one behind the other decreases progressively until they reach an average minimum (Fig. 10). Data from this study indicated that an average time-headway could best be obtained by averaging the time-headway values of the third through the last entering vehicle. It is true that this average value would be in error if an infinite number of vehicles were permitted to continue through an intersection without having to stop. However, the maximum number of vehicles that can every be expected to clear a signalized diamond interchange during one signal phase is approximately 10 to 12 vehicles per lane. This can be attributed to the fact that the signal cycle has to be proportioned to include other movements at the interchange.

Starting Delay. —When a traffic signal interrupts a flow of traffic, the vehicles stopped by a signal are delayed during the time the signal is red plus the time required for the vehicles to get started and underway again. This latter delay is commonly called starting delay. A generally accepted definition of starting delay is the time required for the first vehicle in a queue to commence motion and enter an intersection after the traffic signal displays a green indication. This time does represent a large portion

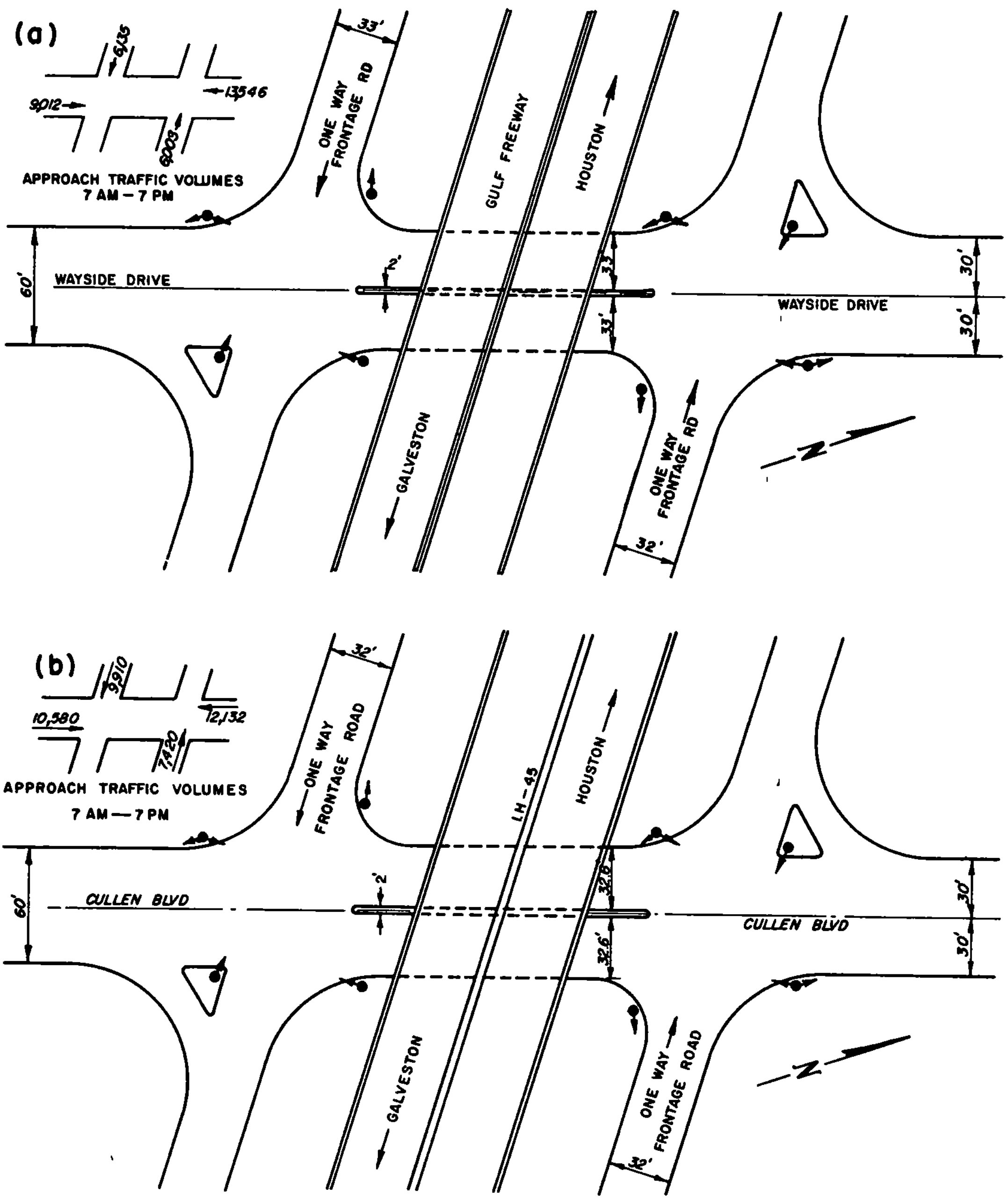


Figure 6. (a) Schematic layout—Gulf Freeway and Wayside Drive interchange Houston; (b) Schematic layout— Gulf Freeway and Cullen Blvd. interchange Houston.

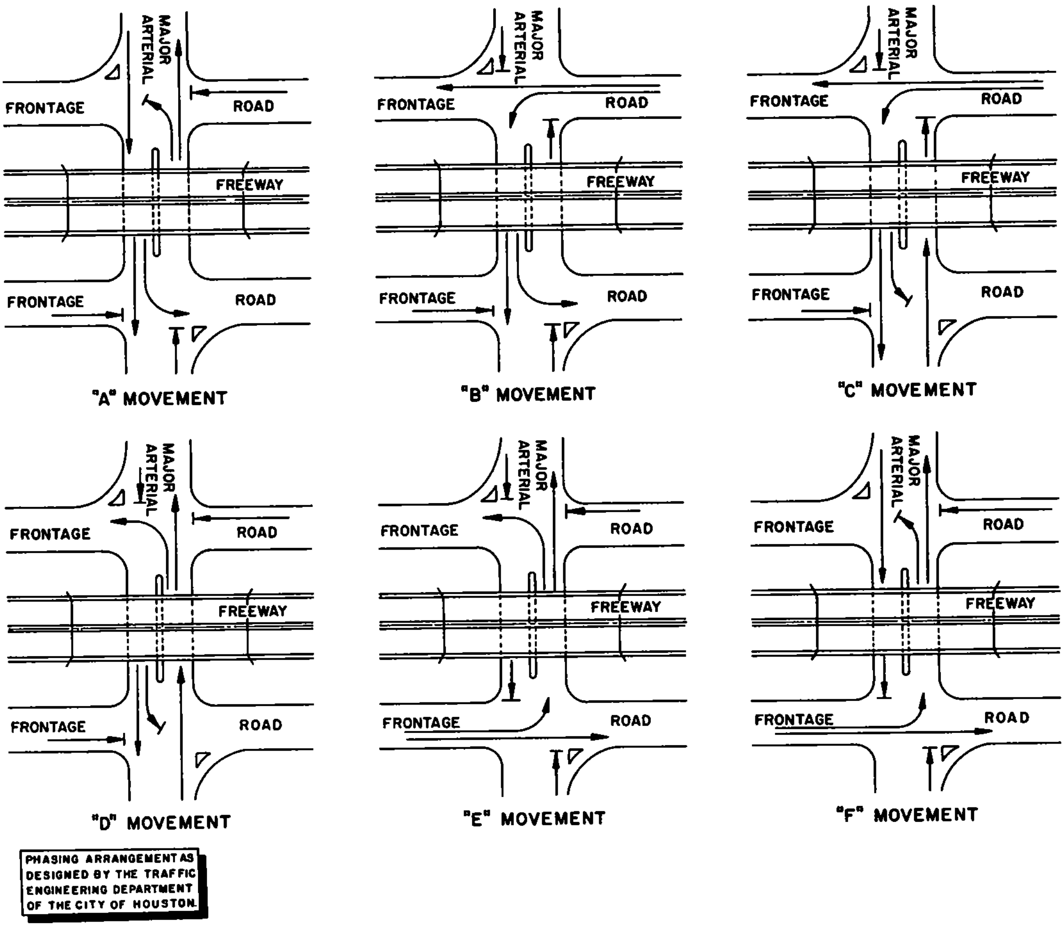


Figure 7. Phasing of traffic movements at a signalized diamond interchange.

of the starting delay experienced during each signal phase, but the operational studies showed that it does not represent the total time required for a line of vehicles to attain a reasonable degree of momentum. This is best illustrated by plotting the average time-headway values of a line of vehicles starting from a stopped position (Fig. 10). The time-headway values decrease rapidly for the first two vehicles in line with a lesser decrease for each succeeding vehicle. This indicates that the starting delay of a line of stopped vehicles can best be attributed to the reaction time and starting performance of the first two vehicles in line. In the evaluation of lane capacities for this report, the time required for the first two vehicles in a lane to enter the intersection was considered as the starting delay experienced at the beginning of each green phase.

Wayside Interchange

Starting delay and average time-headway measurements were determined for each lane approaching this interchange. The traffic on Wayside Drive consisted of two lanes of through movement with a free right-turn lane to accommodate the turning traffic. The frontage road traffic was considerably different with the predominant movements being both right and left turns onto Wayside Drive. Each of these movements was analyzed separately and are discussed as follows:

Wayside Drive Approaches.—The data gathered on each lane for the Wayside Drive approaches were combined because the traffic in each lane was similar and had no major differences in its operational characteristics. Figure 10 is a representation of the starting delay and time-headway measurements between successive passenger vehicles on these approaches. These observations yielded an average starting delay of 5.9 sec and an average time-headway of 2.2 sec.

North Frontage Road Approach.—Data were gathered on three different types of movements at this approach. These movements and their respective time-headways are shown in Figure 11a. The inside lane experienced a heavy left-turn movement, whereas the middle lane consisted of straight through movements. The outside lane had a combination of movements with 82 percent of the approach traffic turning right. The left turning movements from the inside lane and the right turning movements from the outside lane had identical operating characteristics with a 5.8-sec starting delay and a 2.1-sec average time-headway. The center lane, with a predominant straight through movement, was somewhat faster than the adjacent turning lanes with an average

Figure 8. Hydraulic platform truck.

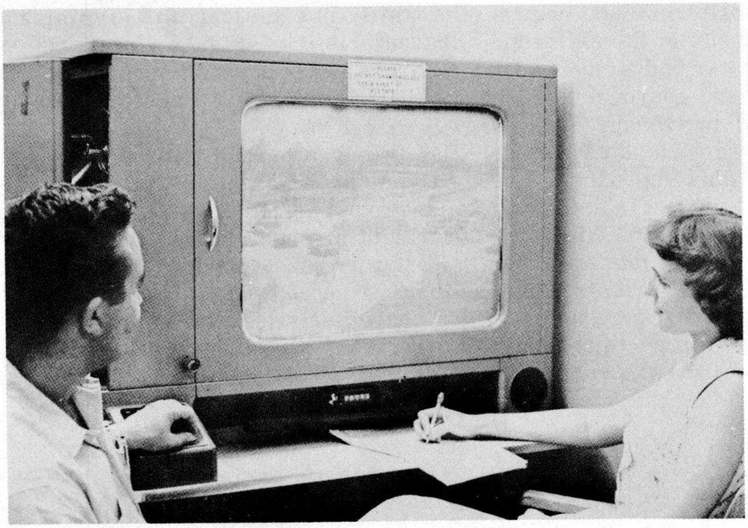


Figure 9. Richardson film projector.

time-headway value of 1.9 sec. There was no significant difference in the starting delay for each lane.

South Frontage Road Approach. —The most significant data obtained at this approach involved the double left, or two-abreast-type turning movement. The lane signing required the inside lane traffic to turn left and gave the center lane an option of turning left or proceeding straight. At this approach, 65 percent of the center lane traffic turned left in conjunction with 100 percent of the inside lane traffic. The performance

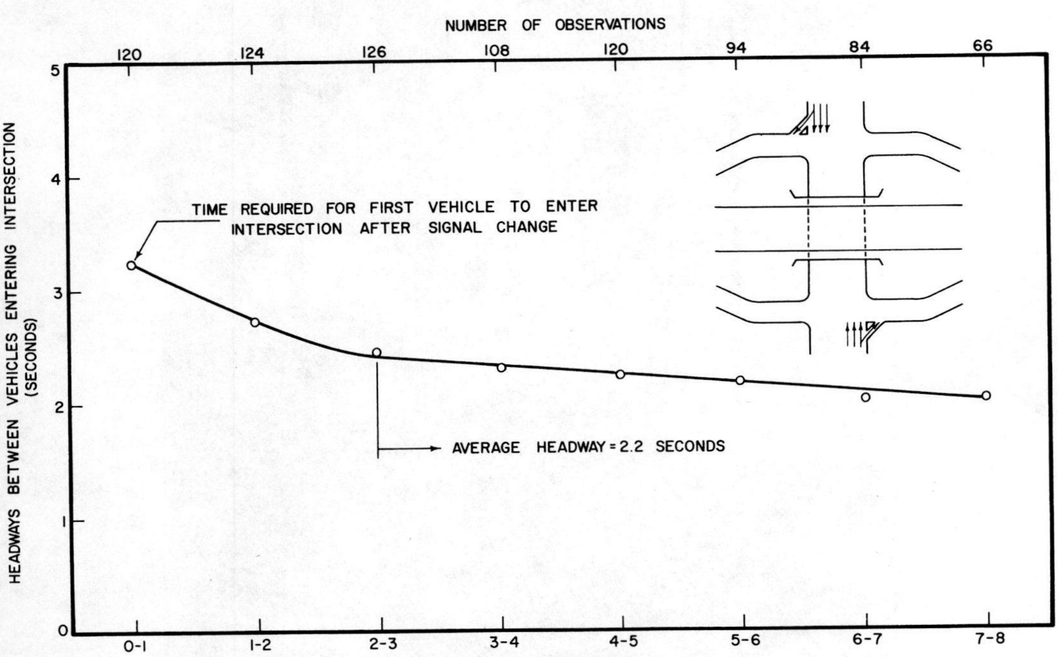
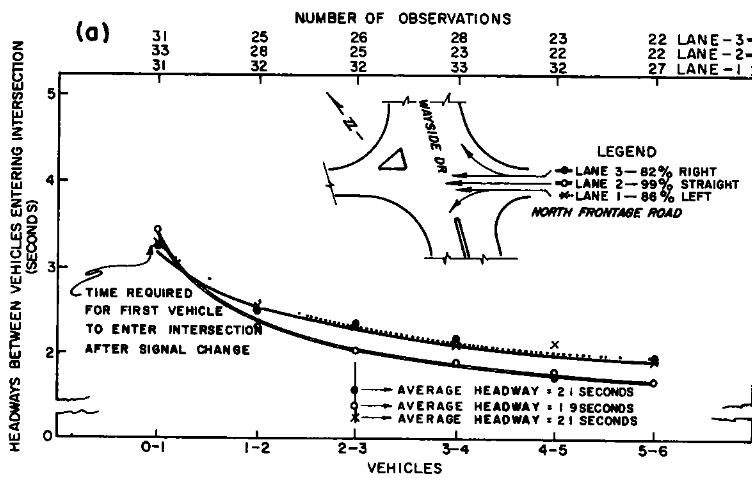


Figure 10. Time-headways between successive passenger vehicles on Wayside Drive at the Gulf Freeway and Wayside Drive interchange, Houston.

data of the traffic in these two lanes are shown in Figure 11b. From these data, it is obvious that the two-abreast turning movement had a detrimental effect on the capacity of the inside lane. The studies yielded a 6.5-sec starting delay and a 2.4-sec average time-headway for the inside lane traffic of the two-abreast turning movement as compared to a 5.8-sec starting delay and a 2.1-sec average time-headway for the traffic making a single left-turn movement on the North Frontage Road. Studies of the filmed traffic operations indicated that drivers in both lanes staggered the position of their vehicles in making the double left-turn movement. This hesitancy accounted for the inefficiency of this type of movement.

Cullen Interchange

As with the Wayside interchange, starting delay and average time-headway measurements were determined for each lane approaching the Cullen interchange. With the



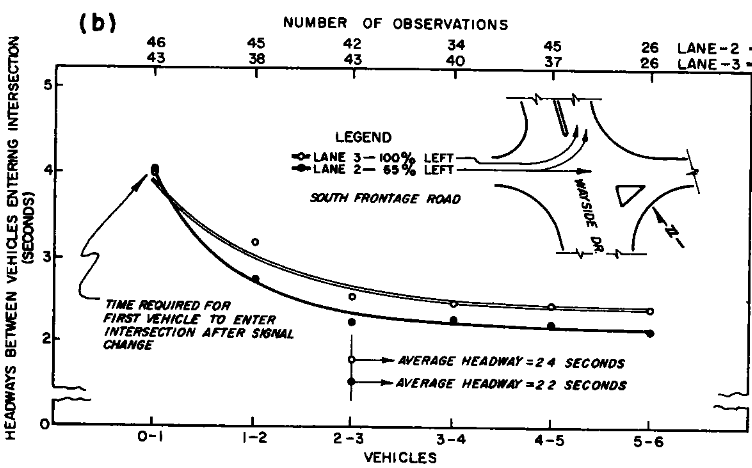


Figure 11. Time-headways between successive passenger vehicles on (a) north frontage road and (b) south frontage road at the Gulf Freeway and Wayside Drive interchange, Houston.

exception of the two-abreast-type turning movement, the movements at Cullen were similar to those observed at Wayside. The movements at Cullen were individually analyzed as follows:

Cullen Boulevard Approaches.—The traffic movement on these approaches consisted mainly of through movements with a free right-turn lane for the right turning traffic. The operational characteristics of the through traffic lanes were the only movements evaluated and their analysis showed that the through movement had a starting delay of 5.6 sec and an average time-headway of 2.1 sec. Figure 12 is a representation of the observed operations on these approaches.

North Frontage Road Approach.—This approach had three lanes of traffic. The center lane handled straight through movements, whereas the left- and right-hand lanes handled a combination of turning and through movements. The traffic in the inside lane and the center lane had similar operational characteristics with a 5.4-sec starting delay and a 2.0-sec average time-headway. The traffic in the outside lane was considerably slower with a 5.8-sec starting delay and an average time-headway of 2.1 sec. The operational characteristics of the vehicles in each lane are shown in Figure 13a.

South Frontage Road Approach.—The traffic movements on this approach were similar to those on the north frontage road. The traffic in the center lane was somewhat faster than the traffic in the adjacent lanes with a 5.4-sec starting delay and a 2.0-sec average time-headway. Traffic in the other two lanes had a large percentage of turning movements and experienced a 5.6-sec starting delay and a 2.0-sec average time-headway. These three movements with their respective time headways are shown in Figure 13b.

DATA SUMMARY

A total of seven studies was made at the study sites during which the operational characteristics of over 4,000 vehicles were recorded. Table 1A is a general summary

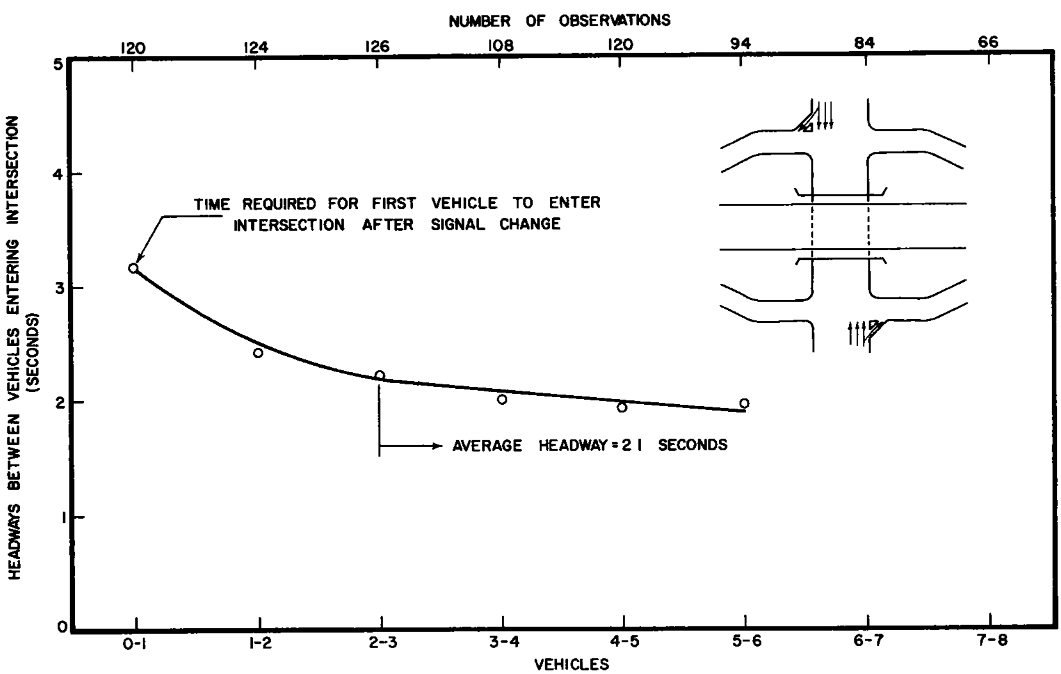


Figure 12. Time-headways between successive passenger vehicles on Cullen Blvd. at the Gulf Freeway and Cullen Blvd. interchange, Houston.

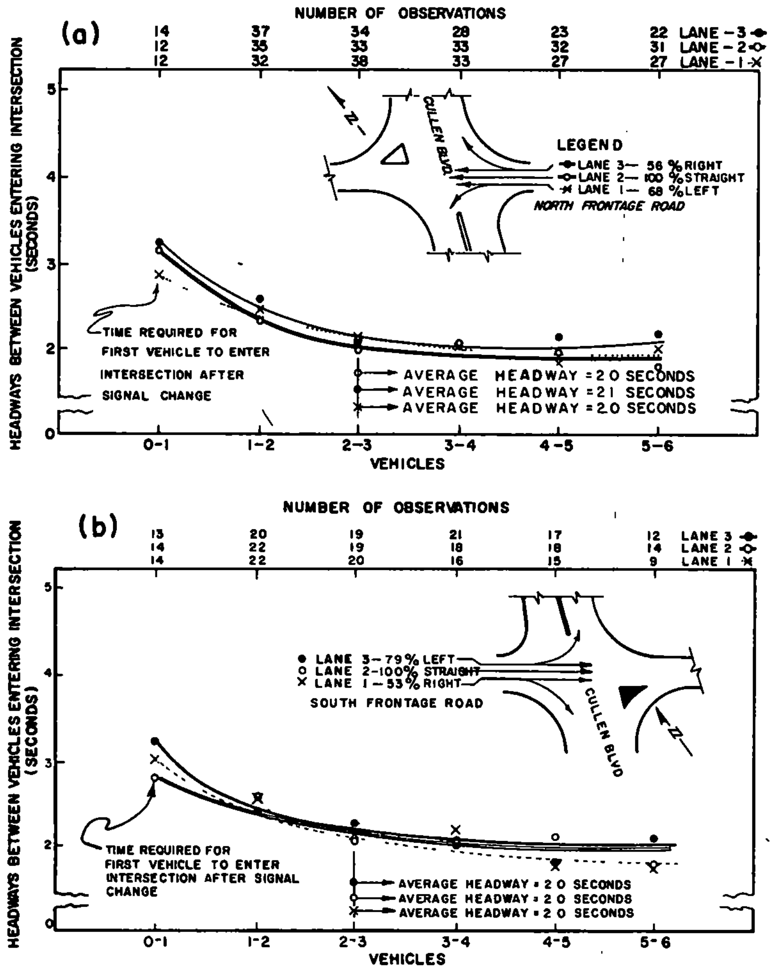


Figure 13. Time-headways between successive passenger vehicles on (a) north frontage road and (b) south frontage road at the gulf Freeway and Cullen Blvd. interchange, Houston.

of the data gathered at both the Wayside and Cullen interchanges. The operational data given in Table 1B indicated little difference in the operating characteristics of (a) straight through movements, (b) left turn movements, and (c) right turn movements. In view of this characteristic, the data on each type of movement at both the Wayside Drive and Cullen Boulevard interchange were averaged for comparison. This analogy showed that there was no significant difference in the starting delay and time-headway measurements of the straight, single left turning, and single right turning movements. However, there was a significant difference in the double left or two-abreast-type turning movement which necessitated separate consideration of these types of movements. A summary of the averaged data which are used in the capacity computations is given in Table 1B.

APPLICATION TO CAPACITY DETERMINATIONS

Capacity Formula

The problem of determining the capacity of signalized diamond intechanges is basically one of evaluating the capacity of two closely-spaced intersections. With proper

TABLE 1A

OPERATIONAL DATA

	Starting Delay (sec)	Average Time-Headway (sec)
Wayside Interchange		
Wayside Drive:		
Through movement	5.9	2.2
North frontage road:		
Lane 1 - 85% left	5.8	2.1
Lane 2 - 99% straight	5.7	1.9
Lane 3 - 82% right	5.8	2.1
South frontage road:		
Lane 1 - 57% right	6.8	2.4
Lane 2 - 65% left	6.5	2.2
Lane 3 - 100% left	6.5	2.4
Cullen Interchange		
Cullen Boulevard:		
Through movement	5.6	2.1
North frontage road:		
Lane 1 - 68% left	5.3	2.0
Lane 2 - 100% straight	5.4	2.0
Lane 3 - 56% right	5.8	2.1
South frontage road:		
Lane 1 - 53% right	5.6	2.0
Lane 2 - 100% straight	5.4	2.0
Lane 3 - 79% left	5.6	2.0

Interchange Studies

Study	Date	Time	Number of Vehicles Observed
Wayside Drive	June 9, 1960	A.M. Peak	
Wayside Drive	June 9, 1960	P.M. Peak	2,221
Wayside Drive	July 7, 1960	P.M. Peak	
Cullen Boulevard	April 20, 1960	A.M. Peak	
Cullen Boulevard	April 20, 1960	P.M. Peak	2,699
Cullen Boulevard	May 4, 1960	A.M. Peak	
Cullen Boulevard	May 4, 1960	P.M. Peak	

signalization, traffic may flow through both intersections on receiving the green on any approach and thus the capacity can be determined on a per-approach basis.

The signalized intersections of a diamond interchange fit into the classification of a "high-type" intersection as defined by the "Highway Capacity Manual" because the following conditions exist: (a) minimum pedestrian interference; (b) separate lanes for each traffic movement; (c) time separation of conflicting flows; (d) high standard of geometric design; and (e) no curb parking. On the basis of this classification, it was felt that the capacity of a diamond interchange should be determined on a per-lane basis for each approach with individual attention being given to the various types of movements.

The statistics on vehicle starting delay and time-headways provided the basic data for determining the number of vehicles per lane that can clear the intersections from an approach during each green interval. Hourly capacity values for all approaches can then be computed from this information.

TABLE 1B

Type Movement	Starting Delay (sec)	Average Time-Headway (sec)
Through	5.8	2.1
Single left turn	5.8	2.1
Single right turn	5.8	2.1
Two-abreast-type turning:		
Inside lane	6.5	2.4
Outside lane	6.5	2.2

In terms of starting delay, D, and average time-headway, H, the number of vehicles per lane, N'_L, that can be expected to clear the interchange from an approach during one green phase, G, is

$$N'_L = \frac{G - D}{H} + 2 \tag{1}$$

To determine the number of vehicles, N_L, that can clear per hour per lane, Eq. 1 can be multiplied by the number of cycles per hour (3,600/C), where, C, is cycle length in seconds, to give

$$N_L = \left(\frac{G - D}{H} + 2\right) (3,600/C) \tag{2}$$

To simplify design and evaluation procedures, capacity charts were developed from the operational data given in Table 1B and Eqs. 1 and 2. Figure 14 shows a chart with lane capacity (vehicles per hour) plotted against length of green phase for various cycle lengths (40 to 80 sec). This chart furnishes lane capacity readings for left, straight or right movements and can be used to evaluate the per-lane capacity of an approach or to determine the amount of green time required to move a specific lane volume.

Figures 15 and 16 are charts of lane capacity versus green time for the inside and outside lane, respectively, of a two-abreast left turn movement. This chart would be used for approaches signed for a double left turn.

DIAMOND INTERCHANGE CAPACITY

After studying the problem of evaluating the capacity of diamond interchanges, it was determined that it would be necessary to consider the two signalized intersections as a single unit. This is due primarily to the requirements of signalization which should perform two basic functions. These functions are as follows: (a) all high-volume conflicting movements at both intersections must be separated, and (b) storing of vehicles between the two intersections must be kept to a minimum due to limited distance between the intersections.

In considering various phasing possibilities, it was recognized that the movement sequence is dependent on existing volume conditions. However, if peak flow conditions are encountered on all approaches the movement sequence, shown in Figure 7, offers the best phasing to meet the two previously listed requirements. In addition, this sequence permits some phase overlap for maximum use of green time. This sequence was thus chosen as a basic phasing on which to base capacity considerations.

With a basic signal phasing established, the next step was to evaluate the capacity of the interchange system. For each approach to a diamond interchange, there exists a critical lane volume that must be accommodated. If the critical lane volume can be accommodated, then the adjacent lane or lanes on the same approach can accommodate less or equal volumes during the same green period. Thus it is necessary to consider only one lane (critical lane) per approach when determining the design and signalization of the interchange.

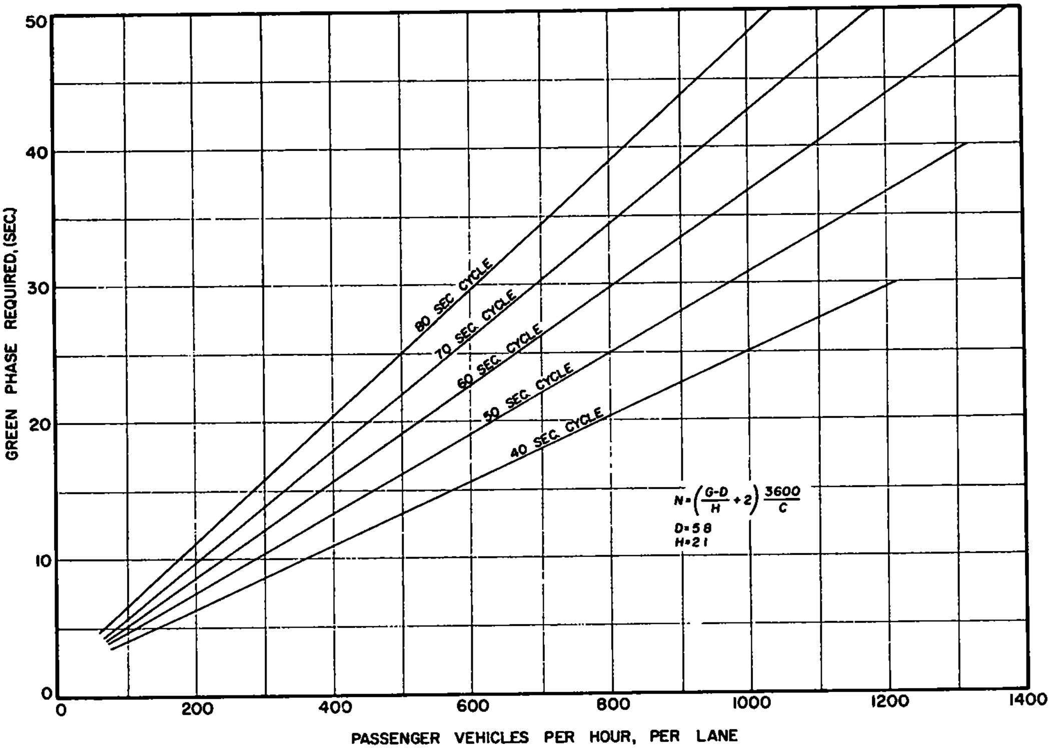


Figure 14. Design capacity of through movements and single lane turning movement.

Using this critical lane concept, an approach to the design and capacity of diamond interchanges was developed. A thorough study of conventional-type diamonds operating with a signal phasing as shown in Figure 7, indicated that the critical number of vehicles which must be accommodated is a summation of the critical lane capacities for four approaches. These critical approaches are shown in Figure 17. The interior approaches (those over or under the structure) are not critical since they receive a much greater percentage of green time than the critical approaches; for example, if the critical approach volumes can be accommodated, the interior volumes can be accommodated. This will be demonstrated in an example problem.

The critical interchange volume was determined by developing a formula for this purpose. This formula represented a summation of the single lane capacities for the four critical approaches. The formula was developed as follows:

The number of vehicles which can be accommodated from a single lane during each of the six movements (Fig. 7) is

<u>Movement E</u> - $n_1 = \dfrac{(G_1 - 5) - D}{H} + 2$

<u>Movement F</u> - $n_2 = \dfrac{5}{H} + \dfrac{8 - D}{H} + 2$

<u>Movement A</u> - $n_3 = \dfrac{G_2 - 8}{H}$

<u>Movement B</u> - $n_4 = \dfrac{(G_3 - 5) - D}{H} + 2$

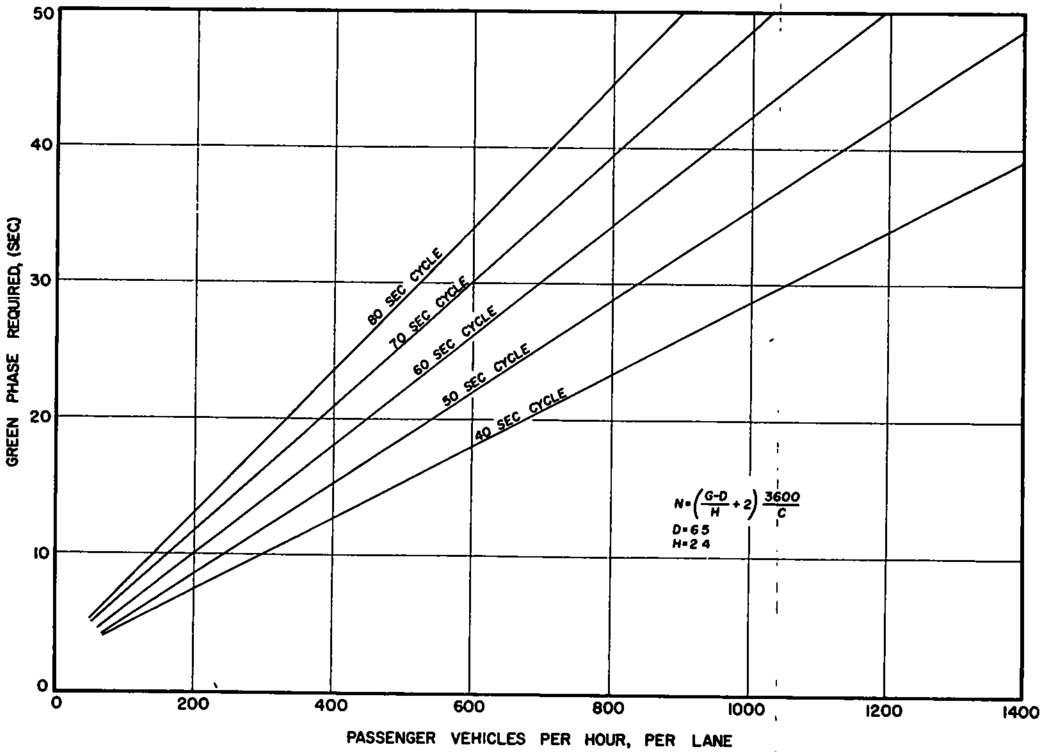


Figure 15. Design capacity of inside lane on two-abreast turning movement.

$$\underline{\text{Movement C}} - n_5 = \frac{5}{H} + \frac{8 - D}{H} + 2$$

$$\text{Movement D} - n_6 = \frac{G_4 - 8}{H}$$

(Note - An 8-sec overlap of the frontage road and major street traffic is permitted by the signalization and ambers of 3 sec were assumed.)

Total critical capacity per cycle, N_C' is equal to a summation of the vehicles accommodated during movements one through 6

$$N_C' = \frac{G - 4D}{H} + 8 \quad (G = G_1 + G_2 + G_3 + G_4) \tag{3}$$

The total cycle length, C, is equal to the following:

$$C = G + 6 - 10 = G - 4 \text{ or } G = C + 4$$

(Note - There are 6 sec of wasted amber time and 10 sec of phase overlap.)

If C + 4 is substituted for G in Eq. 3

$$N_C' = \frac{C + 4 - 4D}{H} + 8 \tag{4}$$

is obtained. If Eq. 4 is multiplied by 3,600/C or the number of cycles per hour, an

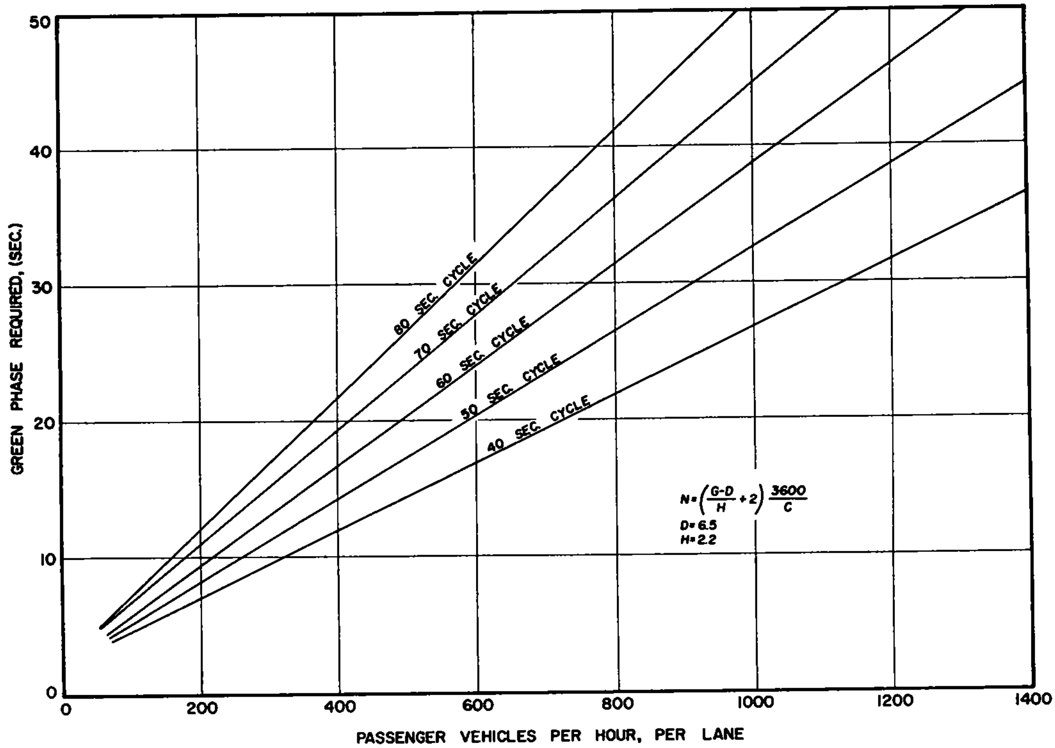


Figure 16. Design capacity of outside lane on two-abreast turning movement.

equation for the critical capacity per hour, N_H, is obtained, as follows

$$N_H = \left(\frac{C + 4 - 4D}{H} + 8\right) \frac{3,600}{C} \tag{5}$$

This critical capacity is a function of cycle length, C, starting delay, D, and time-headway, H. Eq. 5 gives the number of vehicles per hour that can be accommodated by the critical lanes on the four critical approaches. This number, N_H, has been termed "critical capacity" and represents the maximum summation of the four critical approach volumes.

If cycle lengths of 40, 50, 60, 70, 80, 100 and 180 sec and values of D = 5.8 and H = 2.1 are substituted into Eq. 5, the following values are obtained·

Cycle Length (sec)	Critical Capacity (veh/hr)
40	1,611
50	1,635
60	1,650
70	1,660
80	1,668
100	1,674
180	1,692

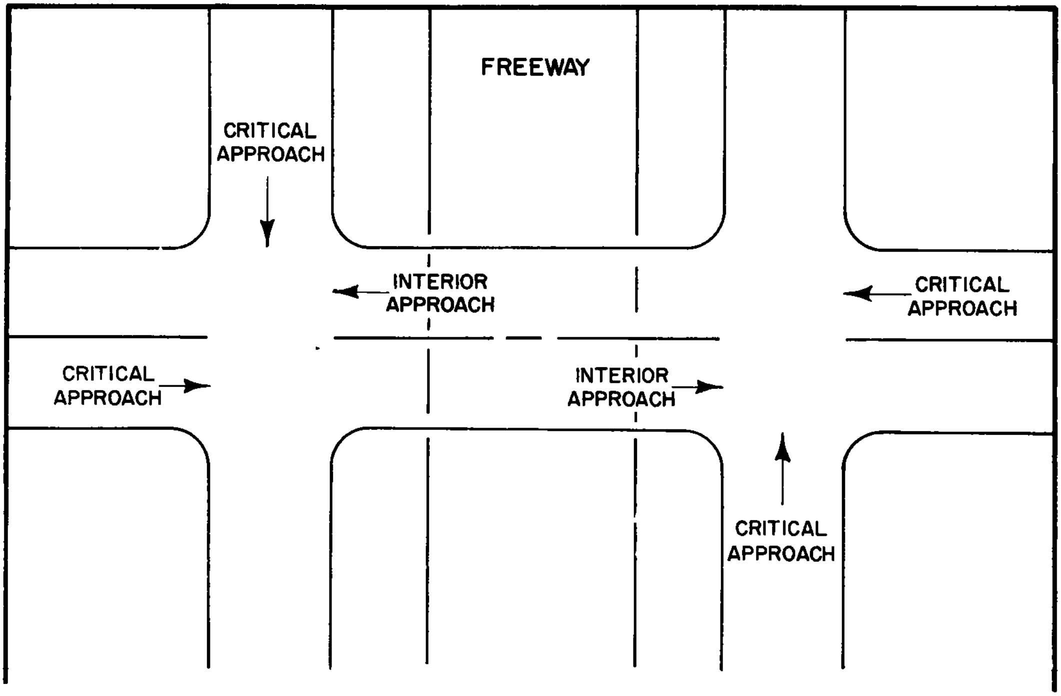


Figure 17. Critical approaches— diamond interchange.

Figure 18 shows a plot of the data in the text table. From these values it is indicated that any cycle length from 40-80 sec will yield basically the same capacity and that no significant gain in capacity is obtained by increasing the cycle length past 80 sec.

If, for example, a 60-sec cycle is assumed for design, the critical capacity is

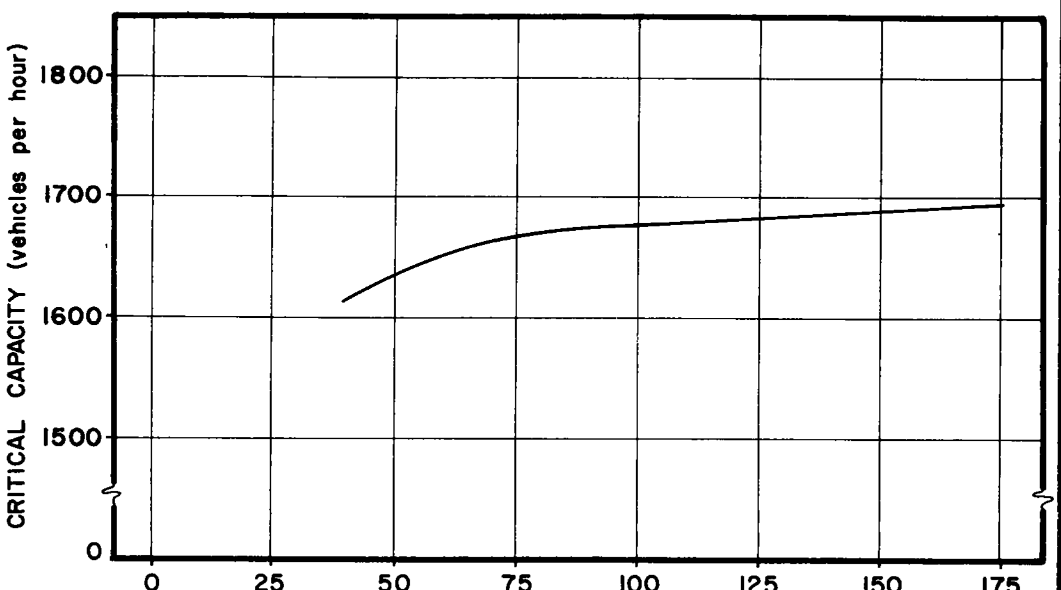


Figure 18. Cycle length (sec).

1,650 vehicles per hour. This indicates to the designer that a summation of critical lane volumes (for the four critical approaches) exceeding 1,650 veh/hr cannot be accommodated. Any combination of critical volumes with a summation less than 1,650 veh/hr can be accommodated with a 60-sec cycle or less.

Another point must be clarified at this time. The critical capacity expressed by Eq. 5 represents the total number of vehicles per hour that can be handled on the four approaches assuming that the total number is evenly distributed over the entire design hour (for example, the exact number of vehicles which can be handled per cycle is always available). Inasmuch as it is reasonable that this will not happen and that actually a great deal of fluctuation in the hourly volume will occur, special attention must be given to this factor.

Figure 19 shows a typical fluctuation of 5-min arrival volumes on an approach during the peak hour. If a design is based on the total hourly volume, it is evident that it will be inadequate to accommodate the short peaks within the hour. The question of just what volume the design should be based on warrants serious study and is presently being given detailed investigation by the Texas Transportation Institute.

On the basis of several studies, it was decided that peak hourly volumes should be increased by 20 percent to obtain a design volume which would be compatible with the indicated design procedure. This increase of 20 percent was obtained by expanding peak 30-min demands to an equivalent hourly flow and comparing this value to the total hourly demand (Fig. 19). In general, a 20 percent difference between expanded hourly demand and actual hourly demand was observed. Additional confidence in this figure was obtained from the "Highway Capacity Manual" which stipulates a 20 percent difference between Practical (or Design Capacity) and Possible Capacity. Therefore, it was determined that expected peak hourly volumes should be increased by 20 percent to obtain peak flow conditions for which Eq. 5 would be applicable.

ILLUSTRATIVE PROBLEM FOR DIAMOND INTERCHANGE

The application of the capacity formula to the design and signalization of a diamond

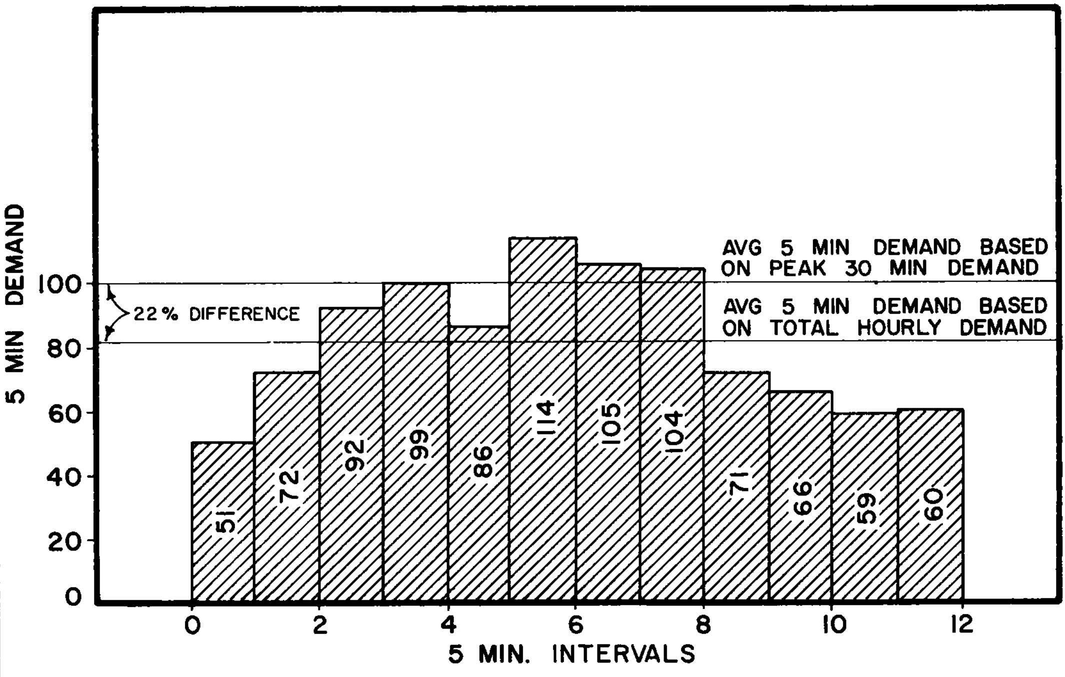


Figure 19. Hourly demand fluctuations.

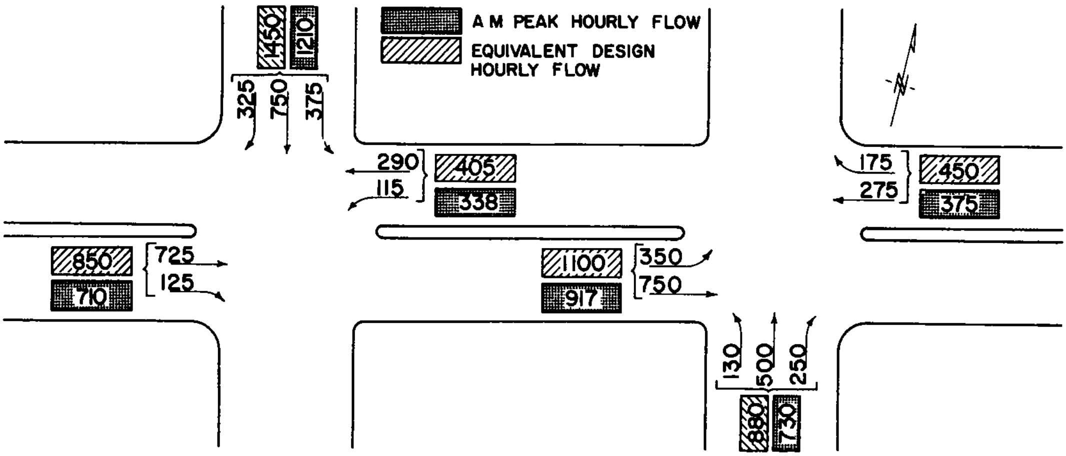


Figure 20. Design movements.

interchange can best be illustrated by an example. For this illustrative problem, traffic volumes were selected which might be representative of any large metropolitan area. These volumes are shown in Figure 20. The traffic selected is related to the intersection of a freeway and a major city arterial at which a diamond interchange is to be provided. As with all geometric design problems, both the a. m. and p. m. peaks should be considered in the design. However, for simplification, only one peak period is considered in this illustrative problem. The step-by-step design procedure is as follows:

Step I. —On the basis of the traffic movements shown in Figure 20, a reasonable geometric design is established and critical lane volumes as shown in Figure 21 are determined.

Step II. —The critical lane volumes are now examined to determine if they can be accommodated and if so, what cycle length is required.

$$\Sigma \text{ critical lane volumes} = 725 + 225 + 475 + 450$$

$$\Sigma \text{ critical lane volumes} = 1{,}875 \text{ veh/hr}$$

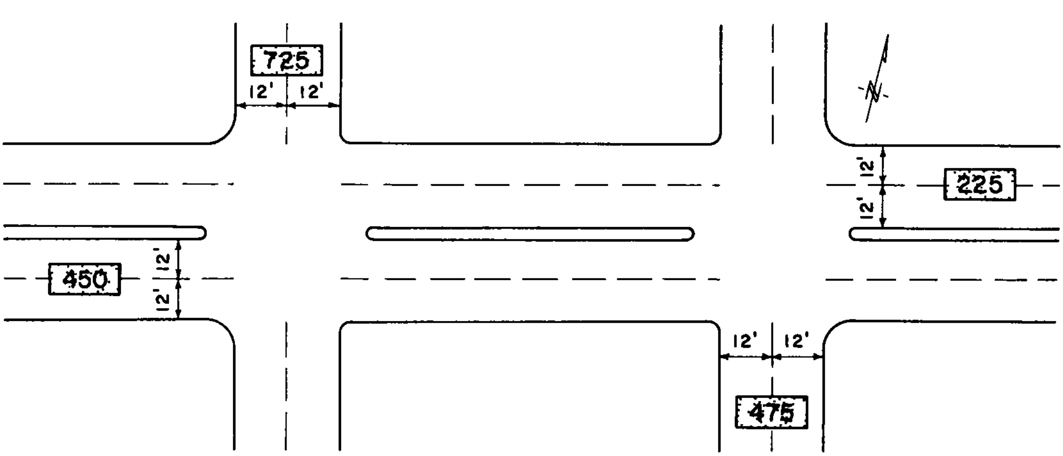


Figure 21. Critical lane volumes.

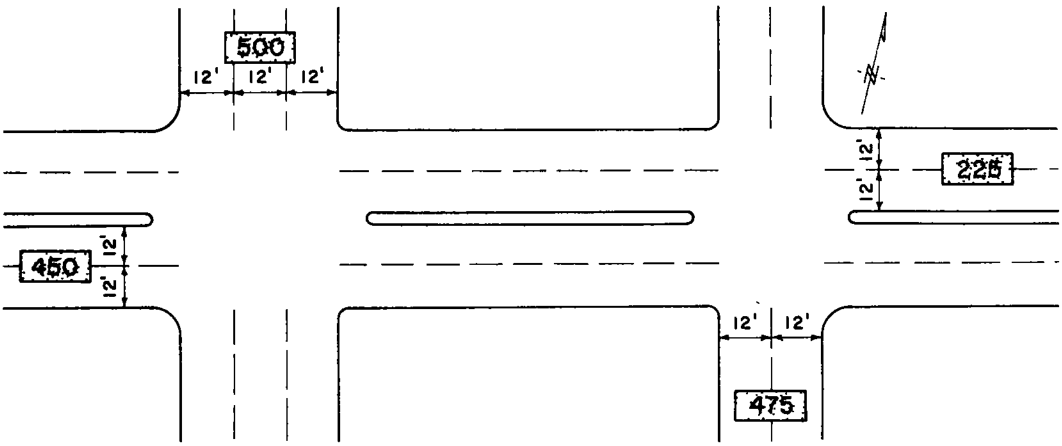


Figure 22. Critical lane volumes.

From Figure 18, it was found that this volume of vehicles cannot be accommodated with a reasonable cycle length. Therefore, the original assumed design is re-evaluated to determine what is needed to reduce the sum of the critical lane volumes. With reference to the assumed design in Figure 21, the most critical approach appears to be the west frontage road. If one additional lane is added to this approach, its critical lane volume can be reduced from 725 veh/hr to 500 veh/hr. This now gives a new design problem with critical lane volumes as shown in Figure 22.

Step III.—Considering the revised design and the new critical lane volumes, another check is made to determine if the revised volumes can be accommodated.

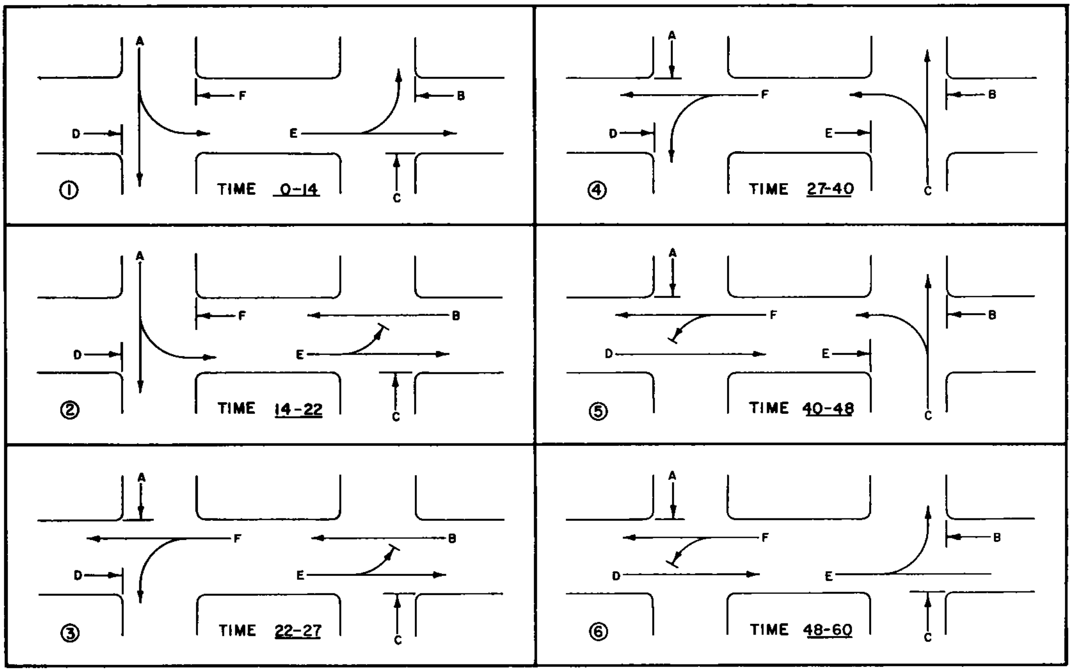


Figure 23. Phasing of traffic movements.

$$\Sigma \text{critical lane volumes} = 500 + 450 + 475 + 225$$

$$\Sigma \text{critical lane volumes} = 1,650 \text{ veh/hr}$$

Figure 18 indicates that this volume can be accommodated with a 60-sec signal cycle.

Step IV.—With the newly established lane volumes, Figure 14 is used to determine the amount of green and amber time required for each approach. The time required for each of the approaches is

Approach	Green (sec)	Amber (sec)	Total (sec)
A	19	3	22
B	10	3	13
C	18	3	21
D	17	3	20

Once the signal cycle and the length of each green plus amber phase is established, a phase arrangement for the movements is determined. A phase arrangement as shown in Figure 23 is used in this illustrative problem. With this phasing arrangement, each movement is timed so as to give each approach the required green and amber time as previously established. The timing of each movement is shown in Figure 23.

Step V.—Now that the capacity of the critical approaches has been determined, a capacity check of the interior approaches is made to insure that they are capable of accommodating their assigned volumes. The volumes to be accommodated on approach E is 350 veh/hr turning left and 750 veh/hr proceeding straight. The amount of green time allotted the straight through movement is 38 sec and will handle 980 veh/hr per lane which is greater than the 750 veh/hr demand. The time allotted the left turning movement is 23 sec and this time will accommodate 600 veh/hr per lane which is greater than the 350 veh/hr demand. Therefore, the traffic on approach E can be handled efficiently with the time which is allotted.

The traffic demand on approach F is 115 veh/hr turning left and 290 veh/hr proceeding straight. The green time allotted the straight through movement is 35 sec and will accommodate 900 veh/hr per lane. The left turning movement on this approach is allotted 16 sec which is capable of accommodating 410 veh/hr per lane. Consequently, this approach is also not critical in view of the amount of green time which is available.

It should be clarified that although the two lanes in each direction under the structure are adequate for this illustrative problem, their adequacy is dependent on the signal phasing used. If, for example, a signal phasing was used that required storage between the closely spaced intersections, then more than two lanes would probably be required.

This illustrative design problem has shown how the capacity-design procedure may be used in designing and signalizing a conventional-type diamond interchange. With modifications, this design procedure may be adapted to various intersection and interchange design problems.

DIAMOND INTERCHANGE SIGNALIZATION

The signalization of diamond interchanges is dependent on the volume conditions which may be encountered. However, of the various volume conditions which may exist, the most critical condition occurs when heavy traffic movements are experienced simultaneously on the four critical approaches (Fig. 17). This volume condition requires a signal sequence which will eliminate excessive storing of vehicles between the two closely spaced intersections. The signal sequence shown in Figure 7 provides the best phasing for this volume condition.

The recommended signal sequence shown in Figure 7 can be obtained by either fixed-time or vehicle-actuated equipment. However, because large fluctuations in traffic volumes are usually encountered on each approach of a diamond interchange, traffic-actuated equipment of the volume-density type could adjust to these fluctuations and provide more efficient operation.

SUMMARY AND CONCLUSIONS

The initial phase of this report represents the results of a study of vehicle operational characteristics at signalized intersections. This study was aimed at developing a method of determining capacity for intersection approaches on a single lane basis. The results of the study are as follows:

1. Starting delay for a queue of stopped vehicles at an intersection can best be determined by considering the time required for the first two vehicles in line to enter the intersection.
2. A time-headway value for vehicles entering an intersection can be accurately represented by an average of the time-headway values of the third through the last entering vehicle.
3. Capacity charts can be developed on the basis of starting delay and time-headways which will indicate lane capacities of an intersection approach for a given amount of green time.
4. The studies conducted on operational characteristics of vehicles at high-type signalized intersections indicated that there was no significant difference in the capacity of a straight through movement as compared to a single-lane turning movement.
5. Double left turns or two-abreast-type turning movements have a reduced capacity per lane as compared to a single left-turn movement. Capacity charts for two-abreast left turns are presented in the report.
6. A limited amount of commercial traffic was observed in the study. On the basis of the data obtained, it was found that heavy commercial vehicles had the equivalent effect (time-headway and starting delay) of approximately 1.6 passenger cars.

The second phase of the report was devoted to developing a method for determining the capacity and design of a conventional-type diamond interchange. This method and procedure is presented in the report.

It is realized that the method presented is related to a conventional-type diamond and to a basic fixed-time arrangement. It is felt that the basic method presented could be modified, adapted and used in the design of other forms of diamond interchanges or on individual high-type intersections. Any design problem requires extensive engineering judgment and the method presented is intended only as a design tool. It is further felt that the design method presented represents a good approach to the design of conventional diamonds regardless of the type of signalization which may eventually be used.

ACKNOWLEDGMENT

Grateful acknowledgment is made to Cooper McEarchen, Director of Traffic and Transportation for the City of Houston, Texas, and his staff and to Dale Marvel, District Traffic Engineer with the Texas Highway Department, and his staff for the valuable assistance rendered during the field studies. Thanks is also expressed to members of the staff of the Texas Transportation Institute who worked in the collection and analysis of the field data.

REFERENCES

1. Greenshields, B.D., et al., "Traffic Performance at Urban Street Intersections." Yale University, New Haven, Conn. (1947).
2. Leisch, J.E., "Adaptability of Interchange Types on the Interstate System of Highways." Journal of Highway Division, ASCE, HWI, Paper 1525, Vol. 84 (Jan. 1958).

3. Moskowitz, K., "Signalizing a Diamond Interchange." Proc. Northwest Traffic Engineering Conf. (July 1959).
4. Webb, G.M., and Moskowitz, K., "Intersection Capacity." Traffic Engineering, 26:147-154 (Jan. 1956).
5. U.S. Bureau of Public Roads, "Highway Capacity Manual, Practical Applications of Research." U.S. Govt. Print. Office, Washington, D.C. (1950).

Freeway Traffic Accident Analysis and Safety Study

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In 1958, the Texas Transportation Institute initiated a research project under the sponsorship of the Automotive Safety Foundation to determine the possible correlation between freeway accidents and specific geometric design features on urban freeways and to investigate methods of improving accident reporting. A report on the phase concerned with improving accident reporting procedures was published in the December 1960 issue of Traffic Engineering.

This study considered some 10,000 accident reports on 54 miles of freeways in the five largest Texas cities, covering from 2 to 5 years of data. The reports, obtained from the files of the various cities, were recorded on microfilm to facilitate handling. The collision diagrams were plotted on continuous strip maps of each freeway showing the geometric and profile features to scale.

A study of the continuous collision diagrams for various freeways indicated that certain locations experienced higher accident frequency than other locations. Because certain locations experienced high accident frequency and others having the same apparent physical characteristics experienced low accident frequency, even with due consideration of the respective volumes, it was probable that some physical factor contributed to the high accident frequency. If this relationship could be determined, a reduction in the number of accidents might be realized by improvements in design.

Concentrations of accidents generally involved the following design features: (a) major changes in vertical alignment (crests and sags), (b) freeway ramps, (c) freeway interchange elements (interchange intersections and frontage roads), and (d) fixed objects. This report covers studies of accident concentrations in relation to each of these design characteristics.

Concentration of accidents at crests and sags indicated that the requirement for visibility or sight distance is particularly important. Freeway traffic, regardless of volume, tends to travel in more or less compact groups or platoons. The headway between the vehicles is often very short. For safe operation, the trailing driver must have a view of the traffic, or vehicles, for some distance ahead of him in both his and/or the adjacent lane or lanes. This visibility distance is greater than the distance required by the criteria for stopping sight distance on 2-lane roadways.

Entrance or on-ramps on the 10 freeways experienced much higher accident frequency than the exit or off-ramps. There was indication that the sight relationship afforded both the on-ramp drivers and the freeway through-lane drivers was especially critical. All high accident frequency on-ramp locations involved poor sight relationships.

Off-ramps with large angles of divergence and especially those affording poor visibility experienced higher frequency among off-ramps.

Diamond interchange intersections contributed three-fourths of the 21 percent of all freeway accidents which occurred on the frontage road system.

Most of the frontage roads were continuous and two-thirds were one-way. The frontage road at the points where the ramps joined the frontage roads and the section between the two ramps (included because the frontage roads were continuous) contributed only 2 percent of all freeway accidents. Practically all of the two-way frontage roads were at extremely low volume locations and afforded no accurate comparison with the one-way sections.

Much improvement is needed in the operational control of diamond interchanges. Their complexity coupled with inadequate operational controls probably contributed to the accident frequency at these points.

Twelve percent of the freeway accidents involved fixed objects. Such accidents caused 38 percent of all injuries and 65 percent of all fatalities. A large percentage of fixed-object accidents occurred on the through-lanes and mostly at exit ramps. Medians (curbs and median barrier) were involved in one-third of the fixed-object accidents.

Early in the study it was discovered that the techniques used by enforcement officials in many cities in the investigation and reporting of freeway traffic accidents did not supply information adequate for proper engineering analysis. This inadequacy of reporting techniques seriously affected the progress and accuracy of correlations of accidents with design features.

The procedure developed in the early phases of the project to provide special freeway accident diagrams and the establishment of reference points along the freeway proved to be beneficial in improving the accuracy of reporting. The police departments reported greater ease in accurately locating and reporting freeway accidents.

Although few conclusive relationships were found between accident experience and specific design elements, inadequate sight relationship appeared to be a factor in all high accident frequency locations.

This study has emphasized the necessity of completely planning the study ahead of time, arranging for proper accident reporting, and obtaining necessary volume data during the study period for a most complete and accurate picture. The procedure of "looking back" followed in this study leads to a great deal of frustration brought about by inadequacy and inaccuracy in accident reporting.

●THE FUNCTION of freeways is the safe and efficient movement of large volumes of traffic. The vast expenditures for their construction and the traffic volume which they accommodate demand that the highest possible degree of safety be embodied in their design. Recent research by the Texas Transportation Institute involving the correlation of design and operational characteristics of freeways in Texas (1) emphasized the value of accident analysis and the need for much improvement in securing information on accidents.

The traffic accident rate (accidents per 100 million vehicle-miles) is generally lower on freeways than on lesser types of traffic facilities. However, the fact that some locations on the freeways experience relatively high accident frequency indicates that there may be some geometric design features contributing to these concentrations of accidents.

Numerous studies have been made on two-lane rural highways and on city streets, but only a few recent studies have dealt with the analysis of freeway accidents. These freeway accident studies have attempted to correlate accident occurrence with general design features such as interchanges, ramps, and curves, and other factors such as weather and light. Some studies have dealt with specific design treatments of components such as medians.

Established knowledge indicates that many accidents may be attributed to driver factors such as lack of skill, inattention, carelessness, or flagrant disregard of restrictions and suggests that the highway engineer should work diligently for the most easily understood design elements, especially on the express traffic facilities designed for the most consistent coordination of safety, speed, and volume. Because loss of life, personal injury, and property damage can be lessened by better design and operation of traffic facilities, research regarding these complex problems is deemed necessary.

To study freeway accidents properly, it is necessary to have precise information concerning the location and manner of occurrence. The techniques used by the enforcement officials of many cities in the investigation and reporting of freeway traffic accidents do not supply such information consistently and adequately for proper engineering analysis. Therefore a need for study of reporting procedures exists rather generally.

If an analysis should show consistently high accident occurrence, particularly of the same type, at locations where the same or similar design features exist, then these features must present some inherent characteristics which adversely affect the psychological or physical behavior of a considerable number of drivers. Similarly, these design features might be related to some of the operational limitations of a significant proportion of vehicles. Where a few locations are experiencing high accident frequency and others having the same apparent physical characteristics are experiencing low accident frequency, even with due consideration for their respective traffic volume, it is probable that some physical factor contributes to the high frequency. Thus, a reduction in the number of accidents might be realized by improvements in design.

RESEARCH OBJECTIVES

In January 1958 the Texas Transportation Institute began a research project, sponsored by the Automotive Safety Foundation, with the primary objective of determining the correlation between accident frequency and specific geometric design features on the urban freeways in Texas.

A secondary but important objective was that of improving accident reporting methods in order to make the investigating officer's job easier and to make the accident reports of more value to the engineer.

SOURCES OF DATA

The research involved examination of microfilmed copies of some 10,000 accident reports on 54 miles of freeways in the five largest Texas cities (Figs. 1 - 7). These were supplied by the police departments of the cities and covered from 2 to 5 years, according to data available, 1954-58. Continuous collision diagrams were prepared by plotting accidents on strip maps of these freeways (Fig. 8). The geometric features of design, profile data, etc., were supplied by the Texas Highway Department and the traffic engineering departments of the various cities. Table 1 gives the mileage, through-lane volume data, accident rates and fatality rates for the various freeways, for the years 1957-1958. Volume data consisted of the average daily traffic at the location of permanent count stations on each of the freeways. Other volume data included ramp volumes and corresponding through-lane volumes taken immediately back of the ramp. Such data were not available for all possible study locations, nor for all years for which accident records were available.

A project advisory committee, composed of key administrative personnel of the five largest Texas cities, the Texas Highway Department, the Department of Public Safety, Bureau of Public Roads and Automotive Safety Foundation contributed valuable advice and assistance throughout these studies.

Figure 1. North-South Expressway, Austin.

Figure 2. Central Expressway, Dallas, Texas.

Figure 3. East-West Freeway, Fort Worth.

Figure 4. North-South Freeway, Fort Worth.

Figure 5. Eastex Freeway, Houston.

Figure 6. Gulf Freeway, Houston.

Figure 7. South, East and North Expressways, San Antonio.

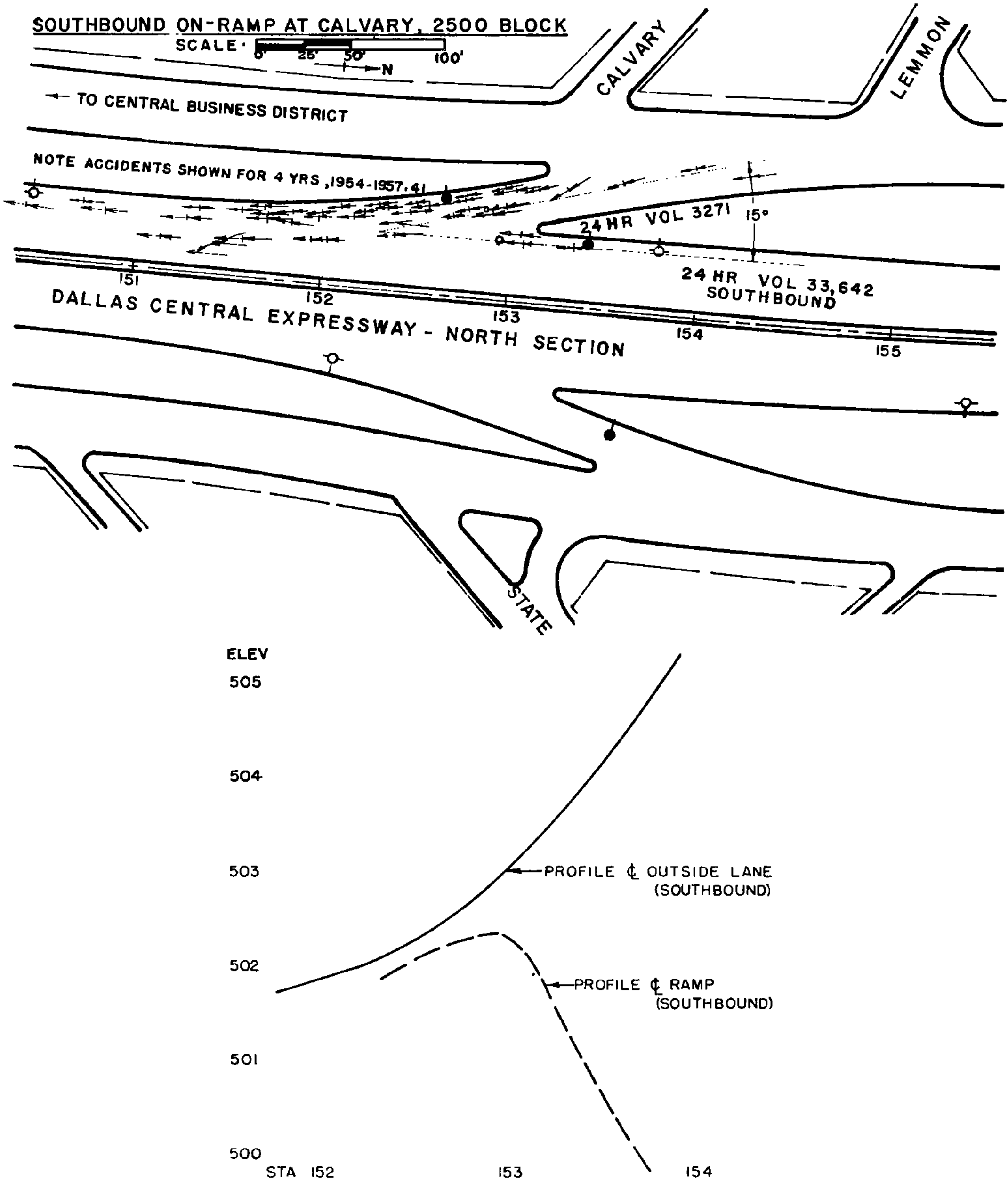


Figure 8. Section of strip map showing collision diagram.

DIAGRAMS AND NOMENCLATURE FOR FREEWAY ACCIDENT REPORTING

The need for development of improved methods of reporting accidents became evident early in the study. A study of accident reports revealed the fact that police personnel were not familiar with the complicated engineering terminology related to the design features. Accident diagrams plotted on the report forms were often so inadequate and terminology so confused that it was impossible to determine on what portion of the freeway the accident occurred. Examples of inadequate reports are shown in Figure 9.

TABLE 1

MILEAGE, ADT VOLUMES, ACCIDENT RATES, AND FATALITY RATES OF FREEWAYS

Freeway	Freeway Code No	Miles of Freeway Studied	ADT (through lanes only)	Accid. in 1957-58 Including Through Lanes, Ramps, and Frontage Roads	Accident Rate per Million Vehicle-Miles	Fatalities in 1957-58	Fatality Rates per 100 Million Vehicle-Miles
Dallas - North	1	8 9	59,000	971	2 533	2	0 522
Dallas - South	2	3 3	29,000	282	4 037	0	0 000
Houston - Eastex	3	7.0	28,400	478	3 294	4	2 756
Houston - Gulf	4	9 2	81,800	1,548	2 818	9	1.638
Fort Worth - East-West	5	5 7	34,000	746	5.273	3	2.121
Fort Worth - North-South	6	6.6	29,000	637	4 559	7	5.010
Austin	7	8 5	22,000	244	1 787	0	0 000
San Antonio - North	8	2 2	35,000	180	3 202	2	3 558
San Antonio - South	9	1 5	23,000	138	5.479	2	7 941
San Antonio - North-East	10	1 2	17,500	50	3 262	2	13 046
Total - Miles, accidents, fatalities		54.1		5,274		31	

Note Rates are based on maximum through-lanes ADT volumes only, as frontage road volumes were not generally available

Effort was made, however, to spot the locations of accidents accurately enough both longitudinally and laterally, to provide correlation of accident location and type as related to specific geometric roadway features such as entrance and exit ramps, speed-change lanes, frontage roads, overpass structures, sight distances, grades, and guardrails.

In an attempt to eliminate at least some of the confusion incurred by accident investigators having to draw complicated diagrams to describe locations of accidents, standard diagrams of several typical freeway sections were developed complete with simple nomenclature. These standard diagrams are shown in Figures 10 and 11. They were furnished to the police departments of each of the participating cities for trial use.

LOCATION OF ACCIDENTS BY REFERENCE POINTS

The general practice of most city police departments to use block numbers to reference accident locations to street intersections is not well suited for freeway accidents. Study of the assembled accident records indicated that the system of referencing freeway accidents did not furnish location information with sufficient accuracy for the engineer to relate the accident occurrence to the design features. The proportion of accidents which could not be accurately plotted, mainly because of inexact location information, ranged from 5 percent for one freeway to 60 percent for another.

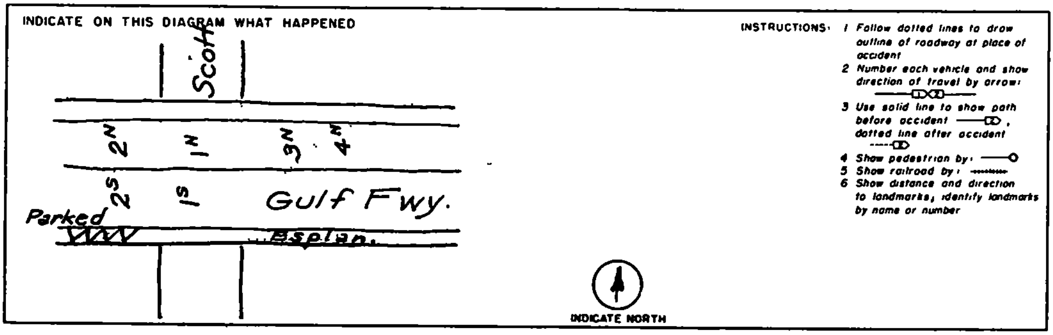
A suggested system of establishing reference points along the freeways from which accidents can be accurately referenced was recommended to the participating cities for trial use. Typical serial numbers established by two of the cities are shown in Figure 12. The system involved the use of a serial number for each reference point and incorporated the block number where applicable, along with the serial number.

STUDY METHOD

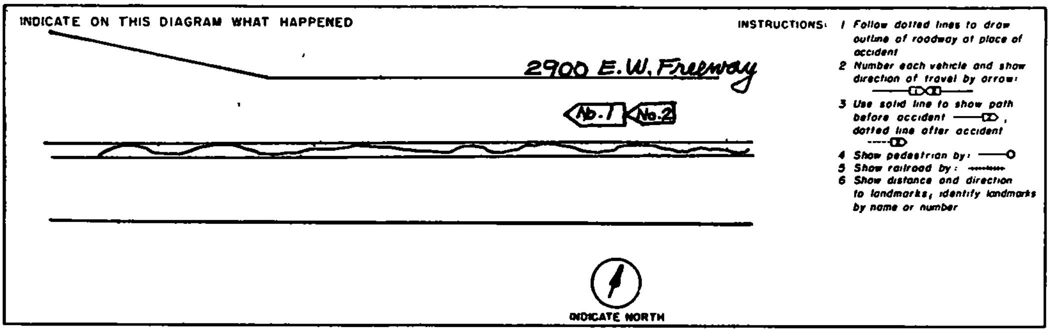
A number of participating cities made trial use of the special freeway accident diagrams and the Texas Highway Department Districts established reference points along the freeways. The combined use of special diagrams and reference points resulted in marked improvement in the ability to locate accidents. It is regrettably noted that few cities continued to use the special diagrams after the trial period. A more detailed report of this phase of the project was published in December 1960 (3).

ANALYSIS OF ACCIDENTS DATA

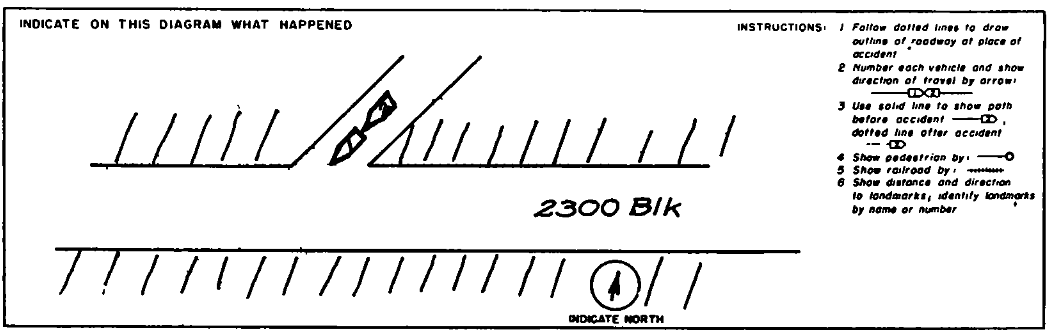
An attempt was made to code all accident information, traffic data and roadway design factors for punched card analysis. This proved to be impractical because of the number and complexity of the variables and the difficulty in locating many accidents accurately enough for this type of analysis. It was found that the loss of "personality" of the individual accident made proper analysis more difficult. Too often it was necessary to estimate certain data to facilitate coding and there was a tendency for these



Through-lane Accident Inadequately Described and Unsatisfactorily Located



Accident Adequately Indicated but Located Only Within the Block Since no Distance is Given to the Presumed Off-Ramp



Accident Indicated but Location Doubtful as to whether on a Ramp Entering the Through-lanes or on a Street Intersecting the Frontage Road

Figure 9. Standard accident report form with collision diagrams unsatisfactory for freeway reporting.

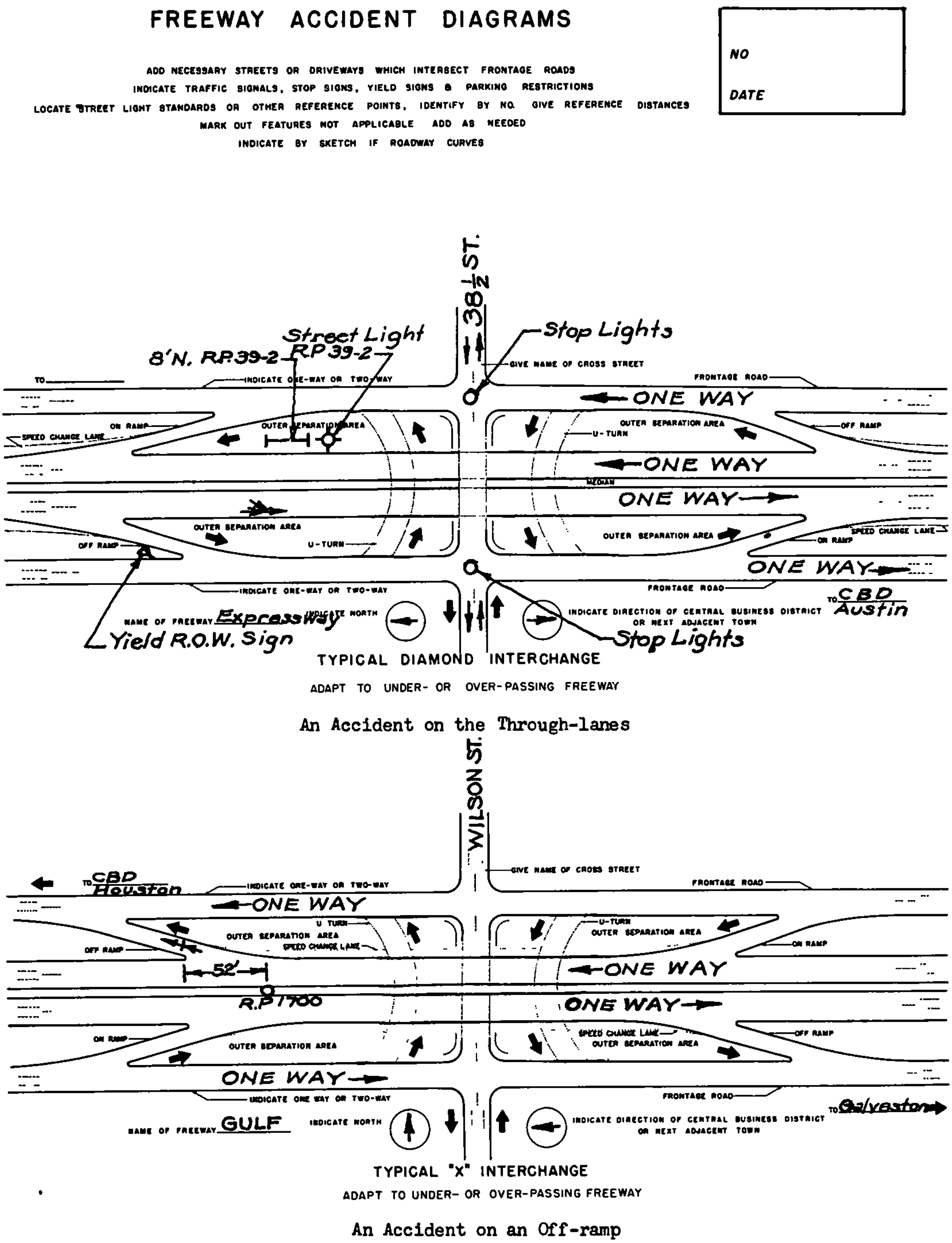
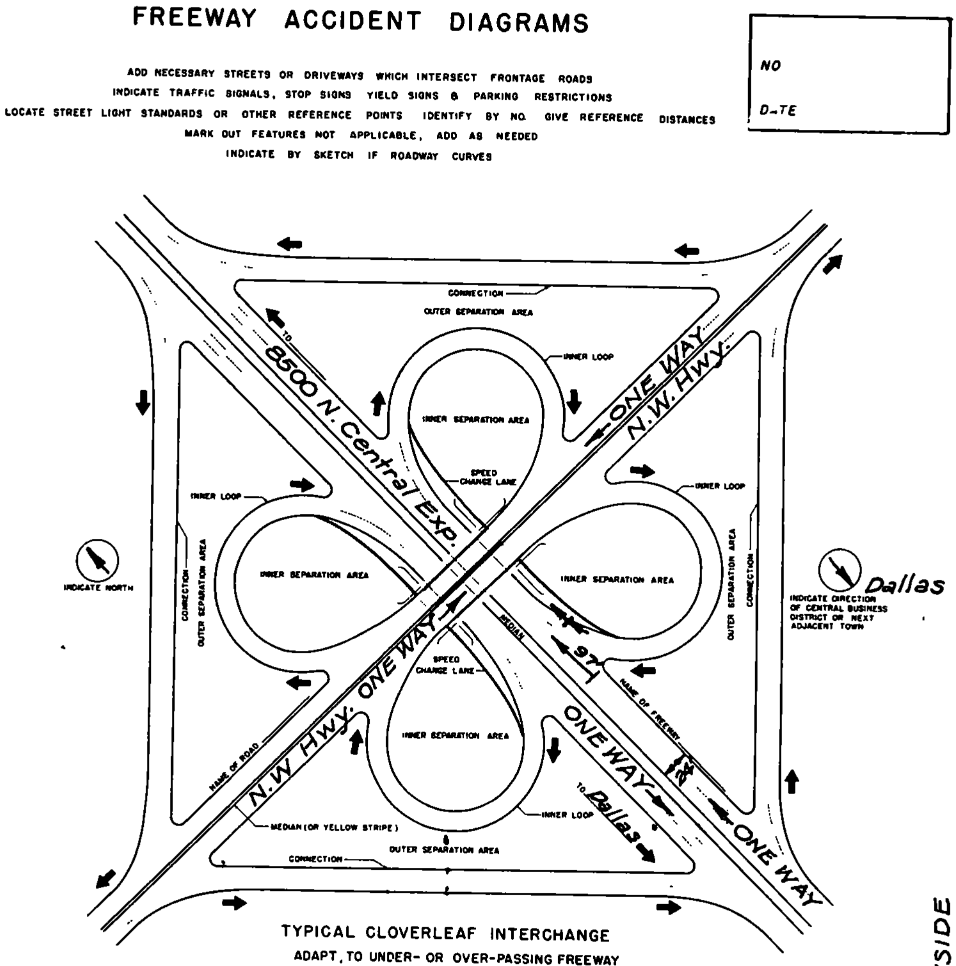
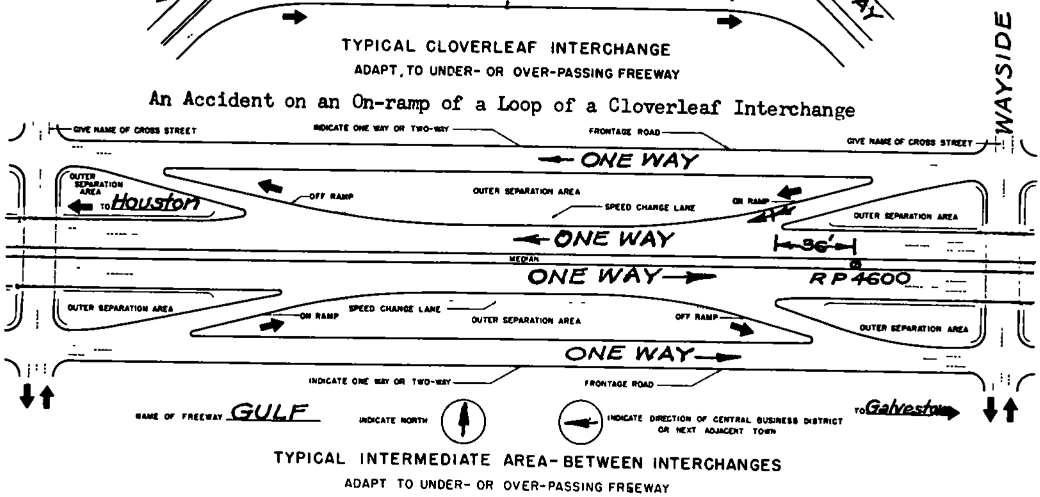


Figure 10. Use of typical freeway diagrams encourages satisfactory location and description of accidents on the freeway.



An Accident on an On-ramp of a Loop of a Cloverleaf Interchange



An Accident on an On-ramp Approaching an Intermediate Area

Figure 11. Better accident reporting on other freeway sections.

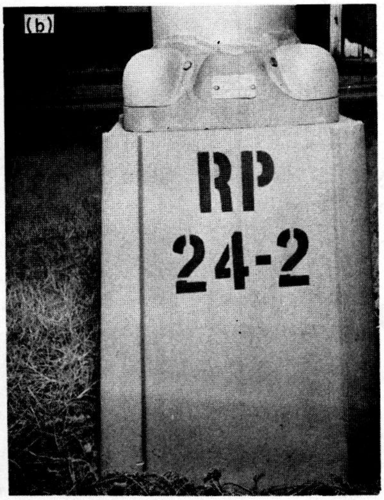


Figure 12. (a) Reference number of median guard fence; and (b) reference number on luminaire standard.

estimated values to then become "fact" when coded and considered only as numbers.

Several comparisons and analyses were made in connection with studies of specific locations on a number of the freeways. Most of these were in cooperation with other studies of traffic operation, ramp design, etc. The results of these special studies have been used in developing the reports of the other studies (1, 2, 4).

After a number of preliminary tabulations and analyses, it was decided, with the help of the advisory committee, to study each design element by determining the accident experience of this element in relation to its design. It was also determined that the study of high accident frequency locations probably would reveal the combinations of elements contributing to high accident experience.

Because of the custom of designing accidents by block numbers, graphs of accident frequency by blocks were prepared to determine the distribution of accidents along the freeway (Fig. 13).

The next step in such an approach necessitated a summary of accidents on all the freeways to indicate the geometric design features of the freeway systems where the greater proportions of accidents occurred. Such a summary, along with the figures for each freeway, is shown in Figure 14 and Table 1. In addition to the percent of all accidents which occurred on the through lanes, ramps, frontage roads and cross streets, the average daily traffic, total accidents and freeway mileage for each freeway are shown. Volume of traffic and number of accidents are shown for the years 1957-58. Table 1 gives the accident and fatality rates for all freeways. Only two years of accident data could be obtained for one of the largest volume and longest freeways, therefore only the two years 1957 and 1958 were used for this summary.

Grouping the frontage roads, the cross streets and the ramps together as interchange elements, the analysis in Figure 14 showed the following distribution: (a) freeway through lanes—53 percent of total accidents; and interchange elements—47 percent of total accidents.

ACCIDENTS ON INTERCHANGE ELEMENTS

Practically all of the interchanges on all 10 freeways included in the study were of the diamond type. Most were regular diamond, with a few reversed diamond or X type. Some were a combination of these two types of ramp arrangement. The one cloverleaf and one fully directional interchange within the study area did not provide enough samples for comparative analysis.

Frontage Roads

Because the Texas freeways included in this study were built on rights-of-way

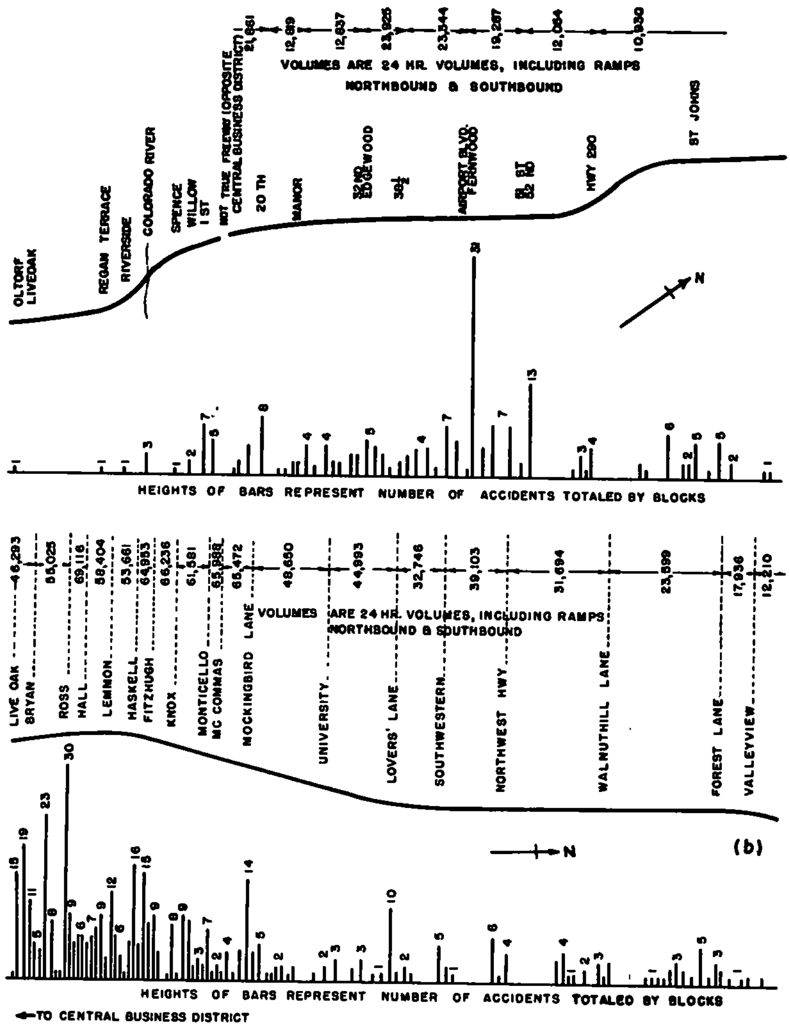


Figure 13. (a) Accident frequency, 1957, North-South Freeway, Austin; and (b) Accident frequency, 1957, North Central Expressway, Dallas.

furnished by local governments, they were constructed with continuous or nearly continuous frontage roads. The interchange elements were considered as the ramps, frontage roads, interchange intersections and cross streets between the two intersections of the diamond interchange.

Accidents on the frontage roads and interchange intersections were 21 percent of all freeway accidents. The interchange intersections accounted for 75 percent of these accidents or 16 percent of all freeway accidents. The frequency of accidents on two-way and one-way frontage roads was closely proportional to their respective mileage and the location of accidents on frontage roads was about the same for both.

Table 2 gives the percentages of accidents at the various locations along the frontage roads.

Volume data were not available for the various locations along the frontage roads; therefore, only summary data are presented. The accident experience on two-way frontage roads could be expected to be higher than on one-way, but because most of the two-way frontage roads were at low volume locations, the data presented are somewhat misleading.

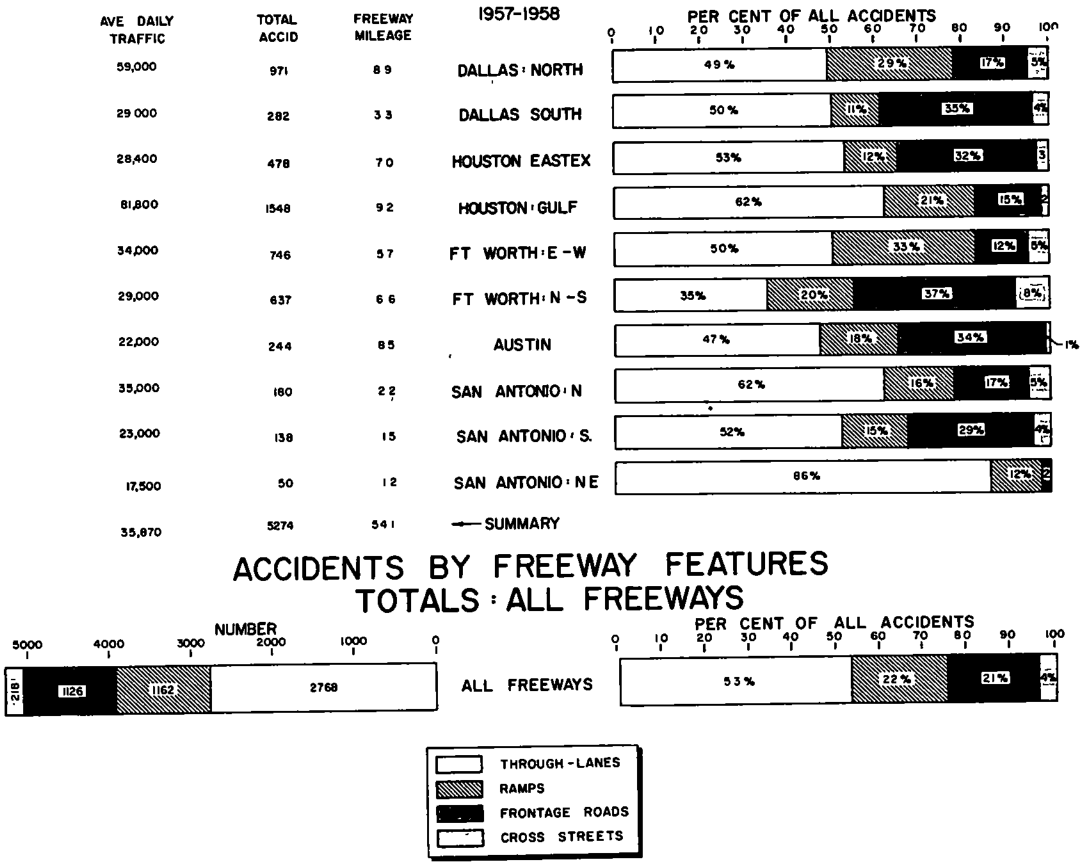


Figure 14. Comparison of accidents by freeways and by freeway features.

One fact worthy of note is the low accident frequency at locations B, C, and D, inasmuch as these would be the only locations not involved if the frontage roads were not continuous. The locations A and E and the intersections, I, would be required for diamond interchanges without continuous frontage roads. Locations B, C, and D contributed only 11 percent of the frontage road accidents. This amounts to only about 2 percent of all freeway accidents.

FREEWAY THROUGH LANES

A study of the through-lane accidents indicated that they were concentrated in the areas of change in vertical profile (crests and sags) and in the vicinity of ramps: crests and sags—35 percent; vicinity of ramps—18 percent; total on through lanes—53 percent of all accidents.

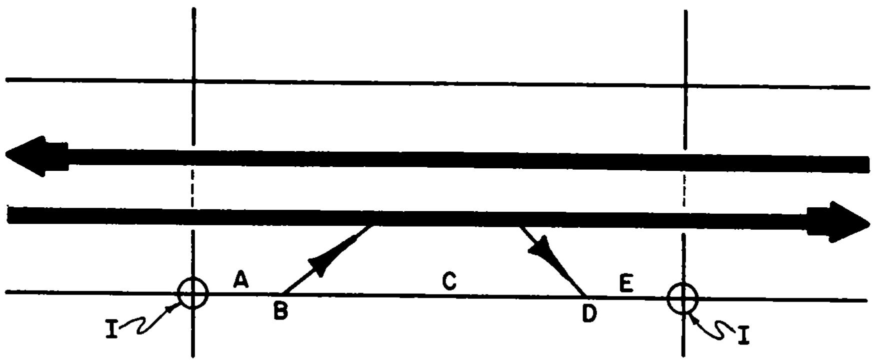
Through-lane accidents were of three basic types—rear end, side swipe, and fixed object, with a scattering of all types of accidents as expected.

Crest-Sag Accidents

In plotting accidents on the freeway strip maps, difficulty was encountered in accurately locating many of the accidents as indicated in the earlier discussion concerning the procedure of developing the accident data. Many accidents occurring on the through

TABLE 2

FRONTAGE ROAD ACCIDENTS (1957-1958)



	% of Frontage Road Mileage[a]	Total Accidents No.	(%)	I No.	(%)	A No.	(%)	B No.	(%)	C No.	(%)	D No.	(%)	E No.	(%)
One-way	67	671	(60)	487	(73)	48	(7)	10	(1)	40	(6)	21	(3)	65	(10)
Two-way	33	455	(40)	368	(81)	16	(4)	7	(2)	28	(6)	16	(3)	20	(4)
Summary		1,126	(100)	855	(76)	64	(6)	17	(2)	68	(6)	37	(3)	85	(7)

Total No.	(%)	I		A and E		B and D		C	
1,126	(100)	855	(76)	149	(13)	54	(5)	68	(6)

[a]Total frontage road mileage: 93.6

lanes were not tied to any reference and therefore could not be accurately located along the freeway. It was possible in most cases to assign these to the general area of occurrence such as the block number or section between interchanges. The long stretches of through lanes between interchanges provided fewer convenient reference points from which to accurately locate the accidents.

An analysis of the accidents on the through lanes on the Central Expressway in Dallas where 94 percent of the accidents could be accurately located to within a few feet of actual point of occurrence, showed that 69 percent of the through-lane accidents occurred on crests and sags. A study of all accidents that could be accurately located on all of the freeway through lanes indicated approximately the same percentage (Fig. 15). Comparison of mileage shows crest and sag areas to be 54 percent of the through-lane mileage (Fig. 16).

A study of the strip maps indicated that the points of grade reversals—that is, accending to descending (crest), or from descending to ascending (sag)—were important in the study of the effect of the profile on accident frequency.

The algebraic difference in grades was used as a common basis for describing the extent of the profile change. In addition, consideration was given to the length of vertical curve, especially where a considerable algebraic difference occurred in combination with a short vertical curve.

Considering the length of roadway under each profile condition, the following rates in terms of accidents per mile were determined for nine freeways: crests—10.7 accidents per mile; sags—12.8 accidents per mile; and tangents—5.1 accidents per mile. The term "tangents" refers to profile sections other than crests and sags.

No steep grades or other unusual grade conditions existed on any of the freeways except at the areas of crests and sags.

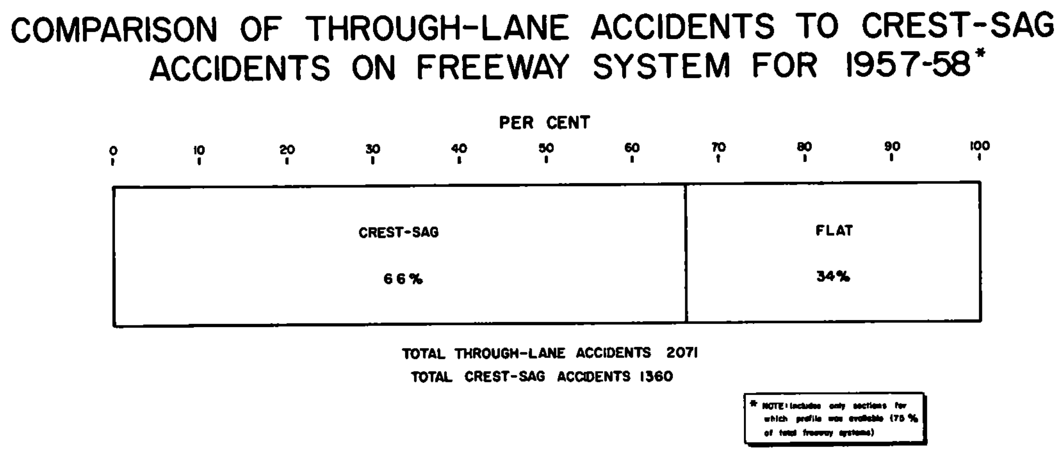


Figure 15. Comparison of through-lane accidents to crest-sag accidents on freeway system.

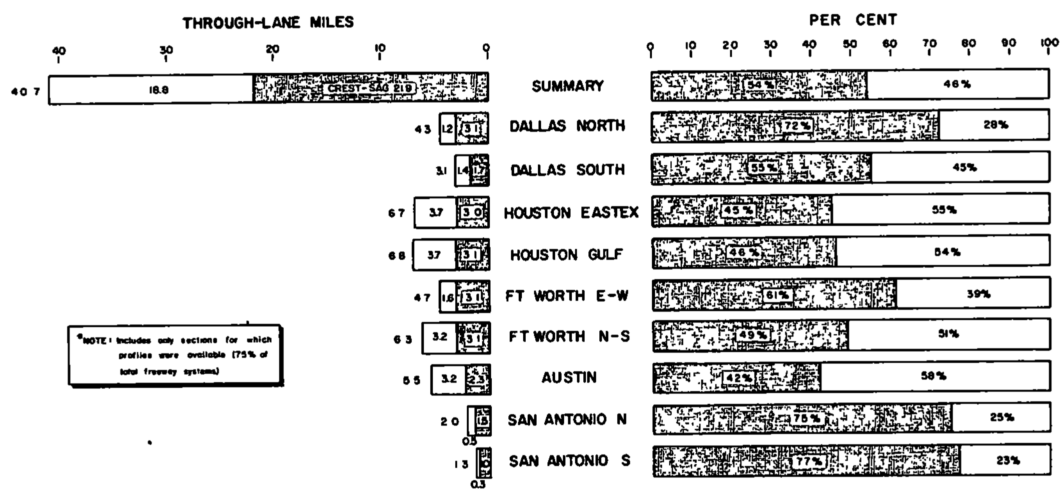


Figure 16. Comparative mileage of through lanes and crest-sag sections (one-way). Note: Includes only sections for which profiles were available (75 percent of total freeway systems).

Table 3 gives the number of crests and sags for all of the freeways studied. Each direction of travel was considered separately. This table also gives the numbers and percentages of accidents by type and the average number of accidents for each location. Considering the accidents per location, there appeared to be no correlation between the algebraic difference in grade and the frequency of accidents.

In considering the effect of sight distance, the numbers of accidents were adjusted to give a rate per 100,000 ADT. This value was then plotted against algebraic difference in grades (Fig. 17a) and against the rate of change of gradient, K (Fig. 17b). Neither of these plots showed any correlation between the number of accidents per 100,000 ADT and the algebraic difference in grades or the rate of change of gradient.

In an attempt to more directly relate effect of sight distance, the accident frequency was related to the algebraic difference, sight distance value, K, and the volume of traffic using the facility. If the sight distance directly contributed to accident frequency, then accident frequency would be directly proportional to algebraic difference and

TABLE 3

TOTAL THROUGH-LANE CREST AND SAG ACCIDENTS BY TYPE AND DIFFERENTIAL GRADE (All Freeways)

Algeb. Diff. in Grade	No. of Locations	Number of Each Type of Accident						Total Accidents	Accid per Location
		R.E.	F.O	S.S	O C	H O.	Pedestrian		
Crests, less than 3%.	2	6	0	0	0	0	0	6	3 00
3 - 4	8	36	4	16	2	2	0	60	7.50
4 - 5	8	47	11	8	4	2	1	73	9.13
5 - 6	16	49	12	24	10	3	1	99	6.19
6 - 7	22	65	23	28	12	4	3	135	6.14
7 - 8	16	30	10	21	2	0	0	63	3.94
8 - 9	28	94	24	16	6	1	1	142	5 07
9 - 10	4	17	5	7	2	1	0	32	5.33
10 - up	20	272	27	36	6	1	3	345	17 25
Summary	124	626	116	155	43	14	9	955	7.70
%		65	12	16	5	1	1		
Sags, less than 3%.	2	8	0	3	2	0	0	13	6.50
3 - 4	4	22	5	7	2	1	0	37	9.25
4 - 5	4	4	1	0	2	1	0	8	2 00
5 - 6	10	27	10	10	8	0	0	55	5.50
6 - 7	8	17	2	8	1	2	0	30	3.76
7 - 8	18	47	12	20	3	1	1	84	4.67
8 - 9	20	70	12	18	4	1	0	105	5.25
9 - 10	6	18	10	4	0	0	0	32	5.33
10 - up	4	31	4	6	0	0	0	41	10.25
Summary	76	244	56	76	22	6	1	405	5.33
%		60	14	19	5	2	0.2		
Tangents	114	476	65	121	28	10	11	711	6 24
%		67	9	17	4	2	1		

volume, and inversely proportional to the sight distance which is dependent on the length of vertical curve for any value of algebraic differences.

$$(\text{number of accidents}) = \frac{\text{ADT}}{100,000} - \frac{(\text{A})}{\text{L}} \ (\text{Factor}).$$

Assumed accident factor = No. of accidents (K/100) x (100, 000/ADT).

This value was then plotted against each of the variables in the expression (Fig. 17c). No straight line or curvilinear relationships appeared to exist, which indicated that no apparent relationship existed between the number of accidents and the sight distance as an independent contributing factor. Data on these locations are given in Table 4.

Table 5 gives the occurrence of accidents by the position on the crests and sags. Also shown are the accident rates per mile for the various elements of the crests and sags. This table again does not indicate any direct correlation between the accident occurrence and these profile features except that the accident rate in terms of accidents per mile is greatest for the peak of crests and is least for the downgrade of crests. The rate of accidents per mile on the tangent sections was less than one-half that on crests and sags.

There were numerous possible interactions between contributing factors that would affect the occurrence of accidents on the crests and sags. For example, a ramp in close proximity would materially affect the accident occurrence on the crest or sag.

Lack of adequate visibility of the traffic stream ahead (sight distance) probably accounted for the high percentage of rear-end and side-swipe accidents at these points of differential grades. The rate of rear-end accidents per mile was 1½ times as high on crests and sags as on the other sections of the through lanes. Because most of the ramps joined the freeway on sections outside the areas designated for crests and sags, most of the ramp-influence accidents were counted on the tangent sections. This substantiates the assumption that the lack of proper sight relations ahead of and behind each driver may well have been a strong contributing factor in accident occurrence.

High Accident Frequency Locations

A special study was made of 14 locations on four freeways where a crest or a sag

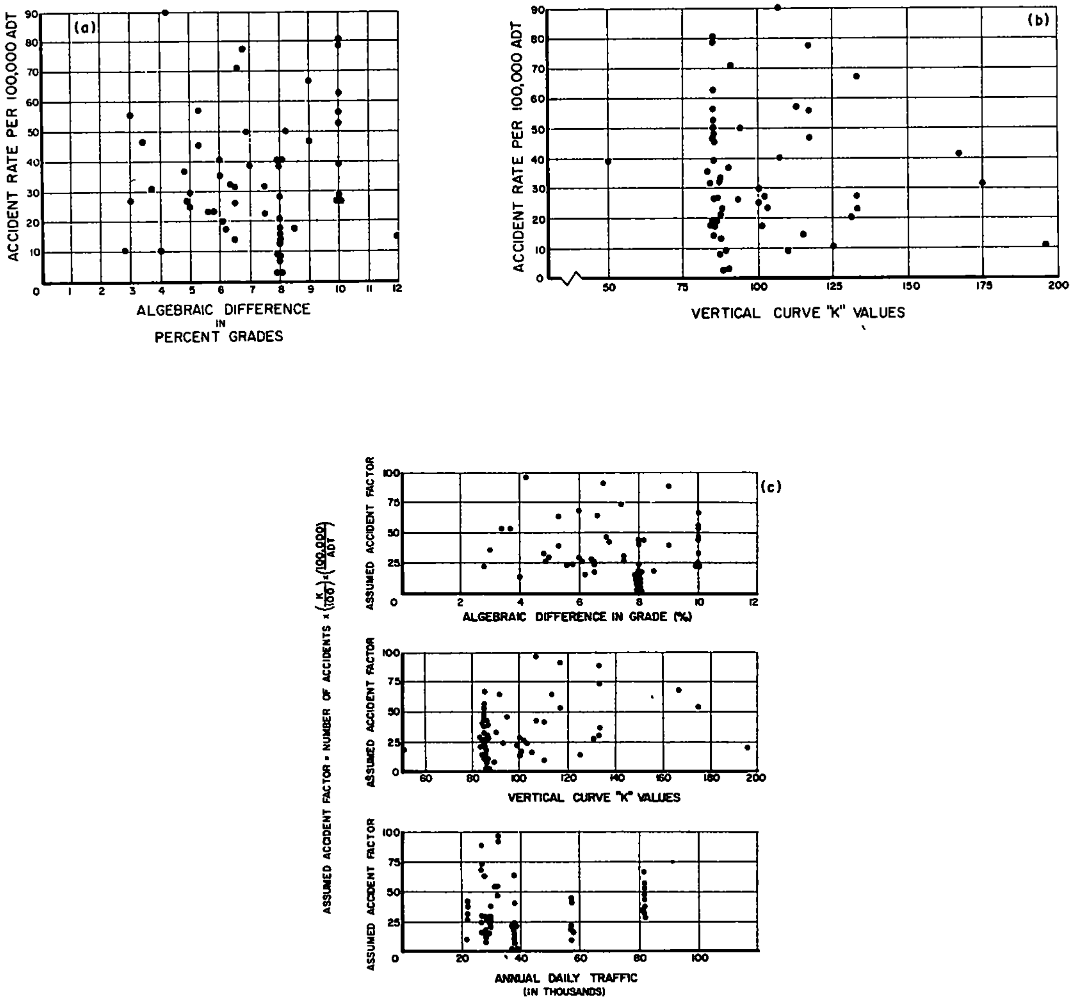


Figure 17. Effect of sight distance on accidents at crest vertical curves.

had 10 or more accidents during 1957 and 1958. The conditions at these 14 locations are given in Table 6. These data are shown in summary in Figure 18.

In addition to actual number and percentages of accidents, tabulation was made of the mileage involved and the volume (ADT) for each of these 14 high accident frequency locations. From these data an exposure factor in million vehicle-miles was calculated for each location; also an accident rate computed by dividing the number of accidents at each location by the corresponding exposure factor.

When based on exposure factor the accident rate for the crests, shows 2.02 per million vehicle-miles, less than the 2.96 rate for the sags. The accident rates by position on grade were 2.33 on upgrade of crest, 1.92 on downgrade of crest and 1.96 on the peak. On sags, 3.57 occurred on downgrade, 2.39 on upgrade, and 2.45 on bottom of sag. The distribution of accidents by position on grade at these high frequency locations was thus much the same as at all crest-sag locations—namely, most on the inclines whether up or down. Especially should it be stressed that the differential grade whether at crest or sag, was high in every high accident frequency case, none less than 5 percent and ranging from 5 to 10 percent.

With respect to horizontal curvature, the respective accident rates were 2.79 for 2 deg or more, 3.48 for 1 deg ± and 2.04 for straight tangent locations. Thus the influence of horizontal curvature on accident frequency seemed to be negligible or doubtful.

Six (43 percent) of the 14 crest-sag high accident frequency locations were at interchanges where the freeway passed under the cross street (sags). Seven (50 percent) were at freeway overpasses (crests). One was a location at considerable distance from any adjacent cross street. The average accident rate per million vehicle-miles for the underpassing freeway sag locations was 2.96 as compared with only 1.73 for the overpassing freeway crest situations. In either case, the prevailing sight conditions for each particular location, such as determined by retaining walls, abutments and bridge structures, added their influence to the effect of differential grade and other factors on accident frequency.

The rear-end type comprised 70 percent of all accidents on the high frequency crest-sag situations, pointing up the driver tendency to follow too closely, as well as unfavorable sight conditions ahead on these locations with high differentials in grade.

Driver violation in following too closely was charged in 44 percent of the accidents in the high accident frequency crest or sag locations, probably constituting a primary or a contributing causative factor.

RAMPS

The accident strip maps of each freeway clearly indicated concentrations of accidents at ramp locations. Accidents occurring on the ramp proper and on the acceleration and deceleration lanes were classified as ramp accidents. There were a total of 416 ramps on the 10 freeways studied. Of 498 ramp accidents during the year 1957 (Table 7), a high percentage (82 percent) occurred on entrance ramps (Tables 8 and 9). Figure 19 illustrates concentration of accidents at relatively few ramps with about one-half of the on-ramps and three-fourths of the off-ramps having no accidents during the study period.

It was assumed that the concentration of accidents at any particular ramp was related to: (a) greater accident potential or exposure, or (b) the geometric design of the ramp and/or ramp area. "Accident rates based on improper exposure values are misleading and can delay the proper correction of accident hazards. There is also a danger that highways of modern design with large traffic volume will be termed unsafe merely because accident rates are not based on proper exposure data" (5).

To arrive at some common denominator or exposure index to facilitate comparison between locations with different accident experience, the following relationships were considered:

1. The chance for accidents is related to the number of conflicts between the through-lane vehicles and the entering or exiting vehicles.
2. The chance for accidents involving only through-lane vehicles is greater in the vicinity of ramps and is related to the number of merging and diverging maneuvers.
3. The chance for accidents involving only the ramp vehicles is related to the ramp volume.

Ramp accident rates in Tables 7, 8, and 9 were calculated by dividing the number of ramp vehicles by each of the following exposure factors: (a) ramp volume, (b) product of ramp volume (ADT) and freeway volume (ADT), and (c) ratio of ramp volume (ADT) divided by freeway volume (ADT). Reports of operational studies (1) showed that the sight conditions and other design conditions probably contributed to the "false-start" rear-end accidents. After considering these operational studies it was reasoned that the chance for accidents was related to the volume of ramp traffic and to the relationship between the ramp and through-lane volumes. The product of the two volumes (ramp and through-lane) yields the same value for high through-lane volume with low ramp volume as for the combination of high ramp volume and low through-lane volume. It would seem reasonable that the exposure is in someway related more directly to the ramp volume because these vehicles create the merging maneuvers. Therefore the ratio of the two volumes was considered significant.

Kinds of Accidents.—Of 498 ramp accidents on the 10 Texas freeways in 1957, TABL

SIGHT DISTANCE FACTORS FOR TEN HIGH A

Location Code No	Freeway Code No	High Rate per 100,000 ADT	High Rate per 100,000 ADT with Respect to Sight Distance	Grades +	Grades -	Algebraic Difference	K	Length of V C	ADT	Number of Accidents in 2 Years
1	4	X	X	5 0	5 0	10 0	85	850	81,800	64
2	4	X	-	5 0	5 0	10 0	85	850	81,800	51
3	4	X	-	5 0	5 0	10 0	85	850	81,800	66
4	5	X	X	3 3	3 5	6 8	117	800	32,300	25
5	5	X	X	2 7	1 5	4 2	107	450	32,300	26
6	3	X	X	1 6	5 0	6 6	91	600	38,000	28
7	6	X	X	5 0	4 0	9 0	133	1,200	26,900	18
8	6	-	X	2 0	1 0	3 0	133	400	26,900	15
9	6	-	X	2 5	3 5	6 0	167	1,000	26,900	11
10	9	-	X	3 5	1 8	5 3	113	600	28,000	16

82 percent were of the rear-end type, 7 percent were side-swipe accidents, 6 percent were fixed-object type of accidents, and the remaining were distributed between out-of-control, angle, and pedestrian accidents (Fig. 20). Of 25 personal injuries, most of them—9 and 11, respectively—occurred in the rear-end and fixed-object types of accidents. Of 4 fatalities, 2 occurred in the out-of-control type of accident and 2 were pedestrian accidents.

Comparison of Accident Rates for On-Ramps and Off-Ramps. —Eighty-two percent of all ramp accidents occurred on on-ramps, whereas only 18 percent occurred on off-ramps. The average number of accidents per on-ramp was 2.03, whereas on off-ramps the average was only 0.40. The average volume product accident rate per on-ramp was 66 per billion compared with 22 per billion per off-ramp. The average accident frequency index per on-ramp was 26.2 as compared with 4.10 for the off-ramps. All of this was despite the fact that the average volume per on-ramp (ADT) was only about 8 percent greater than for the average off-ramp, 2,100 compared with 1950. The accident rates computed on the basis of the number of ramp vehicles without regard to the through-lane volume were much higher for on-ramps: 3.91 per million ramp vehicles on on-ramps as compared to 0.72 for the off-ramps.

In view of the consistency of the results of all these methods of computing accident rates (namely, in showing that the on-ramps have a much higher accident rate than the off-ramps—a finding not in agreement with certain other studies of freeway ramps in other sections of the country), evidently a closer study of the on-ramp design is in order. A later portion of this report presents analysis of a large sample of on-ramps

TABLE 5

THROUGH-LANE CREST AND SAG ACCIDENTS BY POSITION ON GRADE (All Freeways[a])

Position	No of Locations	Number of Each Type of Accident: R.E	F.O	S.S.	O.C.	H O	Pedestrian	Total Accid	Miles	Accidents per Mile	Accid. per Location
Upgrade of crest	124	69	11	16	5	3	1	105	11 4	9 2	0 85
Peak of crest	124	58	14	23	6	2	0	103	5 4	19 0	0.83
Downgrade of crest	124	64	12	26	7	2	0	111	13 0	8 5	0 90
Totals—crest	372	191	37	65	18	7	1	319	29 8	10 7	0.86
Downgrade of sag	76	49	9	11	1	0	0	70	6 8	10.3	0 92
Bottom of sag	76	26	7	4	5	0	0	42	3 4	12 4	0 55
Upgrade of sag	76	68	9	17	3	2	1	100	6.4	15 6	1 31
Totals—sag	228	143	25	32	9	2	1	212	16.6	12 8	0.93
Totals—crest and sag	600	334	62	97	27	9	2	531	46 4	11 4	0 89
Tangents								180	35 0	5.1	
Total through lanes								711	81 4	8 7	

[a]Includes only sections for which profile was available (75 percent of total freeway length) and only accidents plottable on indicated portions of profile

4

CIDENT FREQUENCY CREST LOCATIONS

Accident Rate er 100,000 ADT	Acc $\times \frac{K}{100} \times \frac{100,000}{ADT}$	Distribution of Accidents	Distance in One Direction to Nearest Ramps	Distance in Other Direction to Nearest Ramps	Horizontal Curvature
78 2	66 5	At ends of bridge	1,300'	900'	0°
62 3	53 0	equal	650'	700'	0°
80 6	56 0	equal	600'	950'	0°
77 5	90 7	equal	none	600'	0°
90 0	96 3	at peak and beginning	350' and 650' exit entrance	1,300'	1 5° 1,000° tangent then 3°
71 0	64 6	on 5% grade	1,100'	900'	0°
67 0	89 1	equal	250'	250'	1°
55 8	74 2	equal	700'	700'	0°
40 9	68 3	at ramp	0	none	0°
57 1	64 5	on long 1 8% downgrade	1,000'	1,400'	0°

with high accident frequency and compares them with a similar number of on-ramps with low accident frequency.

Of the 412 accidents on on-ramps, 376 (91.3 percent were of the rear-end type. This is due in part to the dividing of the attention of the entering drivers between on-coming through-lane traffic and vehicles preceding them on the ramp. Several "false-start" accidents occurring on certain ramps have been by chance recorded in motion pictures during the filming of many hours of freeway operation. The false-start accident occurs when a driver preparing to enter the through lanes of the freeway comes to a stop, often quickly and at the very edge of the outside through lanes. At the last minute he has decided to reject a particular gap in the freeway traffic stream. The following driver on the ramp, probably watching in turn for his entering opportunity, assumes the driver ahead is going on in and he takes his attention from the vehicle ahead and a rear-end collision results. The driver of the leading vehicle on the ramp has the sole decision to enter the freeway, to slow, or to stop. Because this decision is possible right up to the time that the leading vehicle enters the outside through lane, it is important that the trailing driver maintain a safe distance behind the leading vehicle and keep the leading vehicle in his line of sight as long as the choice of slowing or stopping is possible. The design of many on-ramp approaches required the driver to look back at a very acute angle in order to evaluate the gap into which he was to move. This diversion of attention violates the principle of safe driving practices which requires the driver to keep a sharp lookout in the direction his vehicle is moving.

Analysis of data and critical examination of on-ramps indicated that the problem is "what the driver sees" as he approaches the freeway rather than simply a matter of sight distance in its common usage. Relative elevations of a ramp and freeway through lanes, grade profiles, horizontal curvature of ramp and through freeway, and the location of structures and other sight restrictions may all be important considerations.

Study of the High Accident Frequency of On-Ramps

A special study (6) of the relationships between ramp accidents and specific geometric design elements of on-ramps was conducted by selecting a group of the highest accident frequency on-ramps and a group of the lowest frequency on-ramps for direct comparison.

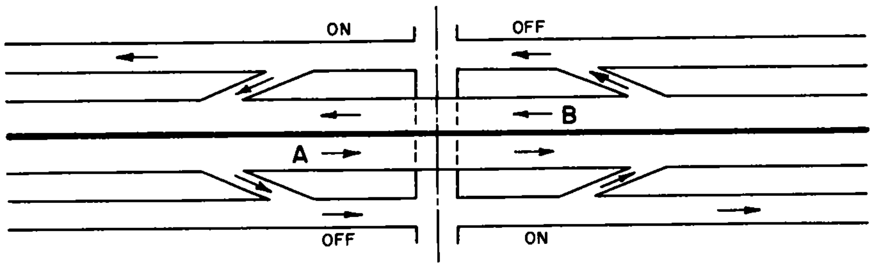
One factor considered in this analysis of selected ramps was the accident frequency on the through lanes in the vicinity of the entrance ramp junctions. Twenty-three percent of all through-lane accidents was found to be in these areas.

Inasmuch as many of the accidents could be plotted by block number only, the through-lane sections considered with each ramp were made to coincide with block lengths. The number of accidents in each of these through-lane sections was adjusted to account for the variable length of the sections. A typical study location is shown in Figure 21.

After considering several different exposure and frequency factors, the following Accident Frequency Index was chosen as a basis for comparing one ramp with another:

TABLE 6

PROXIMITY OF RAMPS TO CREST OR SAG HIGH ACCIDENT FREQUENCY LOCATIONS
(10 or More Accidents on Crest or Sag in 2 Years: 1957-1958)



Location No.	Direction	Algebraic Difference (%)	Length of Crest or Sag (ft)	Length of Crest or Sag (mi)	Distance to Nearest Ramp (ft)		No. of Accidents, 2 Years, 1957-1958
(a) Crests							
1	A	8	2,500	0.47	1,500 off	1,200 off	11
	B				1,850 on	1,500 on	
2	A	8	1,200	0.23	700 off	550 on	24
	B				560 off	720 on	
3	A	7	1,200	0.23	500 off	500 off	18
	B				500 on	500 on	
4	A	5	800	0.15	90 on	550 off	15
	B				500 on	130 off	
5	A	10	1,100	0.21	975 off	600 on	12
	B				600 off	950 on	
6	A	6	1,100	0.21	640 off	440 on	12
	B				1,100 off	700 on	
7	A	8	1,900	0.36	720 off	770 off	12
	B				860 on	500 on	
8	A	6.4	1,150	0.22	500 off	500 on	14
	B				600 off	600 on	
(b) Sags							
9	A	6.7	1,400	0.26	530 on	700 on	22
	B				680 off	580 off	
10	A	8.4	1,200	0.23	600 off	600 on	27
	B				580 off	550 on	
11	A	8	1,050	0.20	850 off	560 on	16
	B				570 off	580 on	
12	A	7.5	1,150	0.22	620 off	730 off	25
	B				670 on	620 on	
13	A	8	1,100	0.21	650 on	625 off	15
	B				675 on	625 off	
14	A	5	900	0.17	500 off	575 on	14
	B				525 off	575 on	

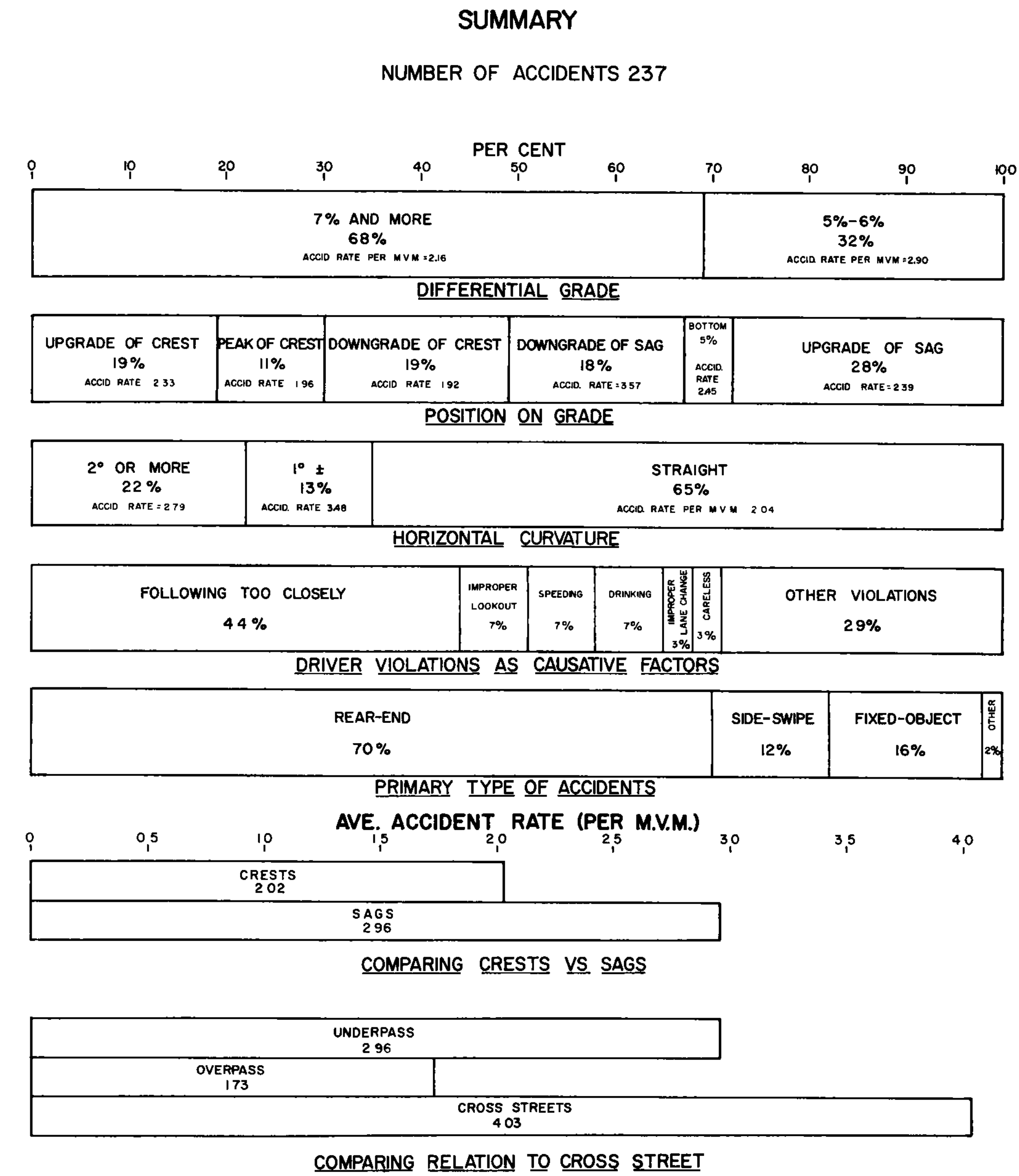


Figure 18. Factors involved in high accident frequency (10 or more in two years, 1957-58): 14 crest or sag locations on 4 freeways.

$$\text{Accident Frequency Index (AFI)} = \frac{VPF+AEF}{(VPF)\,(AEF)}\, A_R + \frac{A_{L_1}}{L_1} + \frac{A_{L_2}}{L_2} + \frac{A_L{}^*}{L}$$

VPF = Volume Product Factor—product of ramp and through-lane volumes.
AEF = Accident Exposure Factor—ratio of the two volumes.
A_R = Ramp Accidents
A_{L_1} and A_{L_2} = Through-lane accidents within L_1 and L_2, respectively (Fig. 16).
A_L = Accidents (which could be) located only by block number.
*L_1, L_2, and L = See Figure 16.

TABLE 7

FREEWAY RAMP ACCIDENTS FOR YEAR 1957

(a) All Ramps

Freeway Code No [a]	No of Ramps	No of Ramp Accidents	Percent of All Accidents	Average Number of Accident per Ramp	Most Accid. of Any Ramp
1	78	113	26	1.45	18
2	26	13	10	0 50	3
3	47	24	10	0 51	7
4	64	154	21	2 41	19
5	51	90	26	1 76	31
6	59	56	19	0.95	9
7	50	25	15	0 50	6
8	17	8	11	0.47	4
9	14	14	26	1 00	3
10	10	1	5	0 10	1
Summary	416	498	20	1 20	31

(b) Ramps for Which Ramp Volume and Through-Lane Volume Were Available

Freeway Code No [a]	No of Ramps	No of Ramp Accidents	ADT Volume per Ramp	Ave. Accid Frequency Index	Vol Prod Factor per Ramp (M)	Ave. Volume Product Accid Rate/(M)	Total Ramp Vehicles (M)	Accid Rate per (M) Ramp Veh
1	78	113	1,805	15 7	39 04	0.031	51.5	2.19
2	20	13	1,420	3 0	15.51	0.028	10.4	1 25
4	40	119	3,660	34.7	129 34	0.024	53.4	2.23
5	29	70	1,865	19 8	32.40	0 100	19 7	3.55
6	35	52	1,765	5.7	17.24	0 069	22 8	2 28
7	32	18	1,334	3.1	13.13	0.032	15 6	1.15
Summary	234	385	2,026	15 2	44 84	0.044	173 4	2.22

[a]See Table 1

TABLE 8

FREEWAY RAMP ACCIDENTS FOR YEAR 1957

(a) All On-Ramps

Freeway Code No.	No of Ramps	No. of Ramp Accidents	Percent of all Ramp Accidents	Average Number of Accid per Ramp	Most Accid on Any Ramp
1	39	108	96	2 77	18
2	12	7	54	0 58	3
3	22	16	67	0 73	7
4	34	130	84	3 53	19
5	25	81	90	3 24	31
6	30	33	59	1.10	9
7	21	20	80	0 95	6
8	8	5	63	0.63	4
9	7	11	79	1.57	3
10	5	1	100	0.20	1
Summary	203	412	.82	2 03	31

(b) On-Ramps for Which Ramp Volume and Through-Lane Volume Were Available

Freeway Code No.	No. of Ramps	No. of Ramp Accidents	ADT Volume per Ramp	Ave Accid. Frequency Index	Vol Prod Factor per Ramp (M)	Ave. Volume Product Accid. Rate/(M)	Total Ramp Vehicles (M)	Accid Rate per (M) Ramp Veh.
1	39	108	1,820	29 6	38 41	0 058	25 9	4.17
2	10	7	1,790	2.8	21.00	0 019	6.6	1 06
4	22	104	3,640	55.2	119.44	0.039	29 2	3.56
5	15	65	1,933	33 6	33.46	0.171	10.6	6 13
6	19	31	1,694	5.3	16.09	0 059	11 7	2.65
7	12	15	1,311	5 0	11 38	0.062	5.8	2.59
Summary	117	330	2,100	26.2	45.13	0.066	89 8	3 75

Complete ramp and corresponding freeway through-lane volume data were available for 69 ramp locations varying from one to four years of data. This resulted in an aggregate of 253 "ramp-year" samples for study. These 253 samples were then categorized into two groups according to high and low accident frequency. From this listing a total of 25 on-ramps having low frequency and 25 high frequency on-ramps were selected to determine if there were any independent geometric features related to accident frequency.

Simple plots were made for each geometric feature for both the high frequency and low frequency ramps. These are shown in Figures 22 through 34. The cumulative

TABLE 9

FREEWAY RAMP ACCIDENTS FOR YEAR 1957

(a) All Off-Ramps

Freeway Code No	No of Ramps	No of Ramp Accidents	Percent of All Ramp Accidents	Average Number of Accid per Ramp	Most Accidents on Any Ramp
1	39	5	4	0 13	1
2	14	6	46	0.43	3
3	25	8	33	0 32	5
4	30	24	16	0.80	4
5	26	9	10	0 35	2
6	29	23	41	0 80	4
7	29	5	20	0 17	2
8	9	3	38	0 33	1
9	7	3	21	0 43	1
10	5	0	0	0 00	0
Summary	213	86	18	0.40	5

(b) Off-Ramps for Which Ramp Volume and Through-Lane Volume Were Available

Freeway Code No	No of Ramps	No of Ramp Accidents	ADT Volume per Ramp	Ave Accid Frequency Index	Vol. Prod Factor per Ramp (M)	Ave Volume Product Accid Rate/(M)	Total Ramp Vehicles (M)	Accid Rate per (M) Ramp Veh.
1	39	5	1,790	1 7	39 67	0 005	25 6	0.20
2	10	6	1,050	3 1	10 00	0 036	3 8	1 58
4	18	15	3,680	9 5	141 44	0 006	24 2	0 62
5	14	5	1,790	5 1	31 27	0 025	9 1	0 55
6	16	21	1,848	6 3	18 60	0 081	11.1	1.89
7	20	3	1,347	1 9	14 17	0 014	9 8	0.31
Summary	117	55	1,950	4 1	44 55	0.022	83 6	0 67

frequency curves indicate the number of on-ramps in each of the two groups at or below any particular value of the element being evaluated.

The geometric features evaluated were:

Ramp Spacing (Fig. 22). —The cumulative frequency curves which compare the spacing to adjacent ramps indicate no effect of ramp spacing on the frequency of accidents at any ramp.

In other research (7), the accident frequency was related to average spacing between ramps. This approach was applied to each of the 10 freeways in this study. Compari-

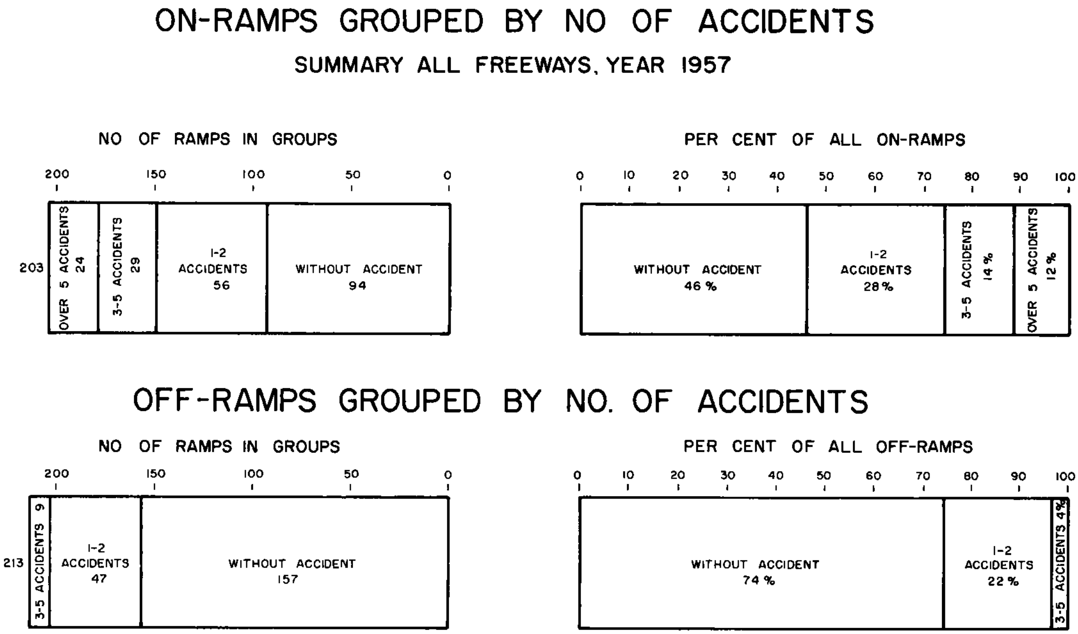


Figure 19. On-ramps grouped by number of accidents.

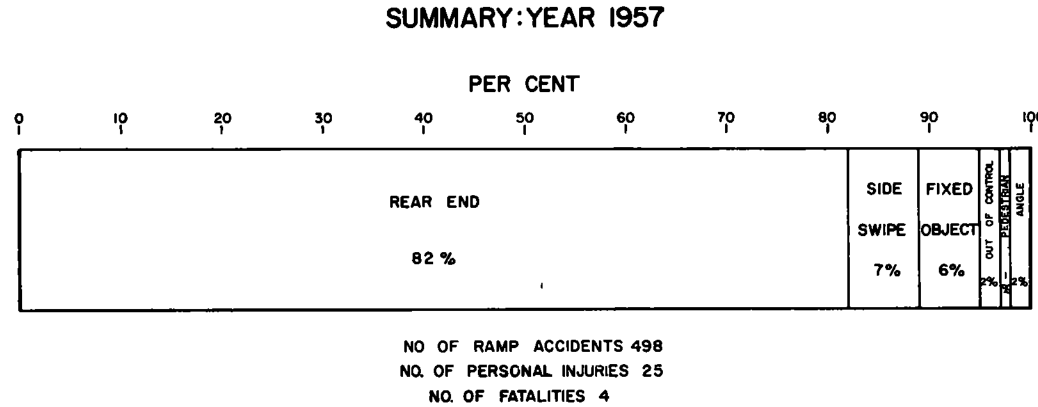


Figure 20. Comparison of ramp accidents by type and by severity.

son of the results of this investigation with the continuous collision diagrams showed the results to be very misleading. A few very long or very short spacings between successive ramps unduly influenced the correlations. The heavier concentrations of accidents at certain ramps appeared to be unrelated to the spacing between successive ramps except that some of the ramps with minimum spacing indicated an influence of this factor.

Length of Ramp (Fig. 23).—The selected high and low frequency ramps showed no correlation between accidents and the length of the ramp.

A grouping of all ramps, both on and off, according to length and the accident frequencies (volume product accident rate) were compared (Fig. 24). There was no indication that ramp length alone was directly related to the frequency of accidents.

Type of Speed-Change Lane (Fig. 25).—On-ramps were classified according to the design of their terminals with the through freeway lanes. Figure 21 shows the number of high and low frequency ramps with each type of speed-change lane. Ramps with long taper design and those with auxiliary lane design (continuous between on-ramp and adjacent off-ramp) are indicated to be more prevalent in the low accident group. It should be noted, however, that the samples of ramps with either long taper or long acceleration lane were too small to be reliable for comparison.

To obtain a greater sample of ramps in each speed-change lane category, all ramps for which volume data were available were compared. Ramps with acceleration lanes and those with auxiliary lanes showed lower volume product accident rate as indicated in Figures 26 and 27 and Table 10.

Junction of Ramp with Through Lanes Related to Vertical Alignment of Freeway (Fig. 28).—The location of the ramp junction for each of the 25 high-frequency and 25 low-frequency on-ramps is shown in Figure 28 in relation to the vertical alignment of the through freeway lanes. There was no strong correlation indicated by this plot.

Distance to Nearest Structure (Fig. 29).—In Figure 29 the distances ahead and back to the nearest structure are evaluated. This cumulative frequency curve shows no indication that this factor by itself was directly related to the accident frequency.

Difference in Elevation and Relative Gradient Between the Ramp and Freeway (Fig. 30).—The fifty selected on-ramps were plotted according to the difference in elevation between the ramp and freeway 200 ft in back of the ramp junction. The relative gradient between the ramp and freeway is also shown. These cumulative frequency curves showed nothing significant about this single factor.

Ramp Width (Fig. 31).—Studies of on-ramp traffic operation (2) have shown that ramps wide enough to permit two-lane operation experienced sluggish and inefficient operation. The widths of the ramps at the ramp nose and 100 ft back from the nose are compared in Figure 31 for the high and low accident frequency ramps. Effect of ramp width is not indicated in these cumulative frequency curves.

Horizontal Curvature of Ramp and Freeway (Fig. 32).—The horizontal curvature of the ramp and of the freeway is shown in Figure 32. The curvature of the high frequency group and of the low frequency group was quite comparable and provided no indication that this factor alone contributed to accident frequency.

Angle Between Ramp and Freeway (Fig. 33).—Operation studies (1) indicated that traffic on-ramps approaching the freeway at very low angles operated with fewer stops and otherwise much smoother flow. The 50 selected ramps illustrated in Figure 33 failed to show any effect of this single factor. None of the ramps studied, however, had what would be termed a "flat" angle of approach; they all aimed the ramp vehicle along a path which encouraged direct entry into the freeway.

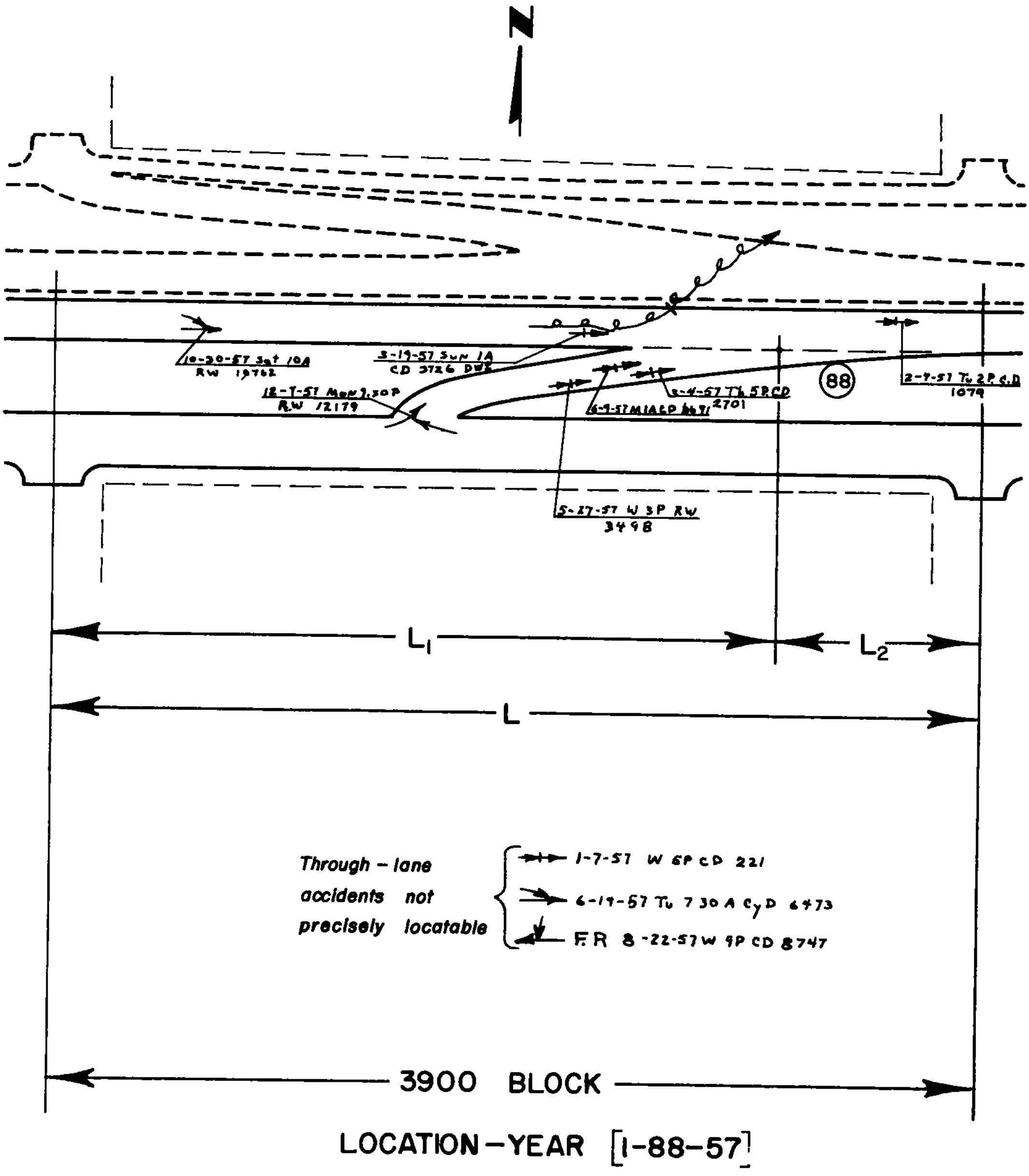


Figure 21. Typical study location.

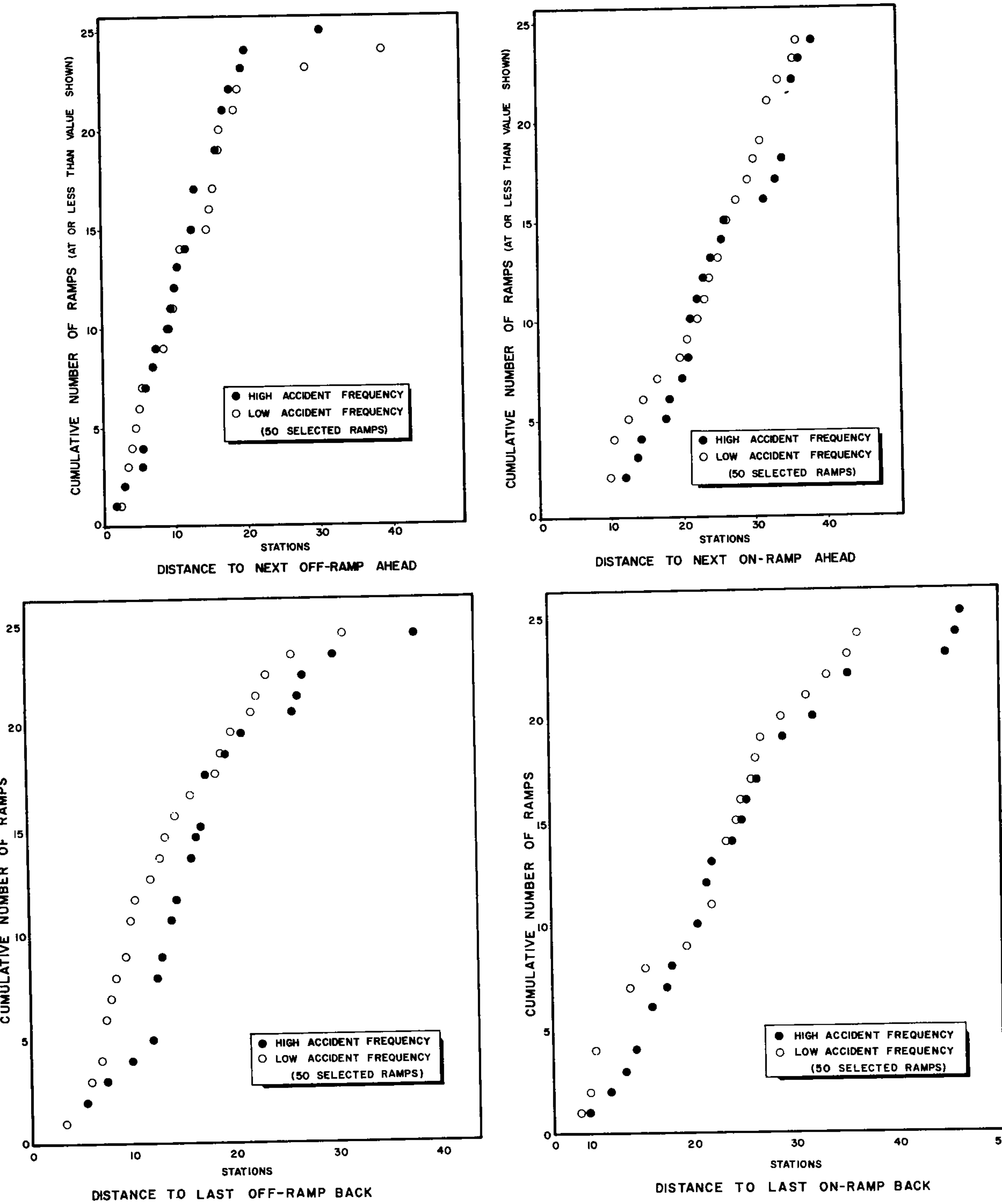


Figure 22. Ramp spacing.

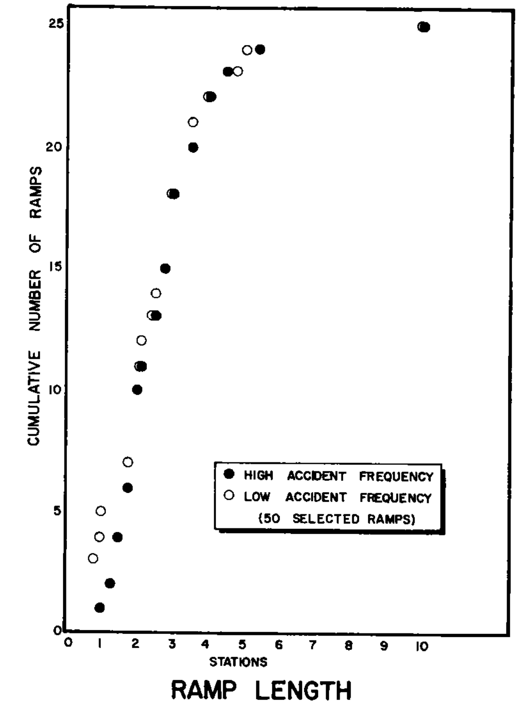


Figure 23.

Volume (Fig. 34).—The freeway volumes at the ramp sites were greater for the high accident on-ramps than for the low frequency ramps as shown in Figure 34. The accident frequency index incorporated several volume and volume-combination factors as well as accident frequency, and served as the basis for selecting the high and low accident frequency categories. The low accident frequency group had accident frequency indices ranging from 0 to 4, while the high frequency group had indices ranging from 70 to 397.

There is no question that the relationships between through-lane and ramp volumes are vital in analyzing the relative safety of various design elements. The accident frequency index (AFI) developed and used in this study is probably not the best for relating the volume and accident characteristics. Additional studies are needed and are recommended to develop a reliable and easily applied index of accident frequency.

Summary of On-Ramp Study

From these plots, no relationships were felt to be strong enough to indicate that a single design element by itself contributed greatly to the accident experience. The complexity of the entering maneuver, coupled with the fact that few if any ramps had identical combinations of geometric elements, invalidated any relationship of a single element with accident experience. Several attempts to determine multiple correlations showed that no particular combinations of design elements were consistent in either the high or low frequency categories.

The closest combination of elements appeared to be these:

High Accident Frequency Ramps.—(a) freeway under structure back, (b) distance back to structure less than about 750 ft, (c) freeway upgrade at the junction of ramp with freeway, and (d) direct entry.

Low Accident Frequency Ramps.—(a) long acceleration or auxiliary lane, (b) distance back to structure greater than about 750 ft, and (c) freeway downgrade at the junction of ramp with freeway.

The sight relationship afforded both ramp and freeway drivers is of primary importance. The highest accident frequency ramps studied incorporated a combination of factors which reduced the effective sight condition afforded either the ramp driver, freeway driver or both (Figs. 35-38).

Off-Ramp Study

Off-ramps had much lower accident frequencies than on-ramps. This was true regardless of the accident frequency rate or index used for comparison.

Off-Ramp Speed-Change Lane Design.—Data for all off-ramps and for those with available volume data are presented in the same manner as for on-ramps (Table 9 and Figures 26 and 27). In examination of Figure 26 where each type of speed-change lane is shown, the auxiliary lanes appear to be no better than the direct off-ramp design. However, as in the case with on-ramps, off-ramps with speed-change lane design showed a lower volume product accident rate than all ramps without speed-change lane design as indicated in Figure 27. The accident frequency on off-ramps was very low. For this reason, the reliability of the comparisons between ramps of various designs is questionable.

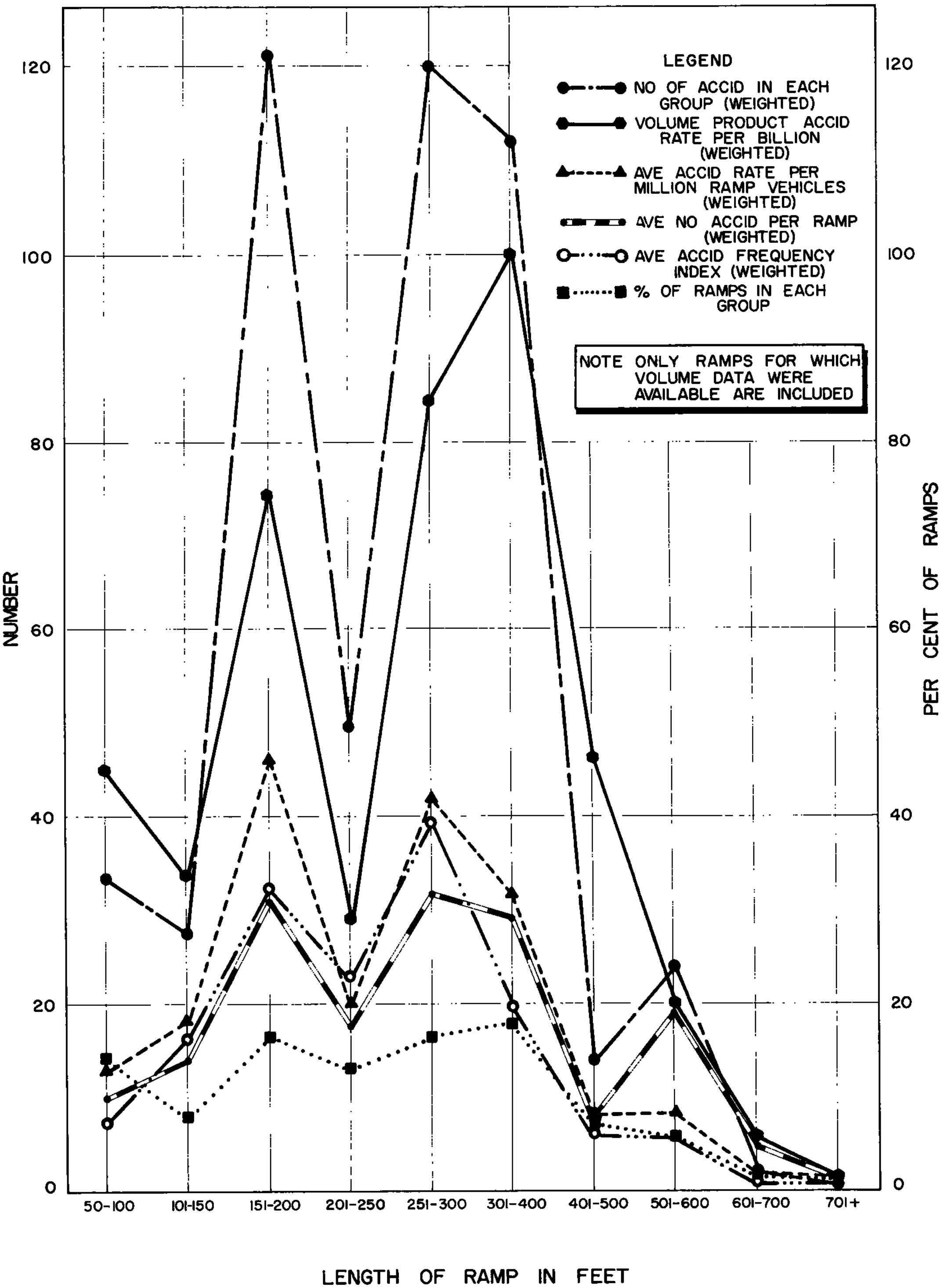
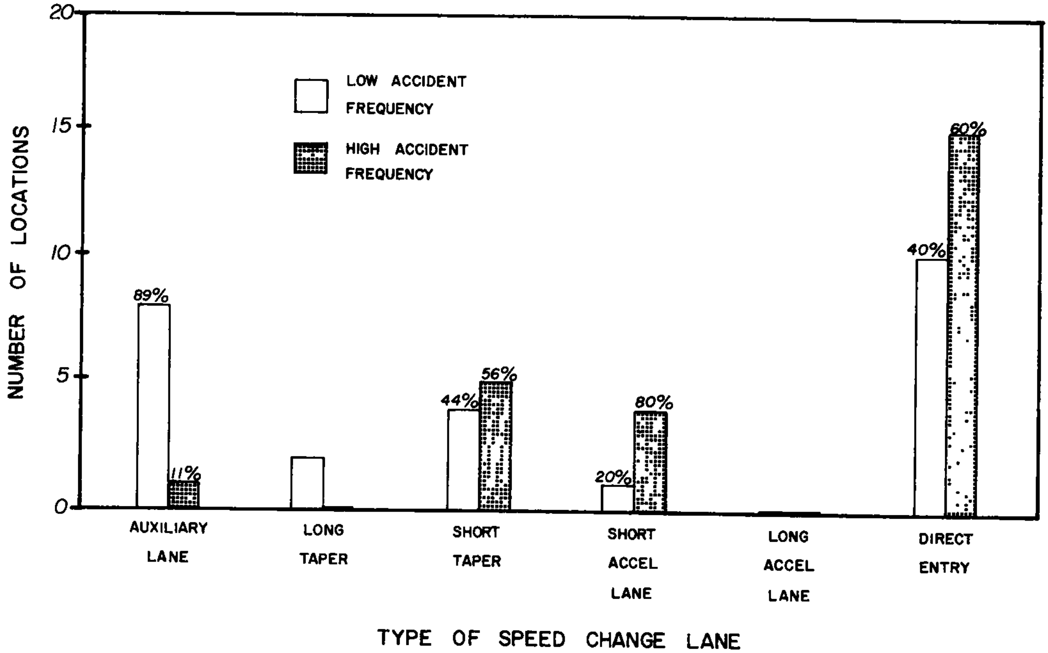


Figure 24. Comparison of ramp accidents by length of ramp.



1. Long taper: distance from ramp nose to end of taper of right curb curb line of ramp > 400 feet.
2. Short taper· distance from ramp nose to end of taper of right curb line of ramp is > 200 feet and < 400 feet.
3. Short acceleration lane: full width of lane; < 300 feet.
4. Long acceleration lane: full width of lane; > 300 feet.
5. Auxiliary lane: an additional lane ahead of ramp extending to the next off-ramp ahead.
6. Direct entry: distance from ramp nose to end of taper of right curb of ramp ≤ 200 feet.

Figure 25.

Comments already made on on-ramps regarding length and spacing between ramps apply in general to off-ramps also. No special study was made of high accident frequency off-ramps, except such as will be reported under "Fixed-Object Accidents."

FIXED-OBJECT ACCIDENTS

Proportion and Severity of Fixed-Object Accidents

Fixed-object accidents accounted for 12 percent of the total freeway accidents and were generally among the more severe accidents (Fig. 39). Therefore, a separate study was made of all accidents involving fixed objects.

It is interesting to note that the fixed-object accident rate per million vehicle-miles was lowest on the two freeways having highest volume and greatest length, averaging 0.250 for these compared to 0.720 for the other eight freeways.

Distribution of Fixed-Object Accidents on General Features of Freeway

Fixed-object accidents were more frequent along the through lanes than on other features of the freeway (Fig. 40). Two-thirds of all fixed-object accidents occurred on the through lanes. Perhaps higher speeds attainable on through lanes were a factor, because speeding was often charged as a violation by one or more drivers involved in fixed-object accidents.

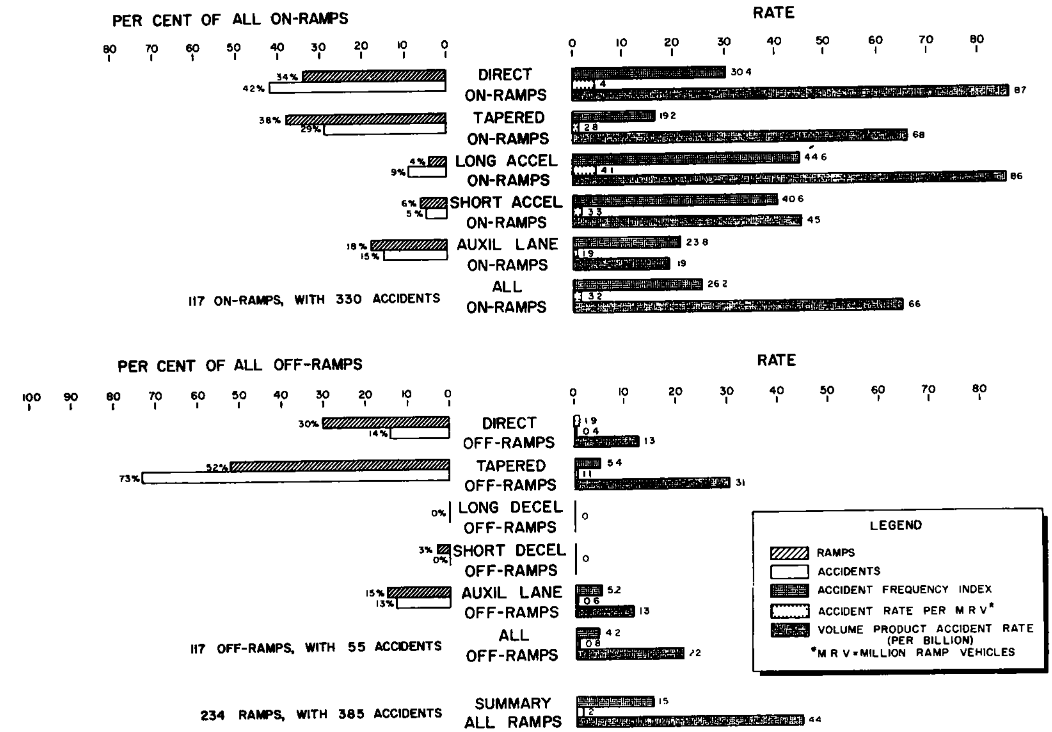


Figure 26. Comparison of ramp accidents by design of ramp terminal, all freeways, 1957.

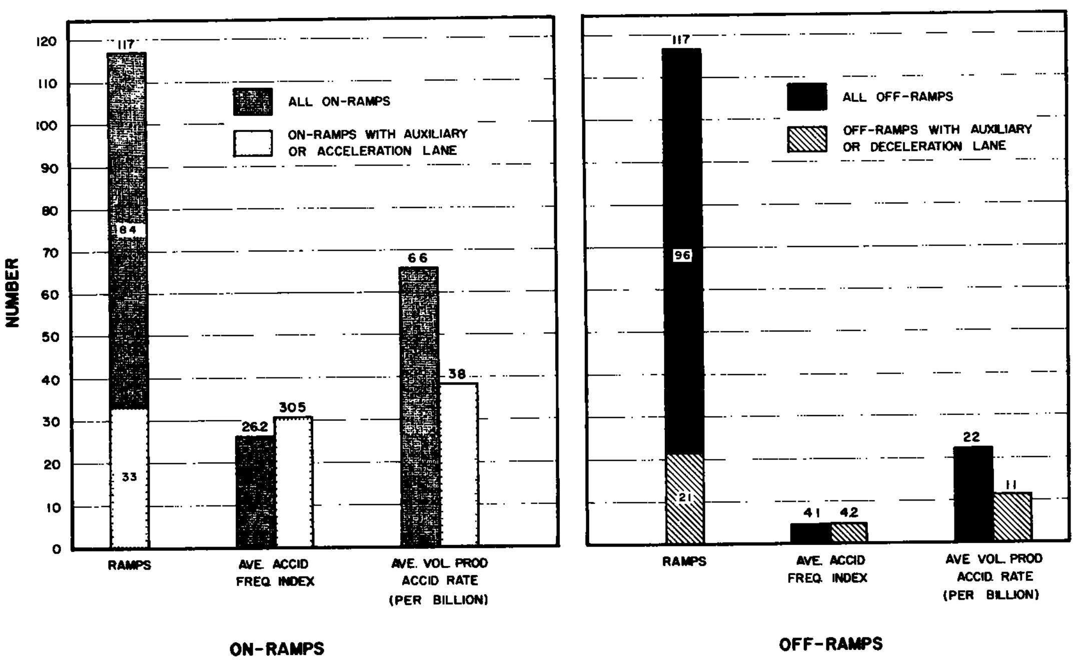


Figure 27. Ramps with speed-change lane compared with all on-ramps and with all off-ramps for six freeways with available volume data, 1957.

TABLE 10

FREEWAY RAMP ACCIDENTS FOR YEAR 1957

(a) On-Ramps with Acceleration or Auxiliary Lane

Freeway Code No	No of Ramps	Percent of All On-Ramps	No. of Ramp Accidents	Percent of On-Ramp Accidents	Average Number of Accid per Ramp
1	4	10	3	3	0 75
2	8	67	4	57	0.50
3	15	68	11	69	0.73
4	17	50	88	66	5 18
5	5	20	9	11	1 80
6	5	17	1	3	0.20
7	9	43	6	30	0.67
8	2	25	0	0	0.00
9	2	29	1	9	0.50
10	3	60	0	0	0.00
Summary	70	30	123	33	1.77

(b) On-Ramps with Acceleration or Auxiliary Lane for Which Ramp Volume and Through-Lane Volume Were Available

Freeway Code No	No. of Ramps	No of Ramp Accidents	Average Number of Accid per Ramp	Ave. Accident Frequency Index	Ave. Volume Product Accident Rate/(M)
1	4	3	0.75	2.5	0.013
2	8	4	0.50	3 0	0 016
4	14	75	5 36	64 4	0.040
5	4	8	2 00	15.3	0.058
7	3	6	2.00	3.7	0.090
Summary	33	96	2.91	30.5	0.038

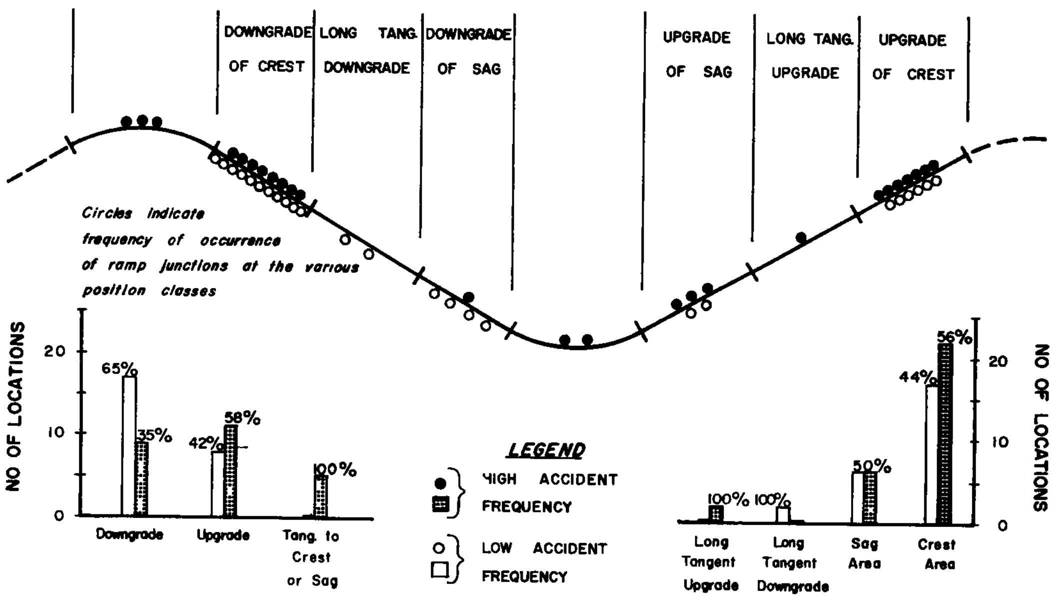


Figure 28. Junction of ramp with through lanes related to vertical alignment of freeway.

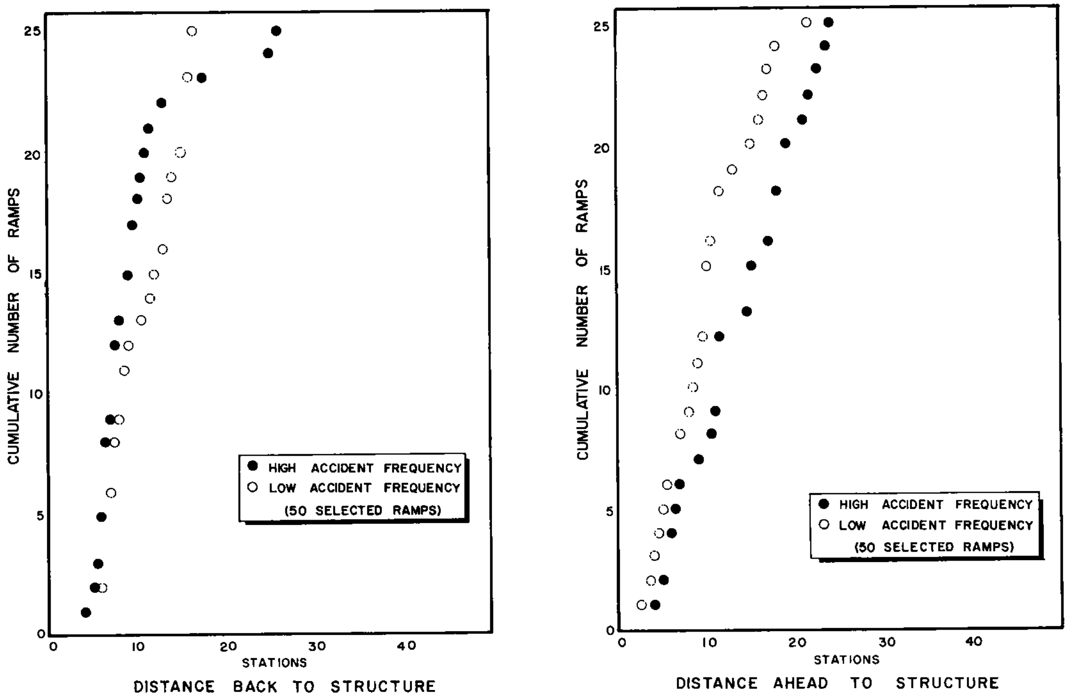


Figure 29. Relationship of ramp to structure.

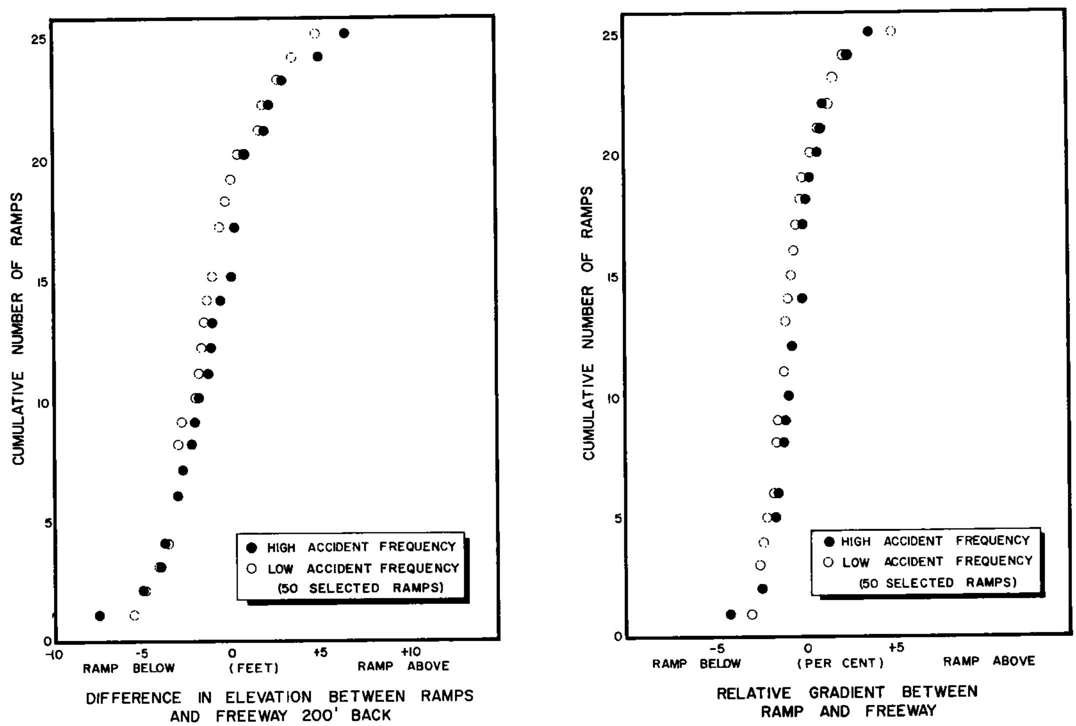


Figure 30.

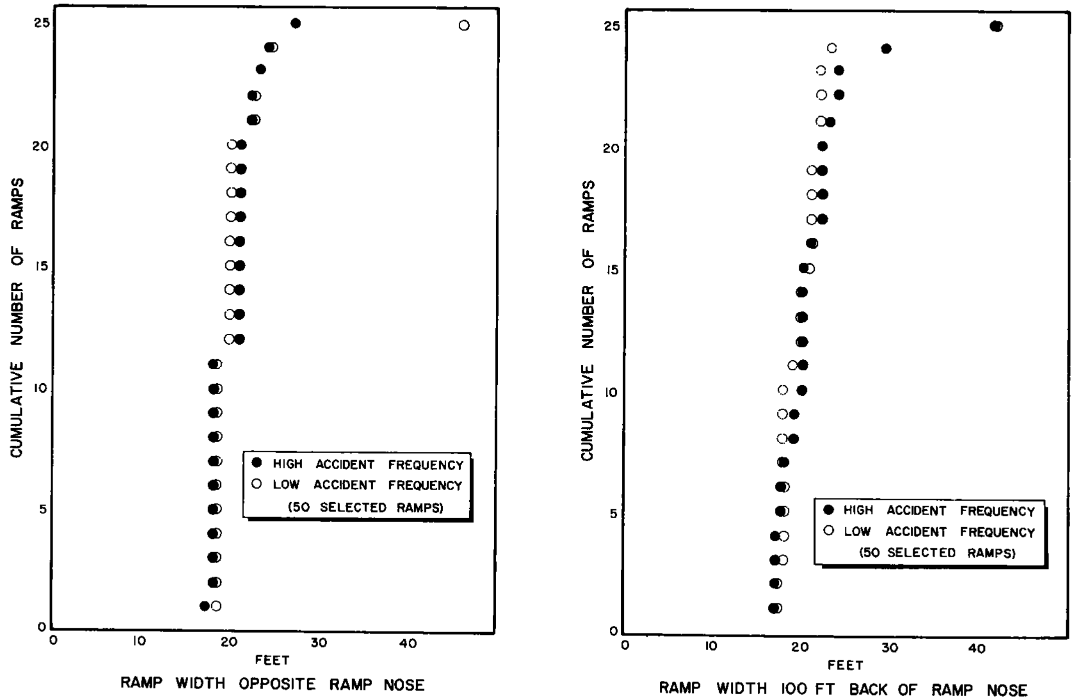


Figure 31. Ramp width.

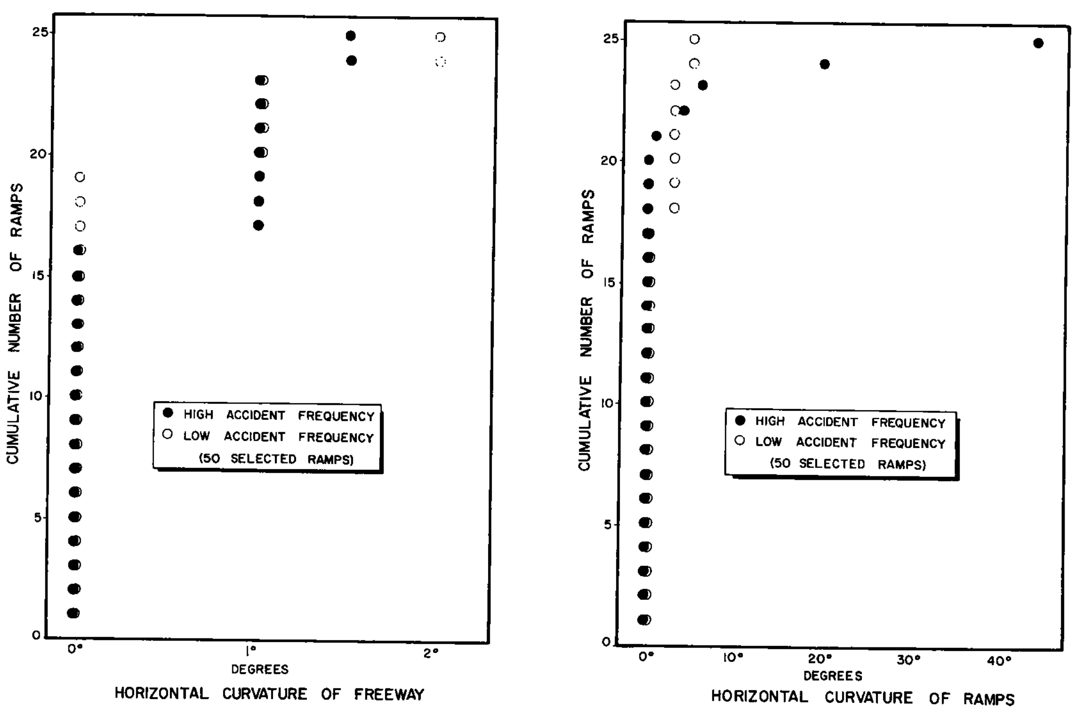


Figure 32. Horizontal curvature.

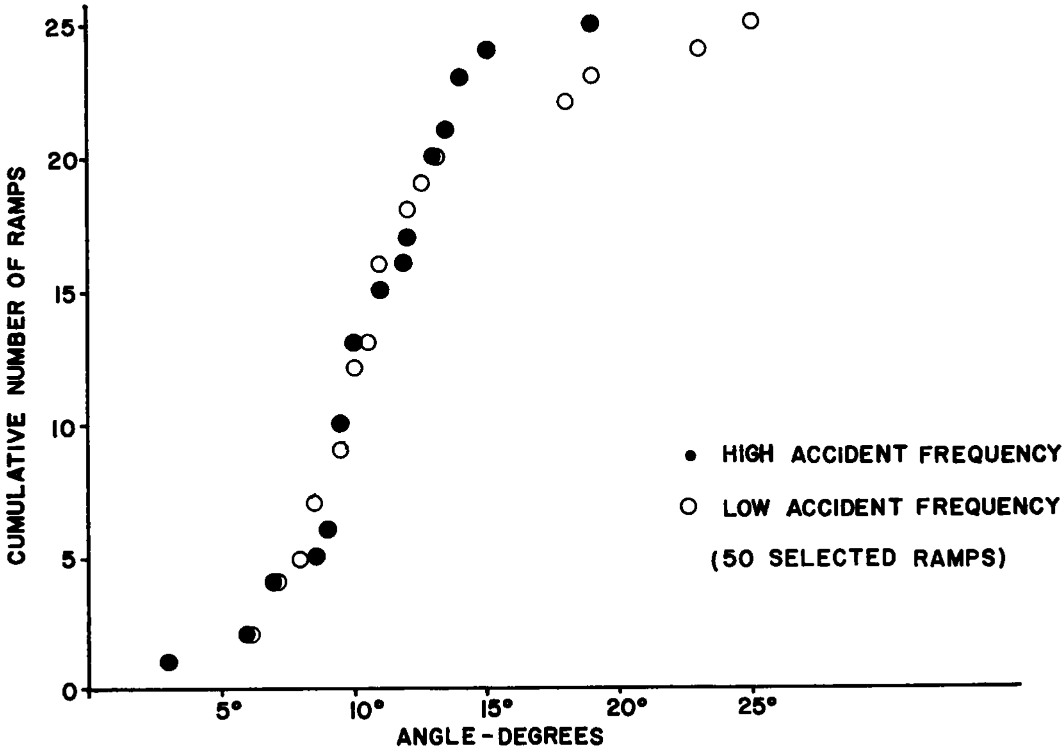


Figure 33. Angle between ramp and freeway.

A search was made along each strip map to determine if any specific fixed object, specific type of fixed object or specific location showed high accident frequency. There were 12 locations where the same fixed object had been struck two or more times in the 2-yr period. Three-fourths of these were located at off-ramps.

Fixed-object accidents on all ramps accounted for 13 percent of all fixed-object accidents. Most of these (70 percent) were on exit ramps (Fig. 41). A study of the strip maps showed that about 56 percent occurred at the freeway end (or entrance) of the off-ramp, 13 percent in the middle of the ramp, 22 percent at the exit end, and 9 percent at an unknown location on ramp. There appeared to be no correlation as to the specific design of the ramp itself. However, the frequency of fixed-object accidents was somewhat higher in those instances where the off-ramp was located beyond and relatively near a freeway overpass.

There was no correlation for the fixed-object accidents on entrance ramps.

The fixed object was the primary contributing factor in all of the off-ramp fixed-object accidents, whereas the fixed object was secondary in one-third of the entrance or on-ramp accidents. The fixed object was considered secondary if the vehicle was in collision with another vehicle prior to striking the fixed object on the ramp.

Types of Fixed Objects and Frequency of Collision

The types of fixed objects most frequently struck were median curbs and median guardrails, other curbs, sign posts, and luminaire standards as indicated in Figure 42.

The Median and Fixed-Object Accidents

The median was involved in 34 percent of the 630 total fixed-object accidents. Of the 214 median accidents, 22 percent struck a median guard fence. Where no guard fence was present, 40 percent crossed the median, nearly one-half of them striking a car in the opposing lanes, resulting in 42 injuries and 3 fatalities. A "before" and "after" study of a median guard fence has been previously reported (4). Figure 43 and Table 11 show the change in median design and the accident experience during the four years of that study. The average daily traffic at one of the highest volume locations on this freeway increased from 69,000 to 81,000 during the time. Although the total accident rate per 100 million vehicle-miles increased (195.94 before to 232.93 after), the rate of the severe accidents decreased slightly (personal injury 26.33 before to 24.34 after and fatal 2.63 before to 2.01 after).

The median accident rate was only slightly reduced from 13.56 before to 11.71 after. By contrast, the severity of the median accidents was materially reduced. There were 4 fatal median accidents before compared with none after, and 28 personal injury accidents involving the median before compared with 11 after.

Study of 172 fixed-object accidents involving the medians in the study of the 10 freeways revealed few placed along the medians where a concentration of fixed-object accidents was noted (Fig. 44).

Frequency of fixed-object median accidents seemed to have little, if any, relationship to the variations in horizontal curvature or profile grade of the freeway, except possibly where grades of 3 percent or more were concerned.

Lateral Curbs

Analysis of 148 fixed-object accidents involving lateral curbs showed no positive influence of horizontal curvature or profile grade of the freeway through lanes (Fig. 45).

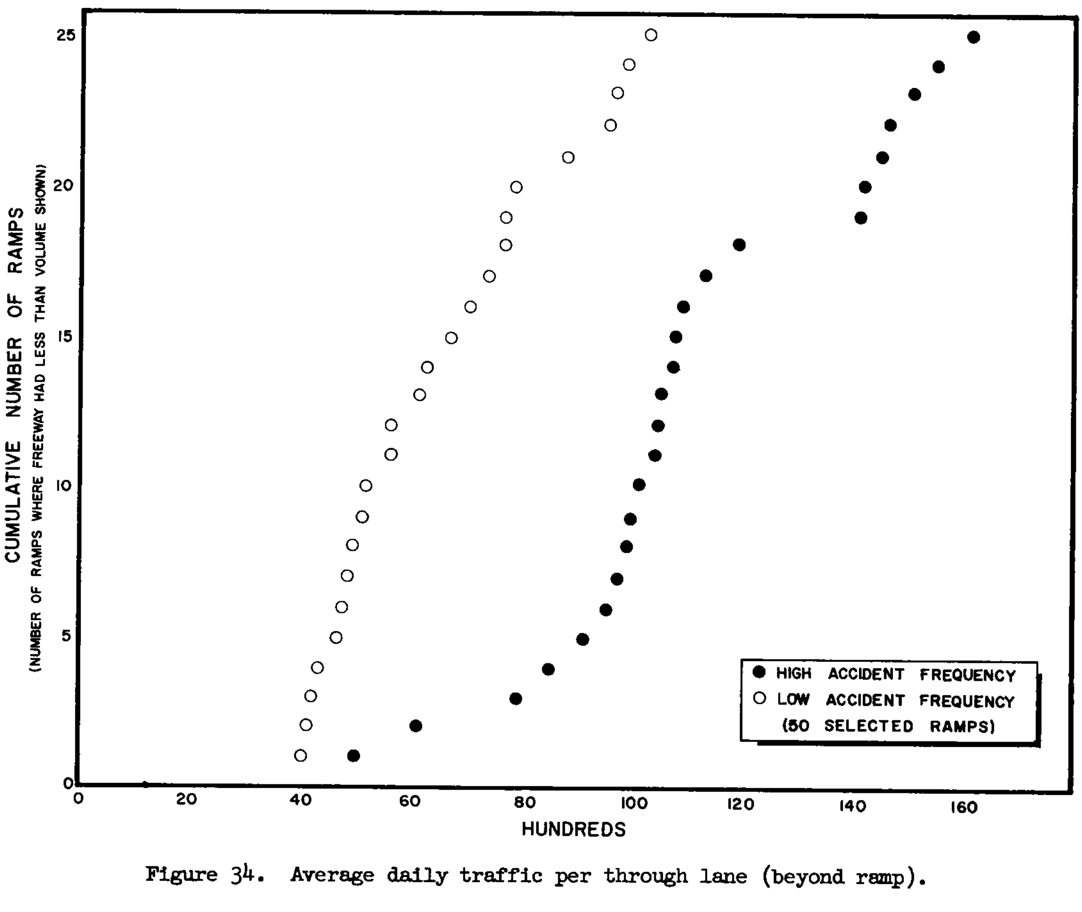


Figure 34. Average daily traffic per through lane (beyond ramp).

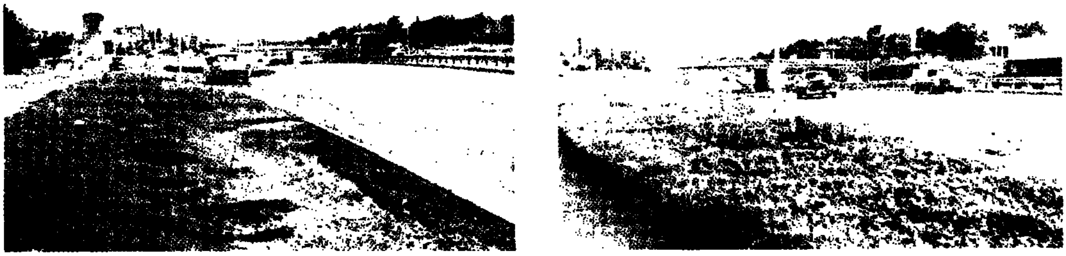


Figure 35. On-ramp outbound from Fitzhugh Avenue, North Central Expressway, Dallas: Left— looking back from end of ramp terminal opening, and right—driver's view of freeway looking back from ramp.

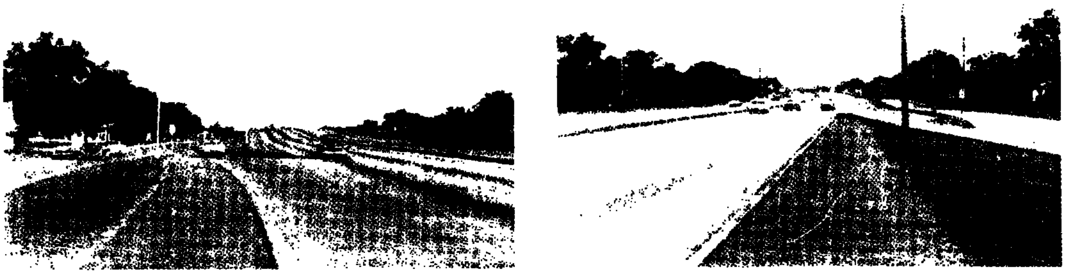


Figure 36. On-ramp inbound from Metropolitan Drive, South Central Expressway, Dallas: Left— view from auxiliary lane, and right—view from freeway.

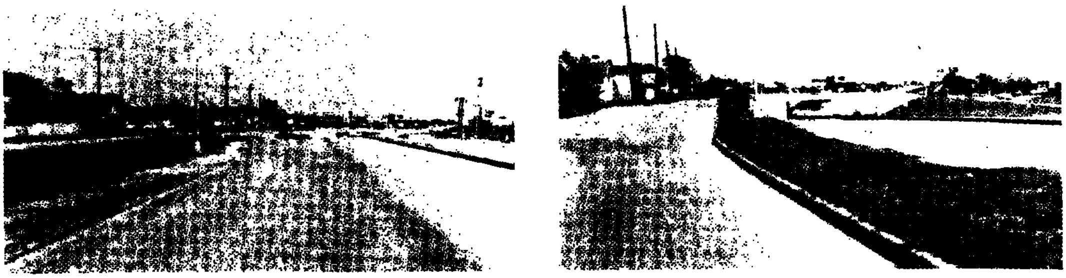


Figure 37. On-ramp outbound from Horne Street, East-West Freeway, Fort Worth: Left— view from auxiliary lane, and right— driver's view of freeway looking back from ramp.

Figure 38. On-ramp outbound from University Drive, East-West Freeway, Fort Worth: Left— driver's view of freeway looking back from ramp, and right— view from freeway.

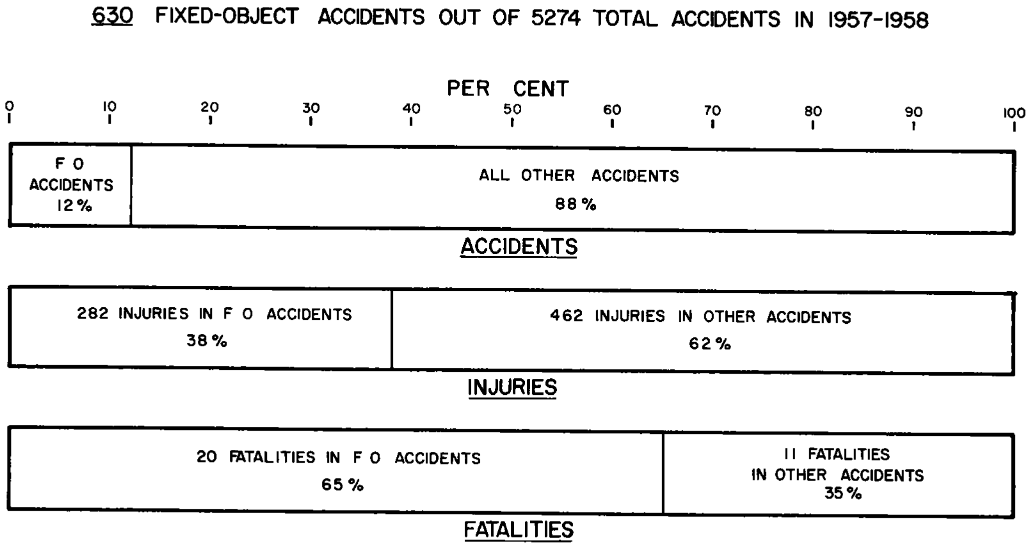


Figure 39. Comparison of number and severity of fixed-object accidents to all accidents.

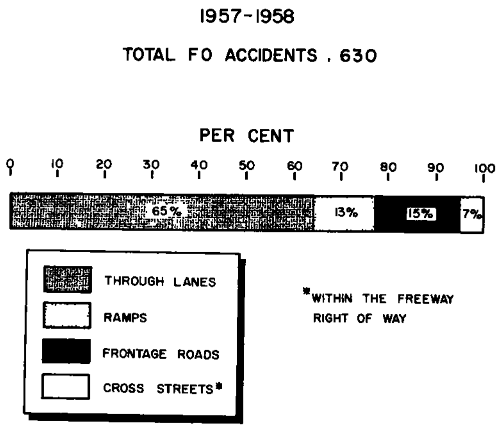


Figure 40. Comparison of fixed-object accidents by freeway features: all freeways.

Review of Diagrams

A review was made of accident diagrams at the 12 locations where the same fixed object or the same design element of the freeway was struck two or more times in the 2-yr period. Four examples of such diagrams are given in Figures 46, 47, 48, 49. The review of the 12 diagrams showed the following:

1. Most fixed-object accidents occurred where there was a significant change in direction of the normal vehicular path. On four of the nine off-ramps included in the 12 special locations, the angle of ramp divergence from the through lanes was large—21 to 33 deg.

The profile grade of each of these four ramps was downgrade about 4 percent. In addition, two off-ramps in this group of nine had rather steep upgrade slopes (4 percent).

2. In four of the off-ramps involved, the profile of the outside freeway lane

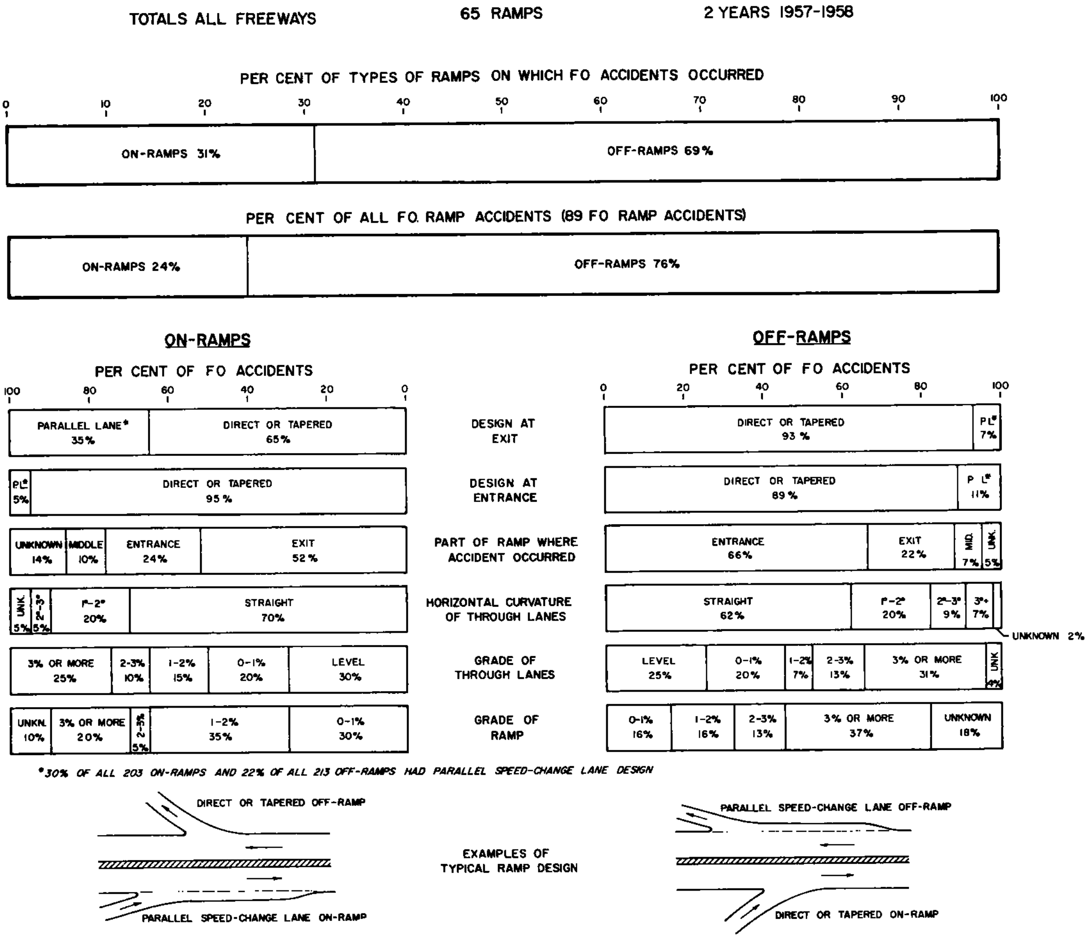


Figure 41. Design features related to fixed-object ramp accidents.

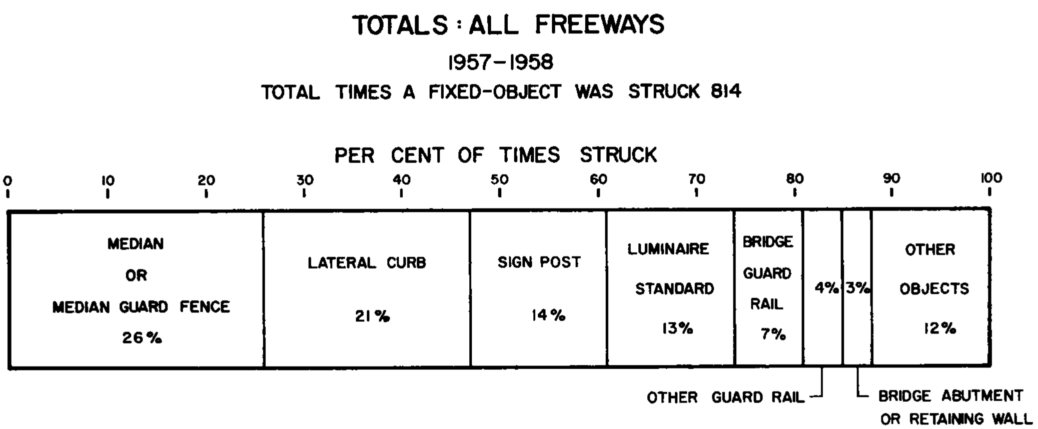


Figure 42. Type of fixed object struck.

TABLE 11

ACCIDENT DATA, GULF FREEWAY, HOUSTON (1954 to 1958)

Type of Accident	Property Damage	Personal Injury	Fatal	Total
Main-lane freeway (no/100 mil veh-mi):				
Before	166.98	26.33	2.63	195.94
After	206.58	24.34	2.01	232.93
Median (no.):				
Before	15	28	4	47
After	34	11	0	45
Median (no./100 mil veh-mi):				
Before	-	-	-	13.56
After	-	-	-	11.71

approaching the ramp was downgrade followed by a steeper downgrade on the ramp. The combination of continuing downgrades and of the change in direction of vehicular path was thought to accentuate difficulty of sight distance, and of control and maneuverability of the vehicle. Three combinations of consecutive upgrades on through lane and ramp were also noted in these locations where the same fixed object was struck repeatedly.

3. Seventy percent of the repeatedly-struck, fixed-object accidents occurred at night, whereas only 26 percent of the average daily traffic occurred during this period (Table 12). Figure 50, indicating the general fixed-object accident frequency related to hours of the day and thus approximately to hours of daylight and darkness, confirms this data.

(Although 82 percent of all fixed-object accidents occurring at night were under streetlight conditions, the general conclusion may be drawn from the foregoing data

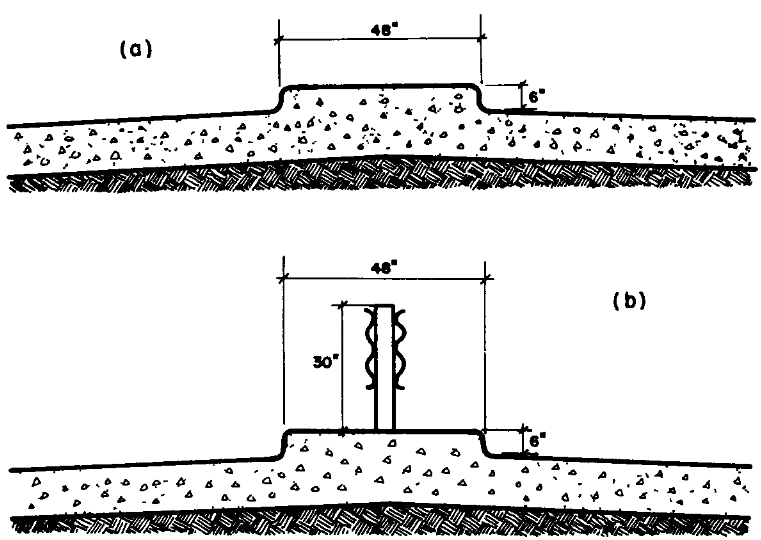


Figure 43. Median sections, Gulf Freeway, Houston: (a) section before construction of barrier fence, and (b) section after construction of barrier fence.

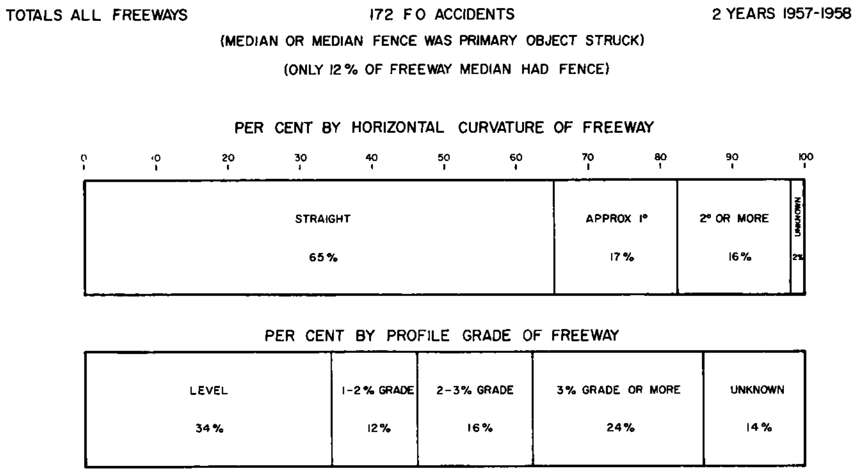


Figure 44. Fixed-object accidents involving the median or median fence.

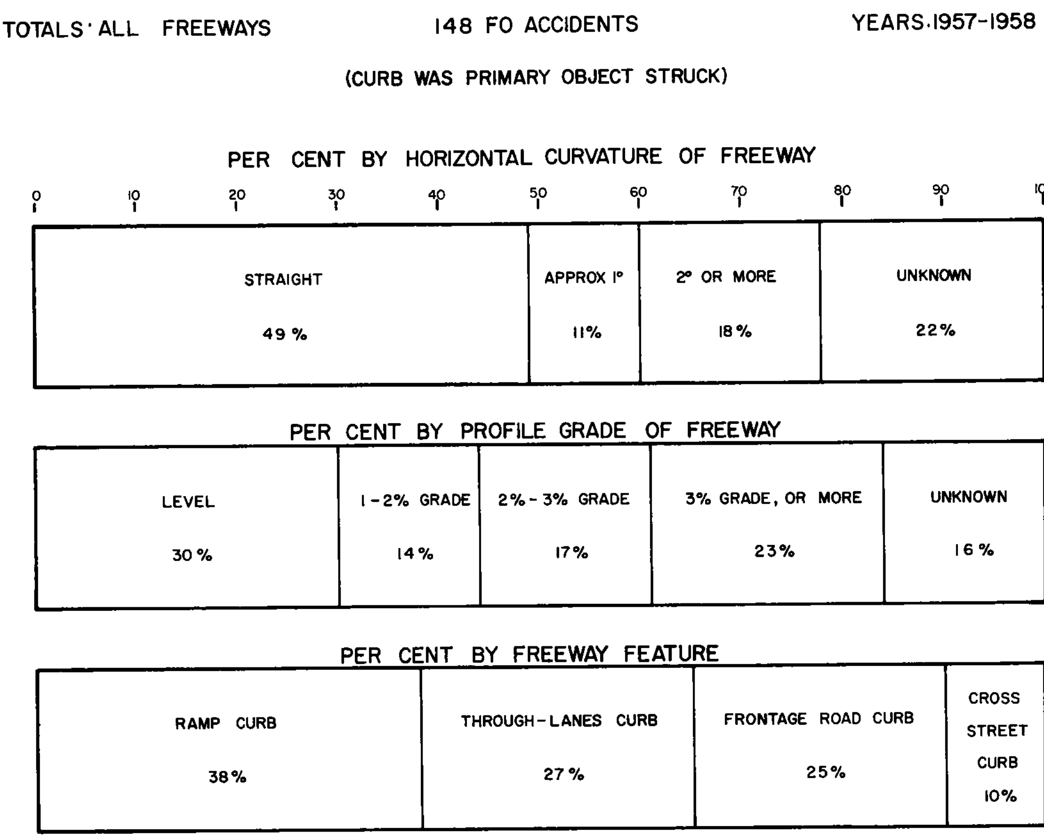


Figure 45. Fixed-object accidents involving lateral curbs.

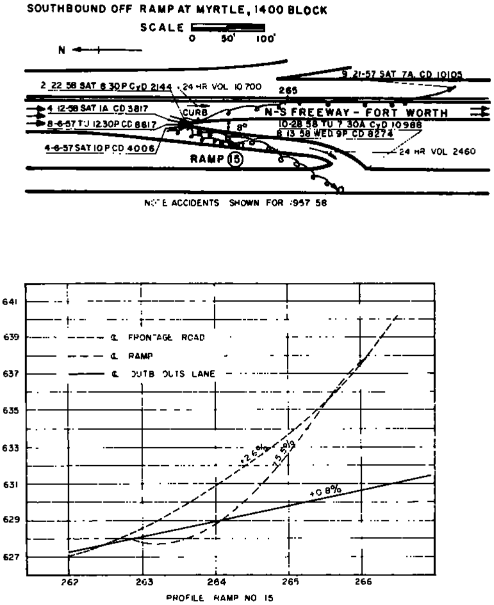


Figure 46.

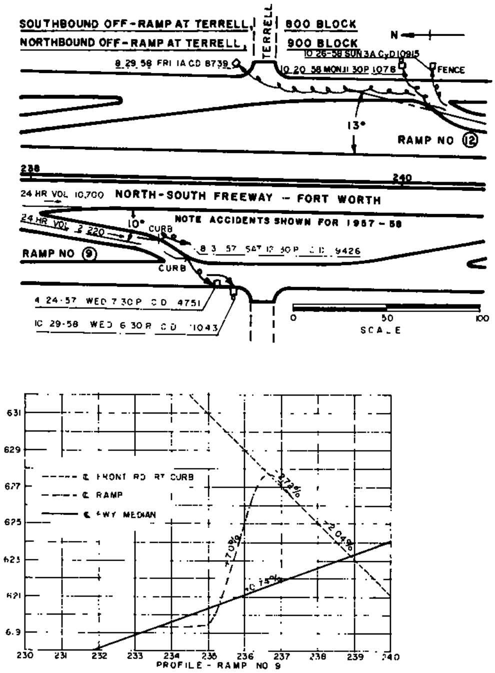


Figure 47.

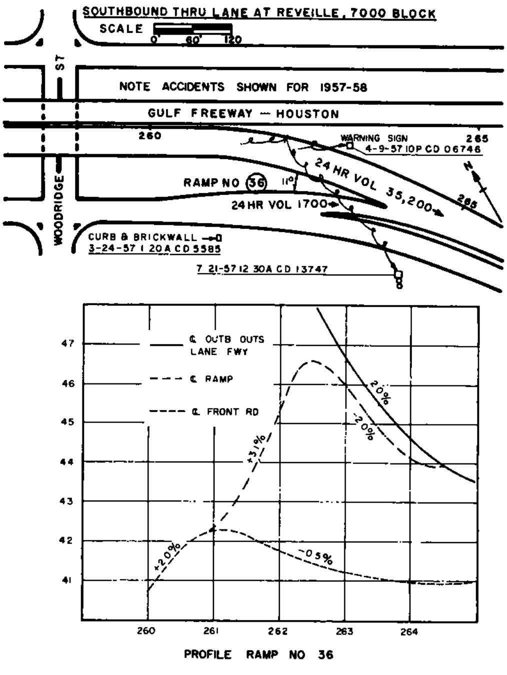


Figure 48.

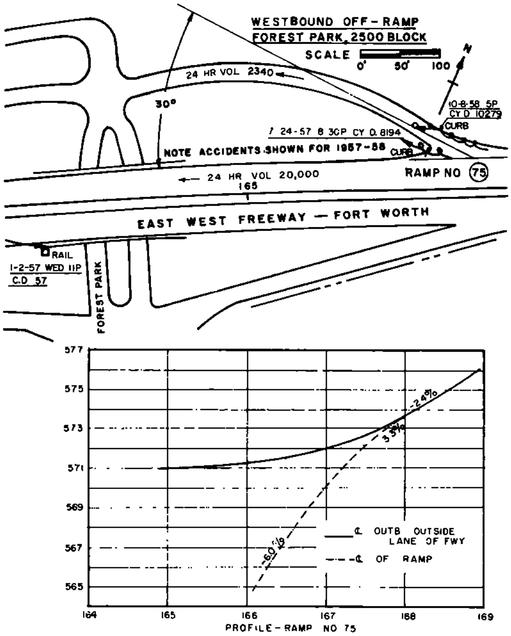


Figure 49.

TABLE 12

FIXED-OBJECT ACCIDENTS AT TWELVE HIGH FREQUENCY LOCATIONS
(Total: 54 Accidents)

Item	Time			
	Day		Night	
	7 AM - 12 N	12 N - 7 PM	7 PM - 12 M	12 M - 7 AM
No. of accidents	6 (11%)	10 (19%)	21 (39%)	17 (31%)
No. by weather:				
Clear and dry	6	10	19	16
Raining and wet	0	0	2	1
DWI	0	0	1	2
Fixed object, secondary*	1	0	0	0
Percent of ADT	30	44	17	9

*In studying all fixed-object accidents on all the freeways for the years 1957-1958, it was noted that the fixed object determined the primary collision type in 90 percent of the cases.

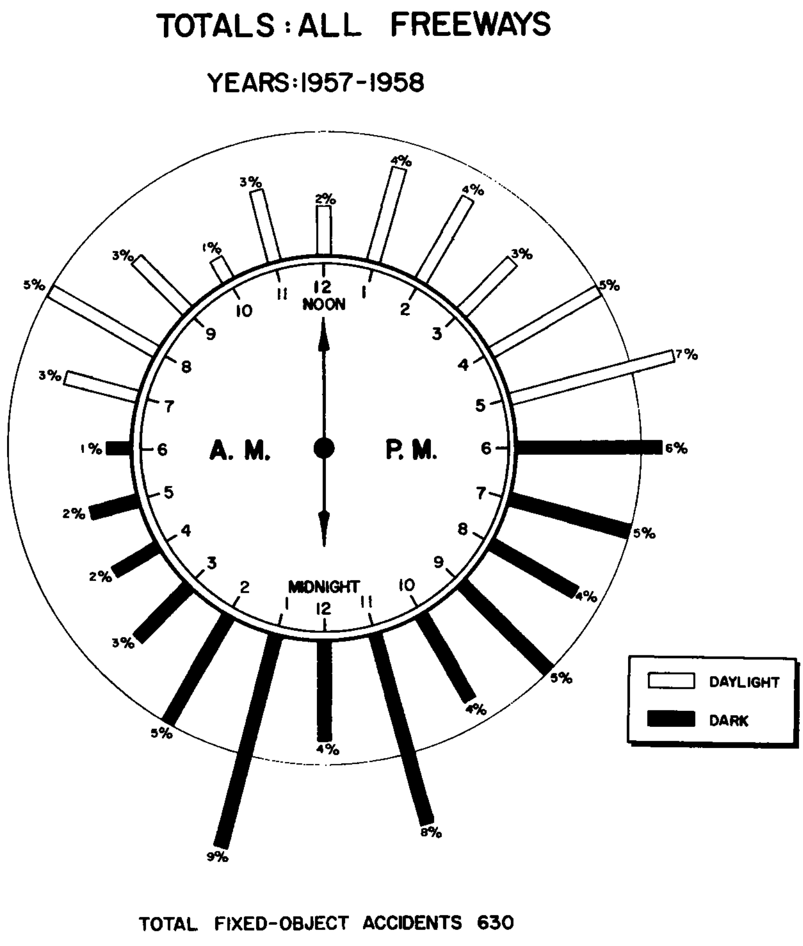


Figure 50. Hour of day for fixed-object accidents.

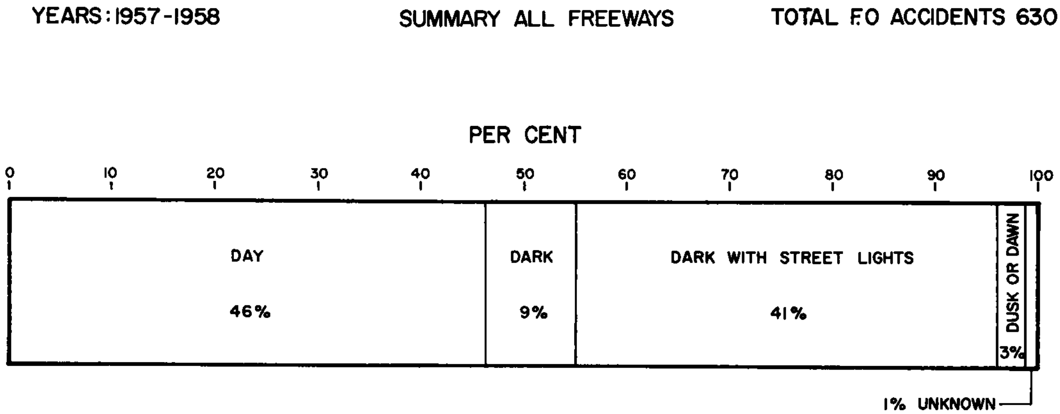


Figure 51. Light conditions in fixed-object accidents.

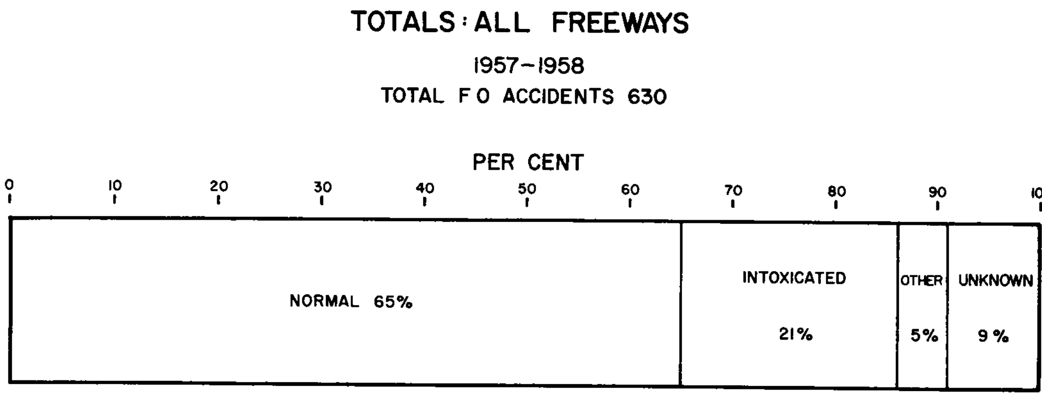


Figure 52. Driver condition (as determined from the report of the investigating officer) in fixed-object accidents.

that there is an over-all problem of night visibility. Figure 51, showing the light conditions in all fixed-object accidents substantiates this conclusion.)

4. It is believed that driver condition enters into the picture much more significantly than is indicated by Figure 52 based on data from the report forms of all fixed-object accidents. Note from Table 12 that 31 percent of the fixed-object accidents in the 12 special locations occurred between midnight and 7 a.m., when drivers are prone to be tired, sleepy, and in some cases experiencing the after-effects of the earlier social hours.

5. There is no apparent influence from rainy weather or road surface conditions indicated by fixed-object accidents at the 12 special locations nor by the data from all fixed-object accidents as shown in Figure 53.

To strengthen the opinion previously expressed that driver condition and behavior were often important contributing factors in fixed-object accidents, along with design features or elements, Figures 54, 55, and 56 are presented. Speeding and drinking were significantly specified in these data taken from the police officers' reports. Saturday and Sunday may be noted as days of the week when fixed-object accident frequency was somewhat higher. These are days when drinking and speeding or other reckless driving are commonly known to be more prevalent. Resulting lack of control of the vehicle is obviously a strong factor in fixed-object accidents.

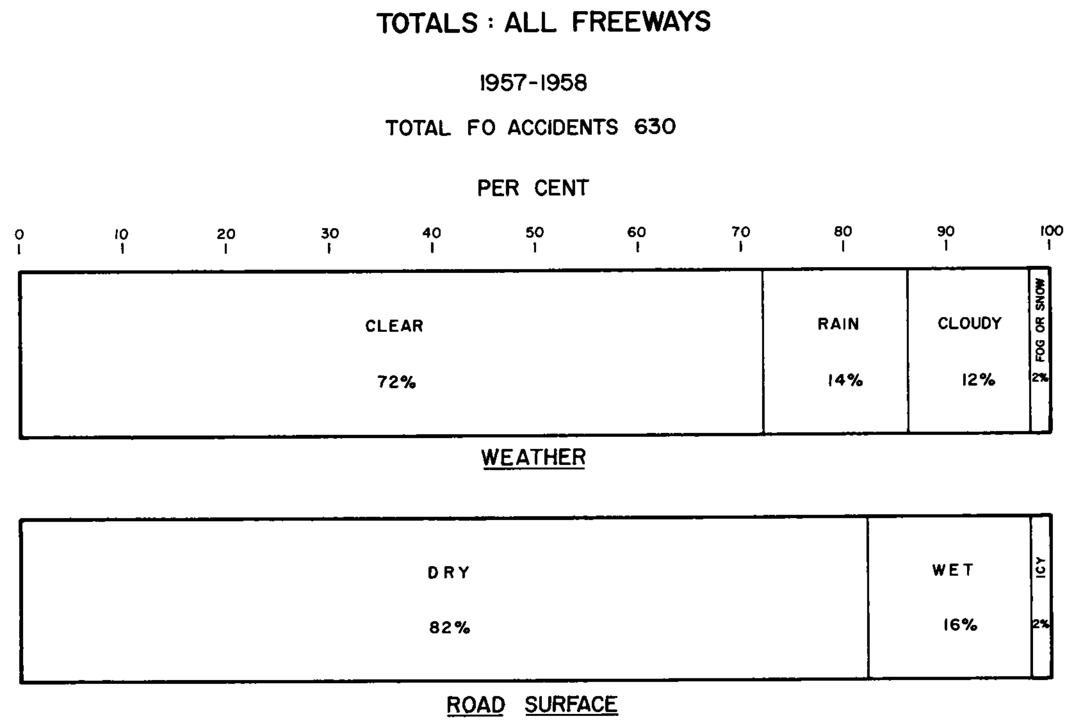


Figure 53. Condition of weather and road surface in fixed-object accidents.

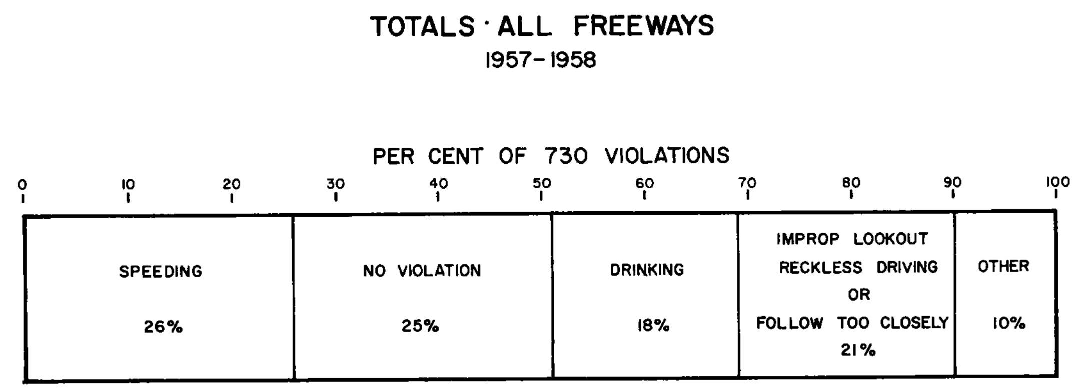


Figure 54. Drivers' violations (as determined from the report of the investigating officer) in fixed-object accidents.

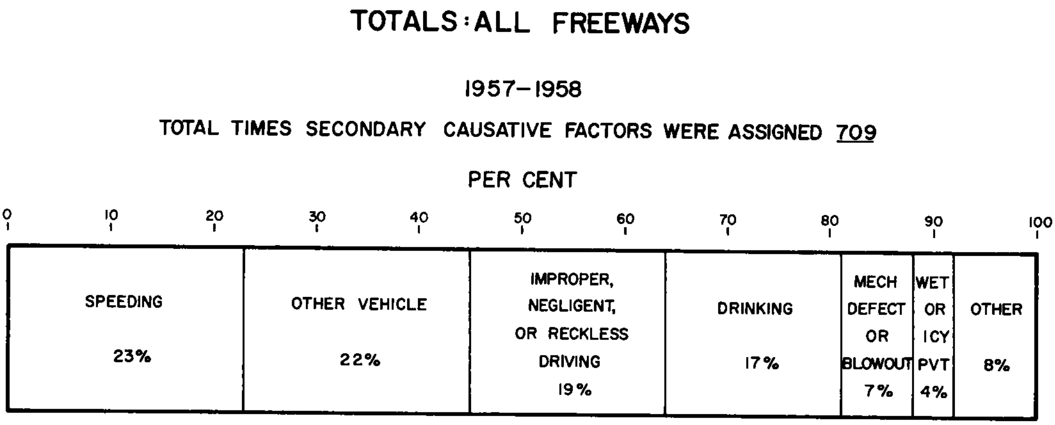


Figure 55. Secondary causative factors in fixed-object accidents.

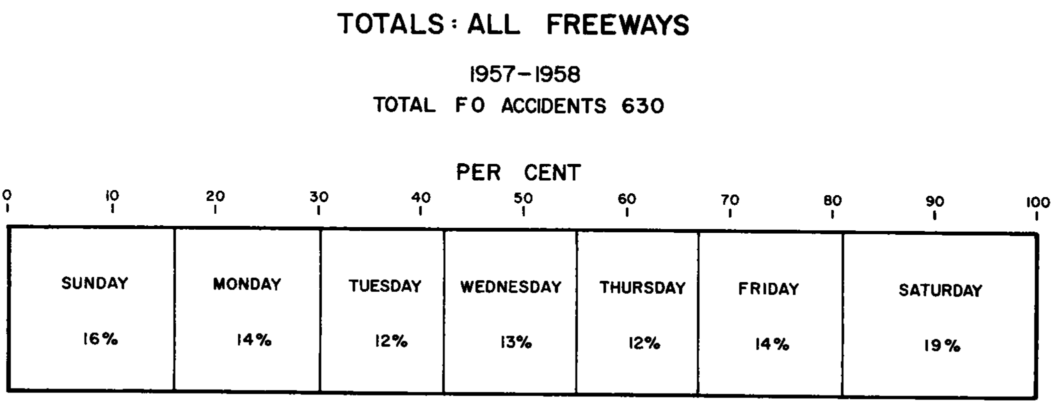


Figure 56. Fixed-object accidents by day of week.

SUMMARY AND RECOMMENDATIONS

1. Further study of accidents as related to design and operation of freeways is recommended.

2. There is a need for improving freeway traffic accident reporting in order to evaluate the relative safety of the geometric design of the various freeway elements. Typical freeway diagrams and established reference points proved in these studies to be useful in improving the accuracy of reporting accidents. Preparations for having adequate accident data must be made considerably in advance of analysis studies.

3. Sight distance or sight relationships are of great importance in the safety of freeway facilities. The vertical alignment must provide adequate visibility for the high-speed traffic traveling under high-density conditions on the freeway through lanes. Good visibility must be provided at points of ingress and egress. This visibility requirement applies to both through-lane traffic and ramp traffic. Standards of measurement of sight distance that are especially applicable to freeways are needed for best advantage in relating conditions and design features.

4. The designs of entrance and exit ramps should provide good sight relationships to encourage smooth flow at proper operating speeds without stopping, for the merging streams of on-ramp traffic with the through-lane traffic, and for the diverging streams of off-ramp vehicles.

5. Night visibility and driver condition are significant factors in fixed-object accidents.

6. Further study to determine the most satisfactory accident exposure factor to be used in calculating ramp accident rates is recommended.

These comprehensive studies of freeway accidents substantiate the evidence of previous studies that full control of access materially reduces accidents, injuries and fatalities. It is felt that the relief of the existing network provided by the freeway results in a reduction of accidents on that system and that the over-all safety afforded by the freeway should include the improved safety of the existing facilities. Further comprehensive accident studies are needed in order to compare accident experience on existing networks of streets and highways prior to the construction of a freeway with the accident experience on this system including the freeway.

ACKNOWLEDGMENTS

This research project was conducted by the Texas Transportation Institute under a grant from the Automotive Safety Foundation. Grateful acknowledgment is made to the Automotive Safety Foundation, especially to C. F. McCormack, Deputy Chief Engineer, for support and interest in this research.

The Texas Highway Department has given excellent cooperation in this research, supplying through the personnel of several division and district offices much of the needed data.

Gratitude is expressed to the Project Advisory Committee formed of key administrative and engineering personnel of the five largest Texas cities where the urban freeways are located which provided the location data, the Texas Department of Public Safety, the Texas Highway Department, the Bureau of Public Roads, and the Automotive Safety Foundation. Various other individuals and agencies also have aided with the study.

Appreciation is extended to Frank M. Smith, Jr., who, as a graduate student in Civil Engineering at the A. and M. College of Texas, made significant contributions in the special correlation study of selected on-ramp locations.

Gratitude is also expressed to the personnel of this project for their valuable work and sustained interest in the various phases of these studies.

REFERENCES

1. Keese, C.J., Pinnell, C., and McCasland, W.R., "A Study of Freeway Traffic Operation." HRB Bull. 235 (1960).
2. Pinnell, C., and Keese, C.J., "Freeway Ramps." Texas Transportation Institute (1960).
3. Mullins, B.F.K., "Improving Freeway Traffic Accident Reporting." Traffic Engineering, pp. 25-31, 54 (Dec. 1960).
4. Keese, C.J., and Pinnell, C., "Effect of Freeway Medians on Traffic Operation." HRB Bull. 235 (1960).
5. Breuning, S.M., and Bone, A.J., "Interchange Accident Exposure." HRB Bull. 240 (1960).
6. Smith, F.M., Jr., "An Analysis to Determine Correlations of Freeway Traffic Accidents with Specific Geometric Features." A thesis, A. and M. College of Texas (Sept. 1960).
7. Langsner, G., "Accident Experience at Ramps—Exits and Entrances—Preceding and Beyond Interchange Structures." Paper presented at the Regional Meeting, AASHO Committee on Design, Denver (Sept. 1959).
8. Blensly, R.C., and Head, J.A., "Statistical Determination of Effect of Paved Shoulder Width on Traffic Accident Frequency." HRB Bull. 240 (1960).
9. Hoch, I., "Chicago's Accident Experience on Arterials and Expressways." Traffic Quarterly, 14:3, 340-362 (July 1960).
10. Malo, A.F., and Mika, H.S., "Accident Analysis of an Urban Expressway System." HRB Bull. 240 (1960).
11. McElhiney, P.T., "Evaluating Freeway Performance in Los Angeles." Traffic Quarterly, 14:3, 296-312 (July 1960).

12. Versace, J., "Factor Analysis of Roadway and Accident Data." HRB Bull. 240 (1960).
13. Blindauer, H.H., and Michael, H.L., "An Analysis of High Accident Rates." Purdue University Engineering Reprints, CE 168 (Dec. 1959).
14. Conklin, R.D., "A Comparison of Vehicle Operating Characteristics Between Parallel Lane and Direct Taper Types of Freeway Off-Ramps." Traffic Engineering, pp. 13-17 (Dec. 1959).
15. Fisher, R.L., "Accident and Operating Experience at Ramps." New Jersey State Highway Dept., 56 pp. (1959).
16. "A Study of Highway Transportation in Indiana." Highway Safety in Indiana, Engineering Bull. 98, Vol. 43, No. 4, Chapter VI (Dec. 1959).
17. Houghton, E.S., "Accident and Operating Experience on Ramps and at Minor Roads." New Jersey State Highway Dept., 49 pp. (1959).
18. "Causes of Highway Accidents." Traffic Quarterly, Vol. 12, pp. 30-57 (Jan. 1958).
19. Ehrman, L., "Causes of Highway Accidents: United States Experience." Traffic Quarterly, Eno Foundation for Highway Traffic Control, Inc. (Jan. 1958).
20. Maier, E., "Planning Our Urban Expressway." Traffic Quaterly, Vol. 12, pp. 132-147 (Jan. 1958).
21. Smeed, R.J., "Road Accidents Statistics in Relation to Road Safety Activities." International Road Safety and Traffic, 6:4, 17-22, 37 (Autumn 1958).
22. Brittenham, T.G., et al., "A Method of Investigating Highway Traffic Accidents." HRB Bull. 161 (1957).
23. "Effect of Motorway Design on Accidents in Germany." Highways and Bridges and Engineering Works, 24:1161, 1, 3-4 (Oct. 1956), and HRA 27:4, 11-12 (April 1957).
24. Head, J.A., "Predicting Traffic Accidents from Roadway Elements on Urban Extensions of State Highways." HRB Bull. 208 (1959).
25. Mathewson, J.H., and Brenner, R., "Indexes of Motor Vehicle Accident Likelihood." HRB Bull. 161 (1957).
26. "Median Design—Effect on Traffic Behavior." HRB Bull. 137, 26 pp. (1956).
27. Owens, T.J., "Some Aspects of Freeway Design and Operation." Traffic Engineering, 27:7, 318-319 (April 1957).
28. Woo, J.C.H., "Correlation of Accident Rates and Roadway Factors." Purdue University, 64 pp. (July 1957).
29. "Extra Cost of Modern Expressways Justified by Accident Reduction." New England Records Builders Assoc., HRA 26:5, 22 (May 1956).
30. Head, Baker, and Baldwin, "Uses of Traffic Accident Records in Appraising Highway Needs and in Priority Determination." HRA, 26:11, 24 (Dec. 1956).
31. "A 5-Year Analysis of Traffic Accidents on the Missouri State Highway System." 134 pp. Supplements "2-Year Analysis" and "1-Year Analysis." Missouri State Highway Department Traffic Control Section (Oct. 1954, 1954-55, 1956).
32. Webb, and Israel, "Freeway Ramps: Proper Spacing Important in Preventing Accidents." California Highways and Public Works, 34:9-10, 14-16 (Sept.-Oct. 1955), and HRA, 26:1, 11-12 (Jan. 1956).
33. Blotzer, Krumm, Krus, and Stark, "Accident Causation—Pennsylvania Turnpike Joint Safety Research Group." Westinghouse Air Brake Company, 318 pp. (Sept. 1954).
34. Eakhardt, and Flanagan, "Role of Roadway Elements in Accidents on Pennsylvania Turnpike." HRA 24:11, 67 (Dec. 1954).
35. "Accidents Fewer But Deadlier on Superhighways." Traffic Digest, 6:11, 9-10 (Nov. 1952), and HRA, 23:1, 12 (Jan. 1953).
36. "California Highway Vital Statistics—1951." California Highway and Public Works, 31:11-12, 31 (Nov.-Dec. 1952), and HRA, 23:2, 23 (Feb. 1953).
37. Grossman, L., "Accident Exposure Index." HRA, 23:11, 11 (Dec. 1953).
38. Raff, M.S., "Interstate Highway Accident Study." HRB Bull. 74, pp. 18-43 (1953).
39. Staffeld, P.R., "Accidents Related to Access Points and Advertising Signs." Traffic Quarterly, Vol. 7, pp. 59-74 (Jan. 1953).

40. Bailey, G.W., "Road Safety: Responsibility of Planners, Architects and Engineers." The Contract Journal, 146:3805, 2096 (May 28, 1952) and HRA (July 1952).
41. Belmont, D.M., "The Effect of Speed and Volume on Motor Vehicle Accidents on Two-Lane Tangents." HRA, 22:11, 45 (Dec. 1952).
42. McMonagle, J.C., "The Effect of Roadside Features on Traffic Accidents." Traffic Quarterly, 6:228-243 (April 1952).
43. Telford, and Israel, "Median Study—1952." HRA, 22:11, 14-16 (Dec. 1952).
44. Halsey, M., "Traffic Accidents and Congestion." Wiley, New York, 408 pp. (1941).
45. Baldwin, D.M., "Accurate Facts for Highway Design." Public Safety, Vol. 31, pp. 10-11, 22-23 (Feb. 1947).
46. Smith, W.S., et al., "Uses of Traffic Accident Records." Eno Foundation for Highway Traffic Control, Inc., Saugatuck, Conn (1947).
47. Jorgensen, and Mitchell, "Accident Analysis for Program Planning." HRA, 19: 11, 40-41 (Dec. 1949).
48. Smeed, R.J., "Some Statistical Aspects of Road Safety Research." HRA, 19:9, 1-23 (Nov. 1949).
49. Noble, C., "Highway Design Research as Related to Safety of Vehicle Operator." HRB Proc., Vol. 17, Part I, pp. 242-253 (1937).
50. "Accident Reporting and Analysis as Related to Rural Highways." HRB Proc., Rural Traffic Problems (1939).

Development of An Automatic Traffic Flow Monitor and Control System

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●RISING TRAFFIC DEMANDS and roadway costs are confronting engineers and roadway operators with problems that seem likely to continue and increase. As one factor that might improve the productivity of existing roadways, traffic monitor and control equipment is receiving increasing interest. This paper describes development work on such equipment systems being undertaken by The Port of New York Authority.

The problems of rising traffic demands and roadway costs are especially clear for the Port Authority, which is responsible for building and operating bridges and tunnels connecting New Jersey and New York. The five existing two-lane tubes of the Holland and Lincoln Tunnels total less than seven miles of roadway, but they are among the most expensive roadways in the world. The present cost of constructing such tubes is about $18,000,000 per lane per mile. Including approaches, the addition of two under-river lanes at the Lincoln Tunnel in 1957 required an investment of nearly $95,000,000.

These costs highlight the importance of assuring the best possible operation of tunnel roadways. Today, tunnel traffic operation is basically the same as it was more than 30 years ago when the Holland Tunnel was first opened to traffic. But great strides have been made, particularly in recent years, in electro-mechanical and electronic equipment. To determine whether such equipment can be used to improve tunnel traffic operation, the Port Authority has been actively developing and testing new systems and equipment in the past few years.

There are two main areas in which it now seems possible to bring about significant improvements through use of new equipment. One area deals with clearing disabled vehicles from active roadways at less cost and more rapidly, thereby minimizing the congestion and delay which usually is precipitated by them. The second area is concerned with maintaining the higher traffic production and the lessened occurrence of disabled vehicles which are characteristics of fluid traffic flow. Prototype automatic equipment systems have been developed which will perform operating jobs in both of these areas, and full-scale field tests are under way now.

Both of these areas involve installing vehicle detecting and other equipment on or near the roadway. Also, both systems are aimed at eliminating or reducing the loss in roadway productivity which occurs when the traffic stream breaks down, either due to a disabled vehicle or to congestion. In view of these similarities, it is likely that the two systems will be integrated eventually into one comprehensive traffic flow monitor and control system.

However, the immediate aim of the studies is to evaluate and improve the functional and performance aspects of each system, and this work can be done best by treating the two systems separately. Accordingly, each system is discussed separately in this report.

Although these two systems are designed for tunnel operation, they might be adapted at least in part to benefit traffic operations on any congested roadway system. Tunnels do differ greatly in many important respects from freeways, but so far as traffic control is concerned the difference is more in degree than in substance. Because tunnels are so expensive to construct and operate, comprehensive traffic equipment systems are likely to find their first application in them, possibly followed by more widespread use on other congested roadways. This is true also because tunnels offer a more controllable test situation in which the effect of equipment systems can be measured.

TUNNEL POLICING

Disabled vehicles on active roadways are the most direct cause of congestion and lost capacity, and their consequences are especially harmful in tunnels. A vehicle which breaks down during rush hours causes traffic tie-ups which not only can be of major proportions, but also occur at unpredictable times. The importance of rapidly detecting and removing disabled vehicles in tunnels is underscored by the extent of the police coverage provided now to do these jobs.

During peak traffic flow there are five police officers in each of the five tubes stationed along the catwalks which run throughout the approximately 8,000 foot-long tubes. At other times, the number of police is reduced to four men in certain tubes. Late at night, tunnel roadways are patrolled by officers in vehicles. With allowances for reliefs, regular days off, etc., it is necessary to have nearly six officers on the payroll for each post manned full time.

However, events requiring policing action are fortunately not very frequent, and these officers spend the greater portion of their time observing the flow of traffic. A system which would save or reduce the cost of this observation time and direct it to more effective tunnel policing would be a definite step forward.

The most important single measure of the effectiveness of any roadway policing system is considered to be the amount of time required to handle any incident. The incident might be, for example, a fire or serious accident, a disabled vehicle, or just a slow moving driver. This time can be divided into three main components—the time needed to detect the incident, the time needed to respond, and the time needed to restore normal operations.

Reducing the man-hours that are now spent observing, can be accomplished most directly by reducing the number of men who are observing. However, this would result in a less effective policing system—that is, more time would be needed to detect and respond to incidents and restore normal operation—unless the policing effectiveness of each remaining officer can be magnified by new equipment.

Therefore, the aim of the system the Port Authority is developing is to magnify the ability of an individual officer to police roadways, by providing him with equipment to detect, respond, and restore normal operations more rapidly. His ability to detect incidents is now largely limited to the length of roadway in his direct view, and it is expected that this range will be extended radically by means of an automatic alarm system and closed circuit television. His ability to respond is now limited to a walking speed of 3 mph, and this speed will be multiplied by a factor of ten through unique catwalk transportation system. His ability to restore normal operations depends in large measure on his effectiveness in controlling tunnel traffic so as to expedite the tow truck or emergency tractor on its way to the disabled vehicle. His control is limited now to traffic in his immediate area, and it is planned to extend his effectiveness by providing several sets of remotely operated signs, signals, barriers, loudspeakers, and other devices which he can control from a central point equipped with television for surveillance of the remote traffic control areas. To summarize this projected system, the present and possible new systems are compared directly in Figures 1 and 2.

Figure 2 shows a possible system of policing tunnels with only one man rather than the five men used in the present system. However, this minimum manpower has been assumed only for study purposes, and is not at this time a serious proposal. The purpose of this study is to develop equipment which will increase the policing effectiveness of individual officers to a maximum, and therefore, it is desirable in the study to assume that only a minimum of manpower is available. But the number of men that would actually be used by the Port Authority to police tunnels under a system such as this will depend on the judgment of operating management, which will be based in part on the actual performance of the equipment described in the following paragraphs.

Extending the Officers' Ability to Detect

In view of the direct applicability of closed circuit television for this function, the question may airse as to why any developmental work is considered necessary. There are two general reasons why television alone is not considered likely to provide the best solution.

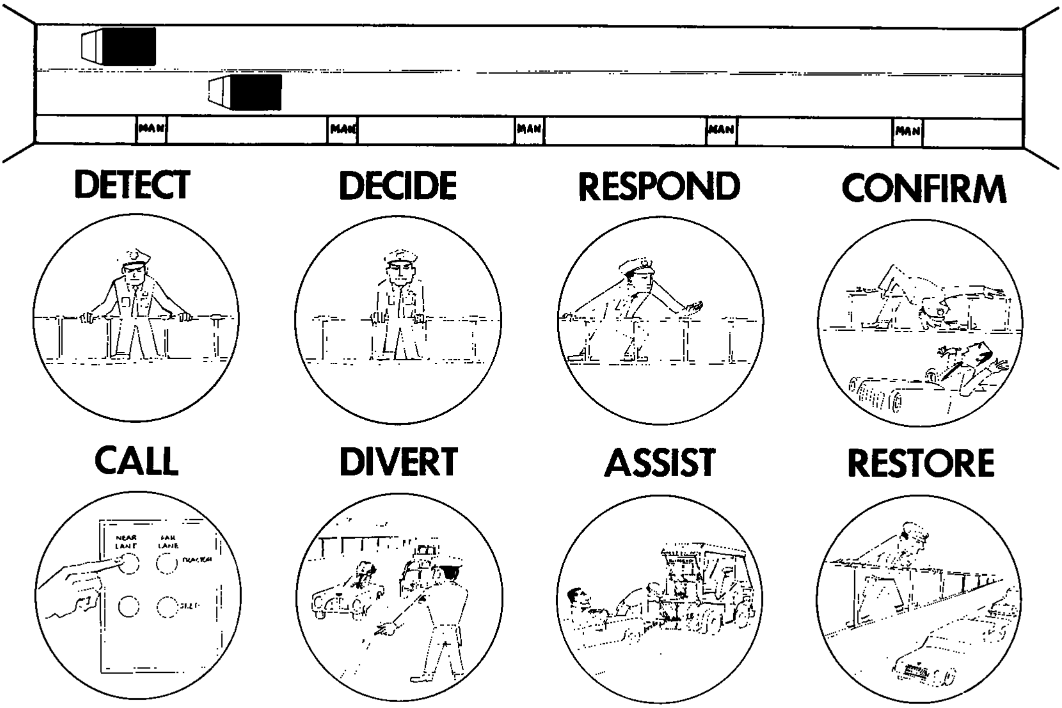


Figure 1. Present system.

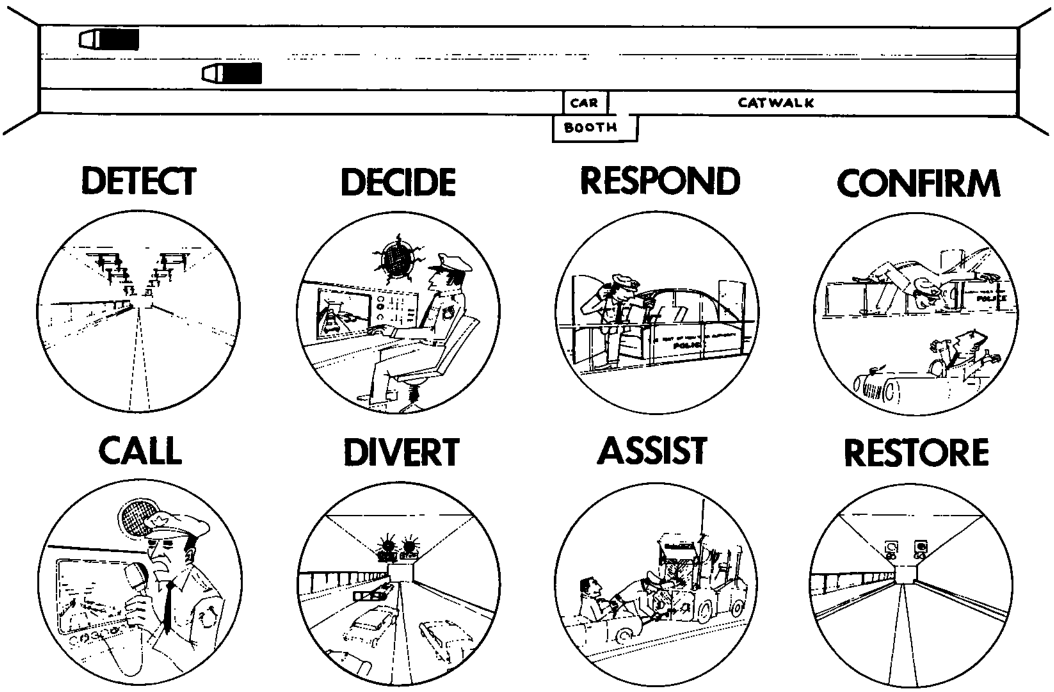


Figure 2. Possible system.

The first reason is the tunnel environment which, because of low light levels, handicaps the performance of television. The 1 to 3 foot-candles of light present inside the tunnel is the minimum level at which television can operate. Nevertheless, through extended testing, it has been found that the performance of closed circuit television is remarkably good. Although the low light level does result in a marginal picture, police observing the picture screen have been able to determine the general condition of traffic movement up to distances of 500 ft from the camera. It is not possible, however, to gain detailed information on any vehicles which might become disabled within the camera's view.

A more serious environmental limitation on television in tunnels arises from the impossibility of placing the camera far enough away from the traffic stream to cover a significant length of roadway. However, one relatively minor modification which appears promising, is to place a mirror at the focal length of the lens so that the television monitor displays on one-half the screen the view seen directly by the camera, and on the other one-half of the screen the view reflected from the opposite direction by the mirror. By a system such as this it is expected that a camera may be able to provide traffic information through a section of roadway up to 1,000 ft in length.

These environmental limits on television performance would not apply on freeways, and it seems likely that television will be more effective in those applications. However, the second general reason the Port Authority has, at this early stage, tentatively decided against using television as the front line component does apply on freeways. Inasmuch as a man can effectively monitor only one picture at one time, television is basically a means of transporting a man's vision rather than duplicating it. Thus, a detection system which depended on many television cameras monitored by one man, would still be relying basically on the attentiveness of one man rather than at present, on five men. And the problem of assuring attentiveness over a period of hours seems likely to be major.

Because of these limitations on television, the Port Authority is developing a system which will automatically generate a signal when and where traffic flow is not normal. The proposed automatic alarm system uses vehicle or axle detectors located along each tunnel lane (Fig. 3). Each detector is connected to a "stoppage computer" which

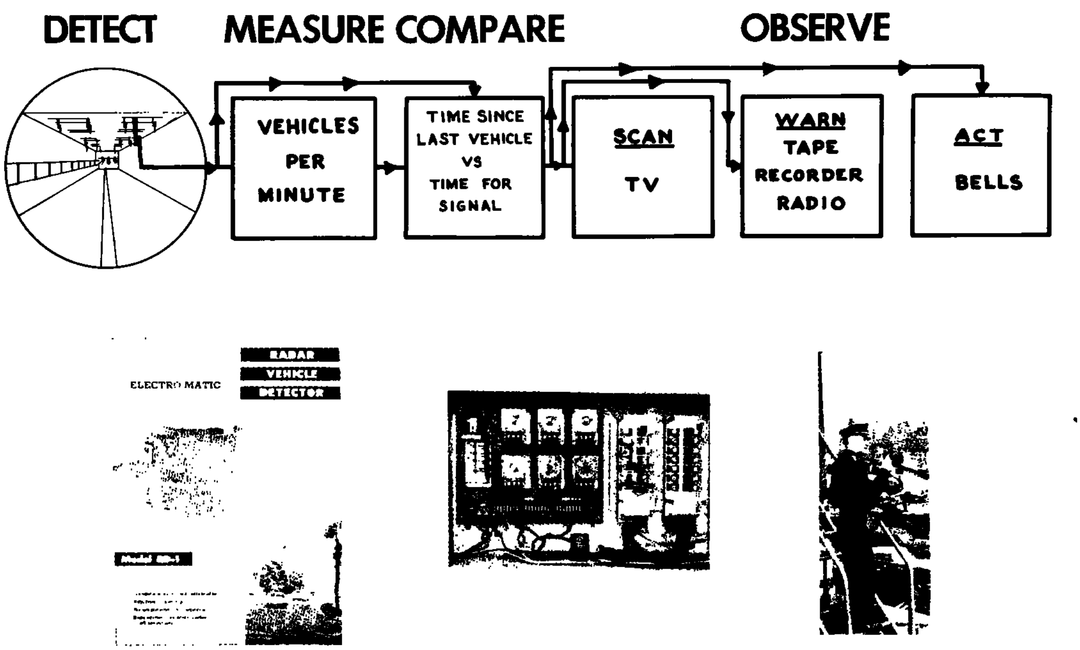


Figure 3. Monitor system.

measures the rate of traffic flow past the detector and determines when, for that level of flow, an excessive amount of time has elapsed without a vehicle passing the detector. When that occurs, the stoppage computer starts a chain of alarms which continue either until a vehicle does pass the detector, or a police officer acknowledges the alarm and resets the system.

Vehicle detectors are available commercially in increasing variety, and there is little question of the feasibility of accomplishing this part of the operation. Detectors are likely to be the front line component in any automatic traffic system. Because of their importance, it might be helpful to review the Port Authority's experience with detectors.

For the past two years the Port Authority has been testing radar vehicle detectors, ultra-sonic vehicle detectors, and induction loop detectors. The first two are especially suited for tunnel use because they can be installed in the ceiling with a minimum of difficulty. The induction loop is also simple to install by cutting a slot in the form of a loop in the roadway. Each of these three detectors is generally comparable in price. In the Authority's experience the ultra-sonic detector has been the most accurate of the three although the induction loop now appears to have been developed to a point of comparable accuracy. Circuitry required for the induction loop appears to be the most simple of the three.

Another vehicle detector which is currently being tested establishes a beam of ultra-sonic energy. This detector cost only about one-third the amount of the three commercial units described earlier, but is more difficult to install because two separate units, a transmitter and a receiver, are required. Because of the geometrics of the tunnel design, one unit has been installed under the roadway in the fresh air supply duct, and the other unit on the tunnel ceiling. These units have not been tested long enough at this time to state their accuracy and maintenance requirements. Another commercial product, which is by far the least expensive detector being tested, is a tape treadle. Although it is hoped that this unit will function satisfactorily, experience to date has been too limited to permit any conclusions.

In addition to these detectors, the Port Authority is testing two other units. Photocells offer a relatively inexpensive device, although maintenance costs will doubtless be higher than with some of the other detectors mentioned. Because of the fresh air

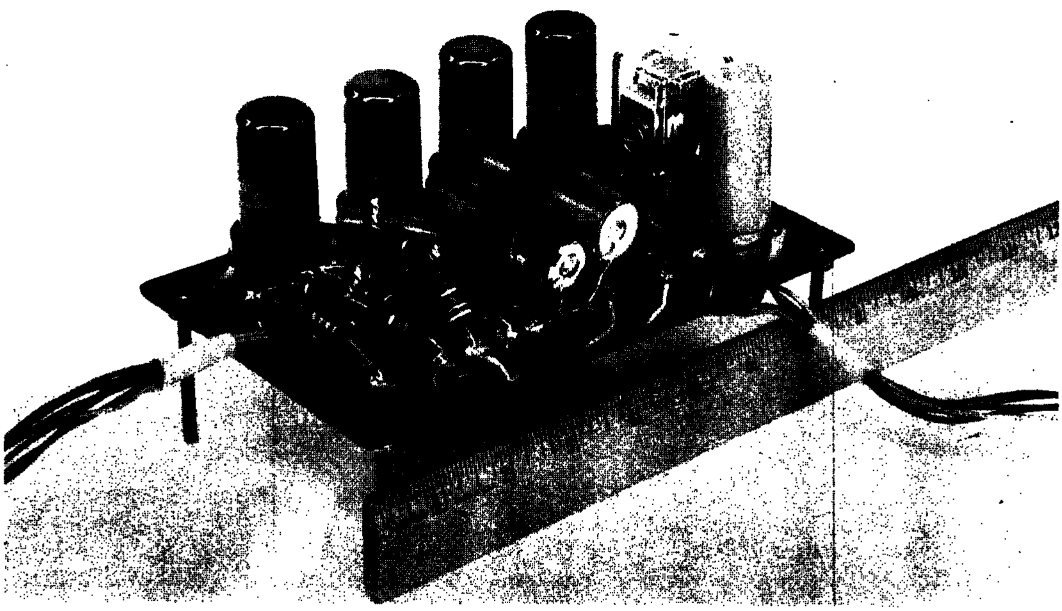


Figure 4.

which must continually be supplied to a tunnel, it is possible to mount the photocell light source in such a way that it will remain cleaner than would be expected in the usual roadside environment.

After studying several types of treadles, the Port Authority staff has devised a carbon pile unit which consists of a steel plate resting on piles of carbon disks. This unit may offer long life and low maintenance. Another prototype commercial vehicle detector being tested also uses a metal plate in the roadway and a strain gage. Maintenance cost of this unit is also expected to be low. However, both of these treadles are still untested and their performance is not known yet.

The stoppage computer is the heart of the automatic alarm system. These units have been developed entirely by the Port Authority staff, as a canvass of possible manufacturers failed to find commercial unit or system which could be easily adapted at a reasonable cost for this purpose. The computer appears to offer a relatively simple and low-cost method of automatically monitoring traffic flow for the occurrence of disabled vehicles. An electro-mechanical version is shown in Figure 3, and a much simpler prototype using vacuum tubes is shown in Figure 4. This unit might be transistorized and made even more compact.

In operation, the computer does two jobs simultaneously. First, by remembering the number of pulses it has received in the past few minutes, the computer establishes a flow rate. Secondly, the computer measures the time that has elapsed since the preceding vehicle has passed the detector with which it is associated. When flow is heavy past the detector a large number of pulses will be received in the few minutes. Then the computer will recognize that the passage of a relatively small amount of time (about 30 sec) without a vehicle passing the detector might be cause for an alarm. On the other hand, when traffic flow is light and few pulses are received, the computer would not generate an alarm until a much longer time (about 2 min) passes with no vehicle passing the detector. One significant advantage of this system is that it fails safe. That is, unless the system continues to operate and vehicles pass the detector, an alarm is generated.

The alarm circuitry has various outputs to provide for alarms of increasing urgency as time passes without a vehicle being detected. This gradation in alarm severity is desirable to match the increasing certainty that the lack of flow is due to a disabled vehicle. If the amount of time the computer waits after the passage of each vehicle before generating an alarm is relatively small in relation to the average headway time between vehicles, a relatively large proportion of false alarms will be generated. On the other hand, if the computer were to wait in all cases until there was a strong probability that the failure of a vehicle to pass its detector indicated a disabled vehicle, then there would be an excessive delay in detecting that stoppage. It is planned to compromise this dilemma by generating alarms of limited scope when the ratio of false alarms is high, but increasing the severity of alarms as time continues to pass without a vehicle detection. The earliest alarm might consist of automatically turning a television camera on to view that section of the roadway in which the alarm is generated. If an officer is observing the television and there is a stoppage, it would be detected very quickly. If the officer is engaged in some other duty and not observing television, then the next step in the alarm process might be to broadcast a prerecorded voice signal. If still no action is taken to investigate the cause of the alarm, bells might be sounded. At any point in this alarm process, when it has been conclusively determined that there has been a stoppage and that response is on the way, or that the alarm is false, the train of alarms can be interrupted.

As a complement to the automatic alarm system, it is planned to test closed circuit television for several purposes in the new system of policing tunnels, including:

1. To verify alarms from the automatic alarm system and indicate the probable types of emergency equipment that will be needed.
2. To assist a police officer outside the tunnel in manipulating the signs, signals and other special devices that will be provided as part of the remotely operated traffic control system.
3. To allow this same police officer to observe vehicles throughout the tunnel during periods when there is a disabled vehicle and normal flow has been suspended.

4. To periodically inspect the condition of roadways to detect large foreign objects which might have been dropped by traffic.
5. To detect violations of traffic regulations.

Several alternate plans for using television are being considered. Decisions as to the disposition of cameras, picture screens and the flexibility of interconnections between these components, will depend in part on the usefulness and performance of television, and in part on the job done by other components of the proposed system.

As an adjunct to these alarm and visual communication systems, voice communication systems are also being studied. The usefulness of radio in traffic police work has been well established, but there is no system presently in use which meets all the requirements for a tunnel application. It is desirable that the officers on tunnel duty be equipped with pocket-sized radio transmitters and receivers which would interfere as little as possible with their freedom of action. A special antenna system is needed and, although the necessary antenna systems and radios are commercially available, they have not been used in environments with the high ambient noise present inside tunnels.

The remote components of this system to extend the officer's range of detection are located in a booth adjacent to the catwalk in which the officer will normally be stationed. When an alarm is received and verified, the officer next needs a rapid means of reaching the scene. Other monitor points will also be equipped outside the tunnel for backup.

Extending the Officers' Ability to Respond

In the two-lane tunnels, as in any roadway where there are no shoulders or other spaces that can be used in an emergency, the problem of supplying police rapidly to a scene requires a specialized solution. In the tunnel the most feasible way of accomplishing this is to use a vehicle on the narrow catwalk along which police officers are presently stationed. Because a man requires 18 in., the 22-in. wide catwalk allows only 2 in. of space on each side of the man to accommodate the vehicle in which he is traveling and the inevitable sway produced as the vehicle moves along the catwalk.

To provide for rapid response to any point in the tunnel, a vehicle was desired that an officer would elect to drive at speeds up to 30 mph along the catwalk. To provide this system, the Port Authority retained Battelle Memorial Institute of Columbus, Ohio.

The vehicle they have conceived is unique (Figs. 5 and 6). There are only three points of contact and only one wheel in the conventional sense. The rubber-tired pneumatic wheel, located near the center of the vehicle, supports most of the vehicle weight and provides the propelling and decelerating force. At each end of the vehicle on the side next to the tunnel wall there is mounted a three-wheel trolley which is firmly clamped to run along a special Z section guide rail. Lateral placement of the vehicle on the catwalk is controlled by a V section welded to the underside of the Z rail. A grooved wheel rides on this rail (Fig. 7). In addition, these wheels are affixed to a member of the trolley assembly which is shaped to slide along the rail in the event of any failure of the trolley wheels. Motive power is supplied by a 7.95 HP gasoline engine, acting through a planetary transmission similar to those used on the Model T Fords. There are duplicate controls at each end of the vehicle which permit travel equally well in either direction.

The 20-in. wide, single-wheel vehicle has fully demonstrated its ability to operate at speeds in excess of 30 mph along a special guide rail which has been attached to the catwalk. Of critical importance is the fact that the vehicle feels both stable and safe when it is operated on the 22-in. wide catwalk at these high speeds.

Extending the Officers' Ability to Restore Traffic

Restoring traffic for normal operation requires clearing one of the two lanes between the disabled vehicle and the tunnel exit so that emergency equipment, which enters the tunnel from the exit portal and proceeds against traffic, can reach the disabled vehicle. Under the present system of tunnel policing, officers on the catwalk between the disabled vehicle and the tunnel exit, stop traffic in both lanes and then move all traffic in the lane nearest to the catwalk over to the other lane. In a system which makes extensive use of equipment it will be necessary to provide remotely operated traffic control devices to accomplish this function.

A possible system to accomplish this would use signs, signals, barrier gates, and other devices operated by an officer possibly located in a traffic control center outside the tunnel (Fig. 8). That officer would be observing the effect of his actions over

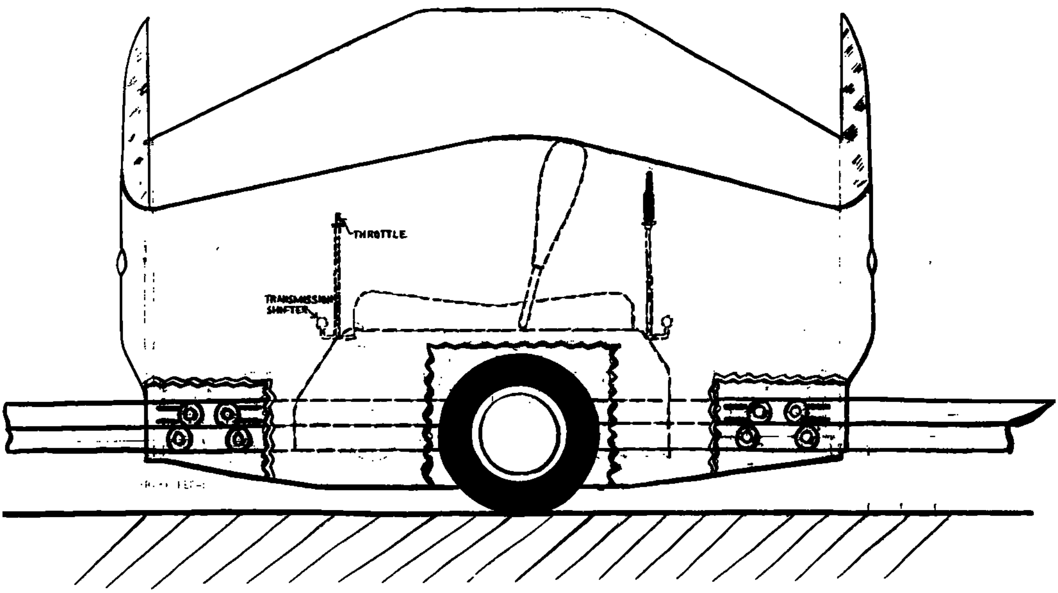


Figure 5. Catwalk vehicle diagram.

Figure 6.

television, and might also have a voice link to motorists in the affected area via loudspeaker.

One of the problems in such a system is to provide a legible sign with good impact, but not reduce ceiling clearance more than 3 in. To accomplish this, the Port Authority designed a sign 8 ft x 10 ft x 3 in. for mounting on the ceiling, using the elongated letters which are standard for pavement marking. This sign is shown in Figure 9 at a distance of 100 ft. It has been found to provide ample legibility and impact, but is not sufficient by itself to assure that traffic will stop. Other devices, including retractable lane delineators, will be used.

When assembled as a system, these three types of equipment—detection, transport and control—can provide considerable flexibility and backup. Monitoring will be performed at several points, and spare catwalk vehicles will be stationed at tunnel portals in the system presently conceived to increase reliability and effectiveness.

FLOW CONTROL

Although it is clear that more expeditious handling of disabled vehicles and other interruptions in normal traffic operation will improve traffic service, it is not immediately clear that controlling the character of traffic flow can also significantly improve traffic operations.

Through study of traffic flow characteristics in the near lane of the Holland Tunnel South Tube conducted in 1959, a pattern of shock waves was found to exist during peak traffic flow. The shock waves are periodic reductions in the flow and speed of traffic.

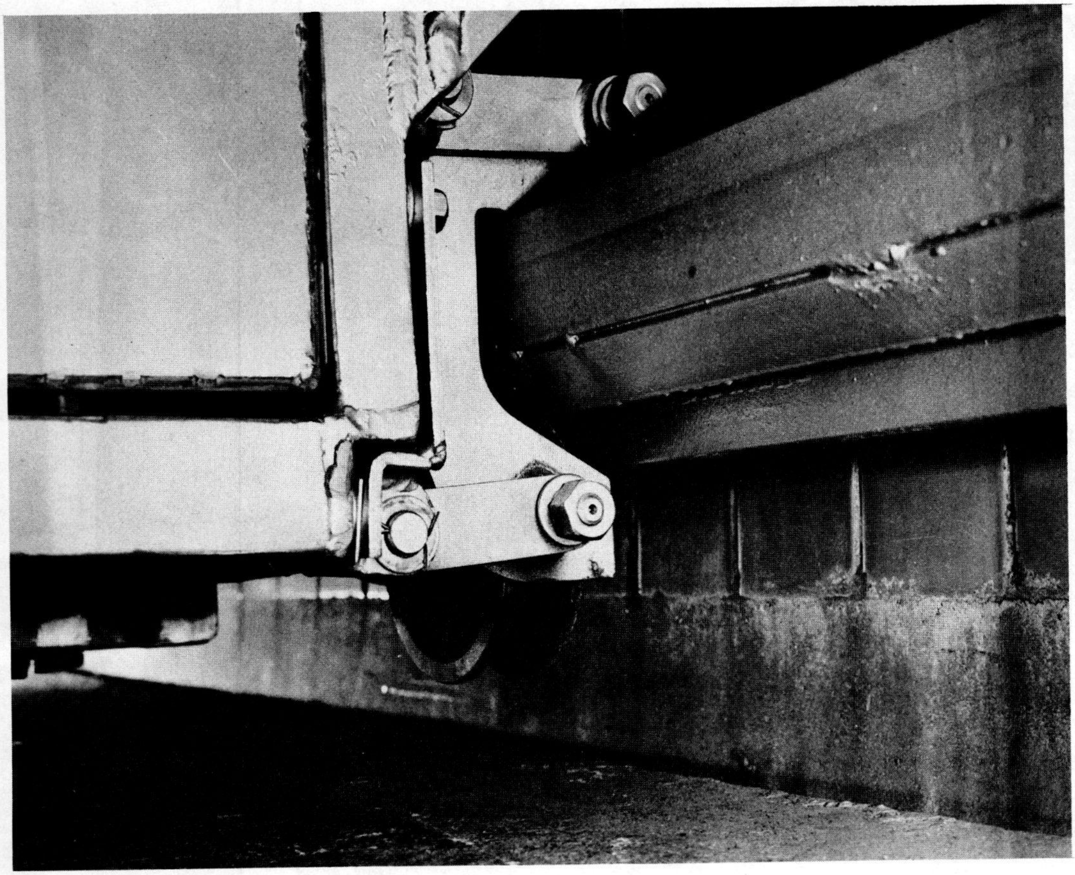


Figure 7.

They are caused by the inability of that section in the tunnel which has the least capacity (that is, the bottleneck), to handle all of the traffic being supplied to that section. Shock waves start at the bottleneck and move back through the traffic stream to the tunnel entrance. On the other end, between the bottleneck and the tunnel exit, gaps appear in the traffic stream because the bottleneck cannot furnish enough traffic to saturate that section.

Because of this behavior of shock waves and gaps, it is possible to locate the bottleneck by observing the progression of shock waves and gaps in the traffic stream flowing through the tunnel lane. Measurements made in the Holland Tunnel South Tube near lane showed that the bottleneck is located near the foot of the upgrade.

Further study showed that several improvements in the traffic flow through that lane could be achieved if shock waves were prevented. This could be accomplished by limiting traffic entering the tunnel to the maximum amount which could be handled at the bottleneck. Experiments have confirmed this expectation.

To understand how these improvements are gained it is necessary to understand two processes which occur in vehicular traffic flow. One process is the decrease in speeds of successive vehicles in platoons. This decrease is caused by the fact that, because the vehicles are in platoons (and hence driving relatively close to each other), the fluctuation of speeds from one vehicle to the next will generally be such as to result in slower speeds. Fluctuations which would be on the side of having the successive vehicles speed up are not likely because accidents would follow due to vehicles becoming too closely spaced.

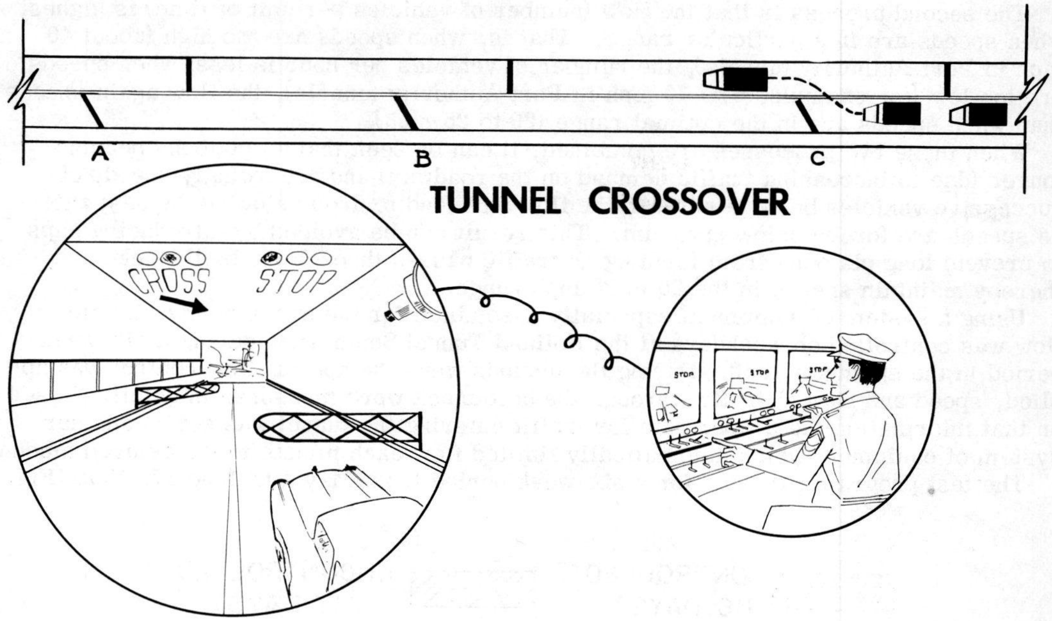


Figure 8. Diversion system.

Figure 9.

The second process is that the flow (number of vehicles per unit of time) is highest when speeds are in a particular range. That is, when speeds are too high (about 40 mph in Port Authority tunnels), the number of vehicles per hour is less; when speeds are too low (for example, 5 to 10 mph in Port Authority tunnels), the flow again is less than when speeds are in the optimal range (20 to 25 mph).

When these two processes are combined, it can be seen that as platoons become longer (due to increasing traffic demand on the roadway) and accordingly speeds of successive vehicles become slower, the flow will tend to drop below its highest rate as speeds are forced below optimum. This result can be avoided by introducing gaps to prevent long platoons from forming as traffic passes through the bottleneck, and thereby maintain speeds in the 20 to 25 mph range.

Using a system of equipment especially assembled for the experiments, traffic flow was controlled on weekdays at the Holland Tunnel South Tube during a six-week period in the spring of 1960. During the periods when the special flow control was applied, speed and flow of traffic through the bottleneck were measured in detail. Based on that information the proper rate for traffic entering the tunnel was set on another system of equipment which automatically limited flow each minute to the desired amount.

The test program extended for a six-week period from May 4 to June 17, 1960 (Fig.

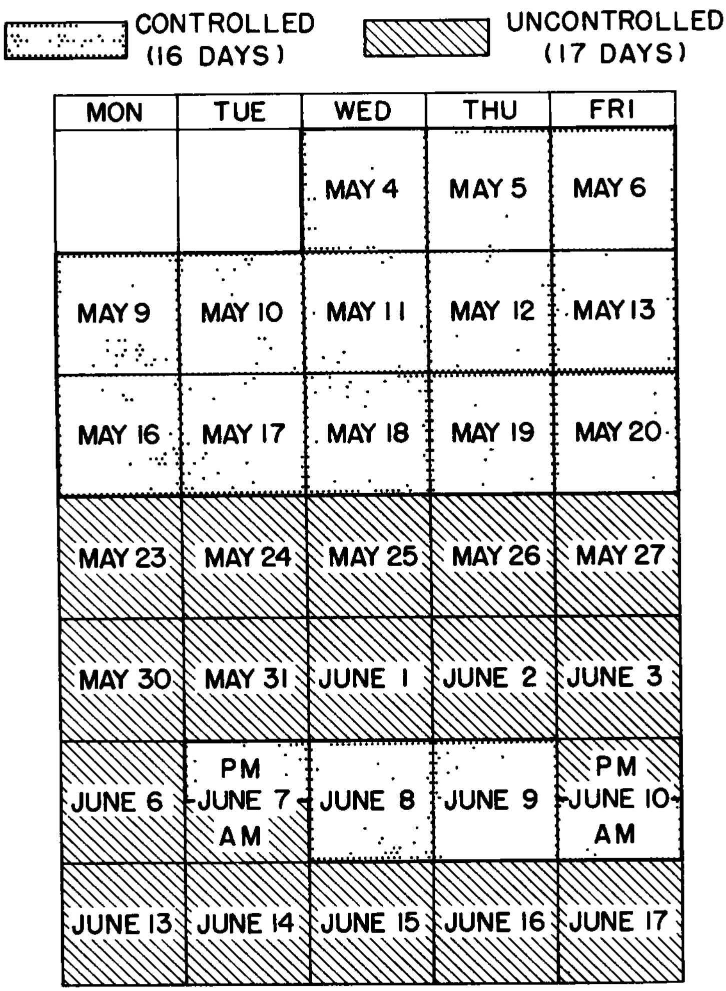


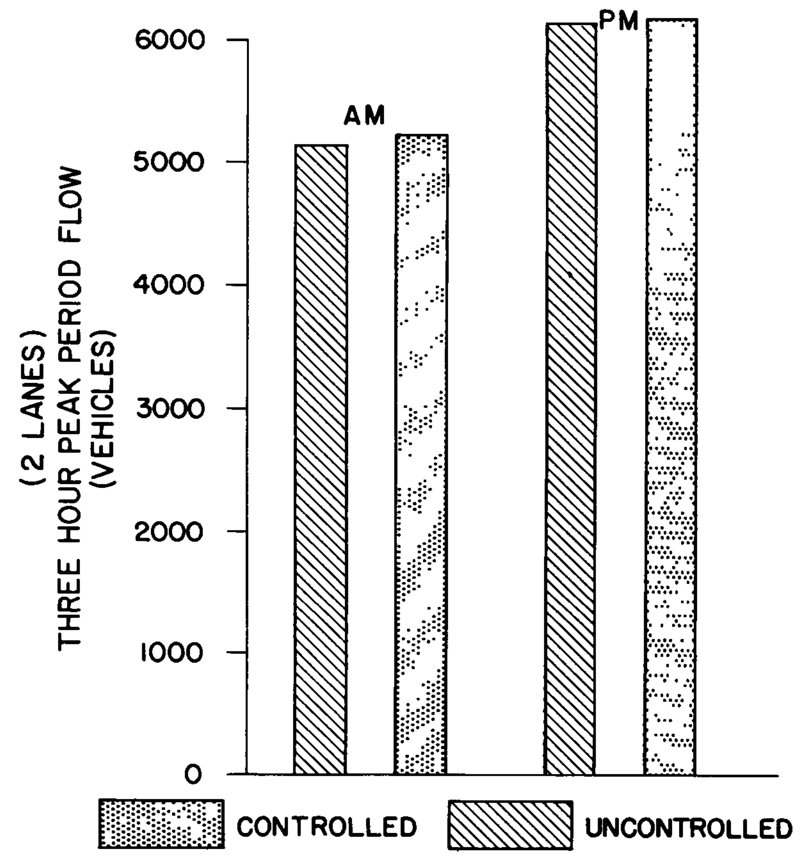
Figure 10. Calendar of controlled vs uncontrolled survey days.

10). During the first two weeks, from May 4 to May 20, the control procedure was followed. From May 23 to June 6, the traffic flowed without control but with the major indices of tunnel operation being measured. June 7, 8, 9 and the morning of June 10 were periods when the traffic was controlled. From June 10 until June 17 uncontrolled traffic production was measured.

The average 3-hr peak period traffic demand comparison for the controlled and uncontrolled days during either the morning (7-10 a.m.) or evening (4-7 p.m.) rush, indicates that there is no significant difference in the demand, and hence no difference in the volume handled (Fig. 11); nor was there any significant difference in the weather conditions during the two periods. In other words, the tunnel had to do essentially the same job under essentially the same conditions. The basic question is: how was the traffic served when the control procedure was used, in comparison with when no control was applied?

Test Results

A detailed analysis of tunnel production in 15-min intervals clearly shows a consistent difference in production (Figs. 12 and 13). During the evening peak period the



	AVG TEMP		σ TEMP		AVG HUM		σ HUM	
	AM	PM	AM	PM	AM	PM	AM	PM
CONT	59	66	4.6	18.4	64	63	5.8	16.2
UNCONT	63	70	4.8	11.0	75	62	6.5	19.4

Figure 11. Average traffic demand and weather conditions during survey.

average demand was less than tube capacity until 4:15 p.m., and there was no difference in the controlled versus uncontrolled flow. From 4:15 and 6:15 p.m. however, the control procedure maintained a consistently higher average production level than did the uncontrolled operation. During the critical hour the increase was 5 percent in the total tunnel flow. From 6:15 to 6:45 p.m. the uncontrolled 15-min volumes were higher than the controlled volumes. The apparent reason for this reversal at 6:15 p.m. in the production pattern is that the demand under the control procedure was satisfied earlier.

A similar production analysis of the morning peak period also showed an increase in production of 5 percent in the critical hour from 7:45 to 8:45 a.m. However, in the 15-min periods before and after this hour, there was no significant difference in controlled and uncontrolled flow. It is believed this record reflects the high proportion of trucks present in both lanes, except in the 7:45 to 8:45 a.m. period when passenger cars are present. The flow control procedure did not appear to benefit truck traffic, because these vehicles generally would not take advantage of gaps to maintain higher speed.

Whereas the significance of this production increase lies more in its consistency than in its magnitude, the relatively modest 5 percent increase in the production during

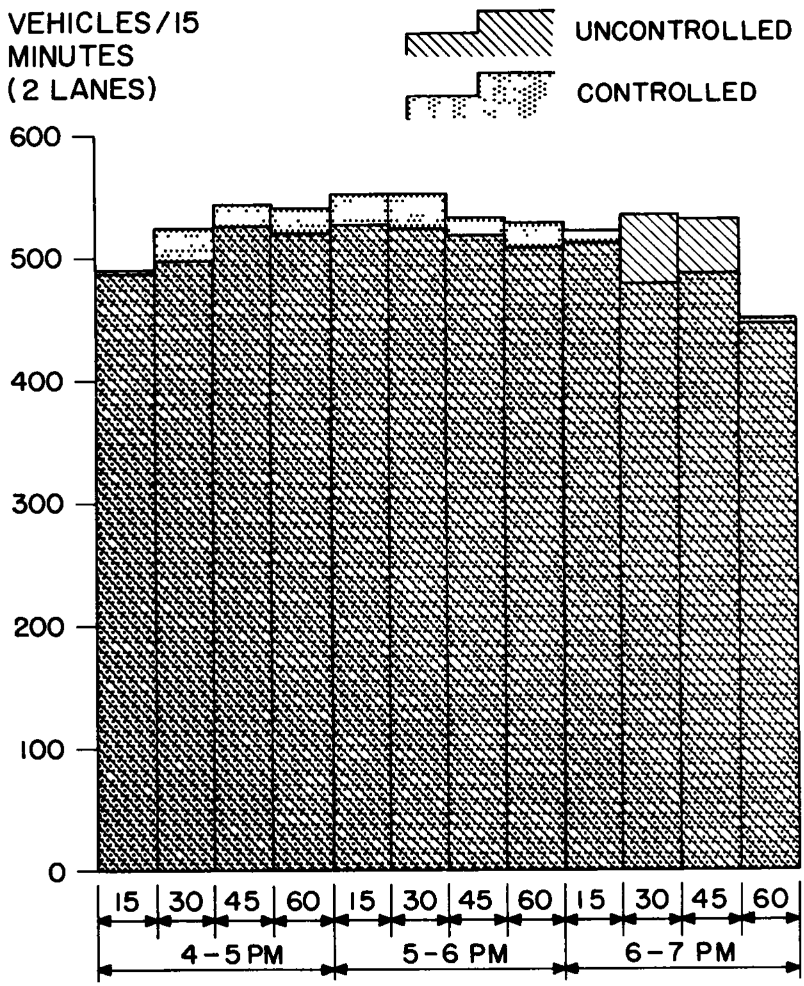


Figure 12. Average volume for 15-min intervals during PM peak period.

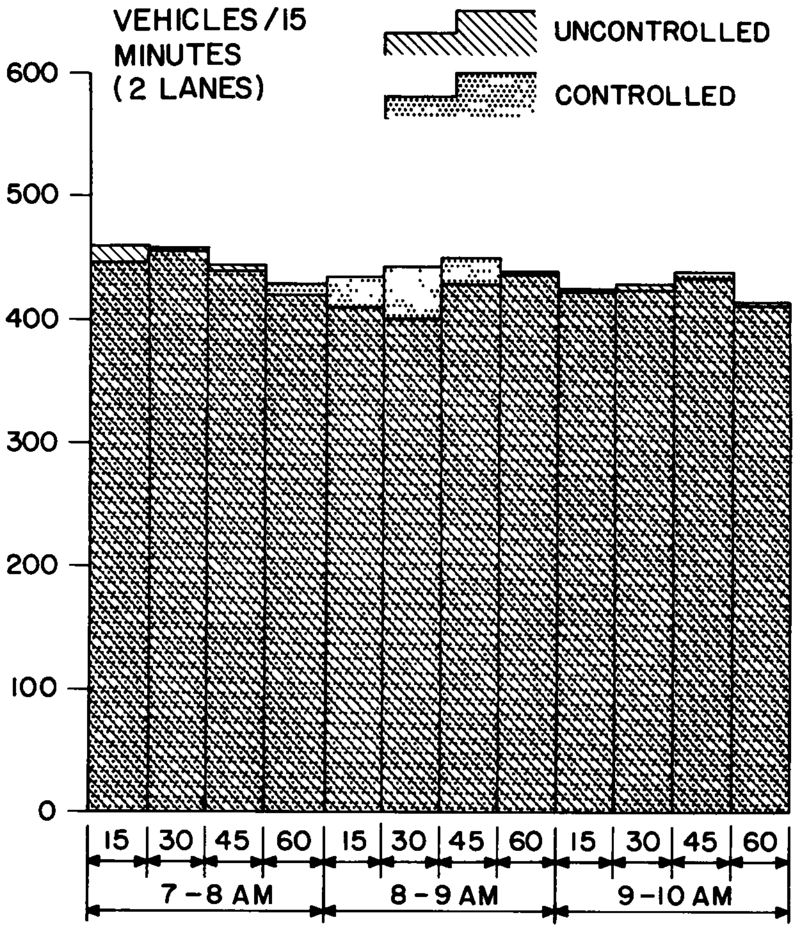


Figure 13. Average volume for 15-min intervals during AM peak period.

the critical hours from 4:15 to 5:15 p.m. produced a very marked reduction of 33 percent in the duration of congestion on the approaches in the evening peak period (Fig. 14). The average congestion during the "uncontrolled" evening period was 3 hours 2 minutes, whereas the "controlled" period had a congestion duration of only 2 hours 2 minutes. Analysis of congestion in the morning period showed no significant reduction in the length of congestion during the controlled period, and this probably can be attributed to the fact that morning congestion is made up largely of truck traffic that was not amenable to improvement by flow control methods.

The improvement in tunnel production recorded in these tests when the input control was used can be attributed to two general factors: a significant reduction in the occurrence of disabled vehicles, and an improvement in the speed-headway relationships maintained by traffic passing through the bottleneck.

In a 3-hr peak period two disabled vehicles will occur on the average in each tube. By maintaining free-moving traffic flow through the tube, the flow control procedure was expected to reduce the occurrence of vehicle failures caused by overheating, motor trouble, stalling, vapor lock, and other failures increased by frequent stopping and slow speeds. The test experience confirmed this expectation.

Analysis of the disabled vehicles in the controlled versus uncontrolled periods showed a total of 60 vehicular breakdowns during the uncontrolled a.m. and p.m. peak periods, compared with 44 disabled vehicles in the controlled periods (Fig. 15). The reduction in stoppages occurred in the classification of motor trouble (which covers stalled vehicles, vapor locked vehicles, and overheated vehicles), and was from 43 during uncontrolled flow to 16 in the controlled period. Other classifications (such as out of gas) were not reduced by the flow control.

To determine whether an improvement in the speed and headway relationships of traffic passing through the bottleneck had occurred, a random sample of near lane speeds and volumes at the foot of the upgrade was plotted for both the controlled and uncontrolled periods (Fig. 16). For each speed, the mean volume was computed. The flow versus speed curve for the controlled flow depicts a clear relationship, with maximum flow of approximately 1,290 vehicles per hour occurring in the optimal speed range between 20 and 25 mph. For the uncontrolled flow, no single flow-speed relationship is evident. However, the points do suggest two flow-speed curves, with one extending through the optimal speed range and having a maximum flow of 1,235 vehicles per hour.

Another result of these tests that is particularly important for tunnels, and may have some importance in urban areas generally, was a significant reduction in tunnel

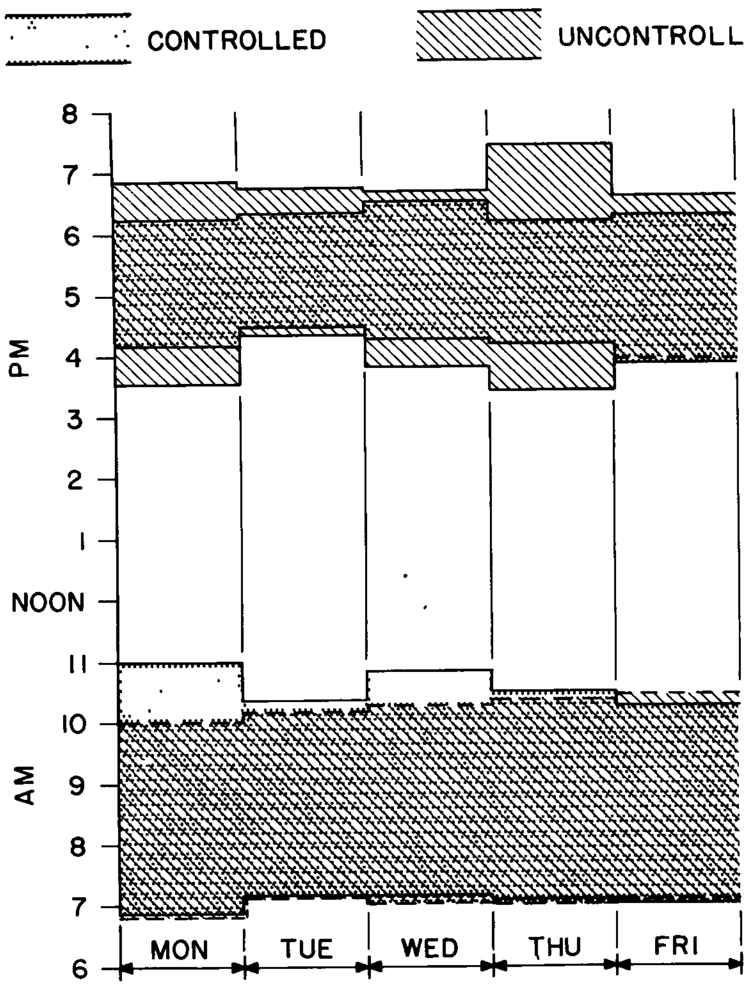


Figure 14. Congestion on approaches.

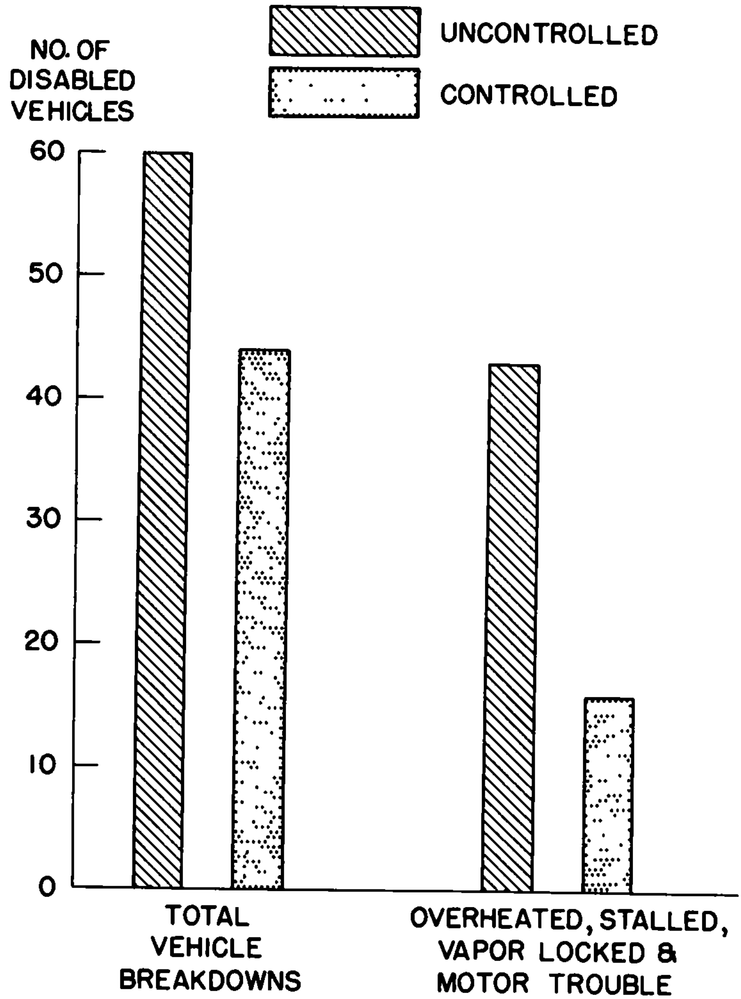


Figure 15. Vehicle breakdown comparison.

ventilation required during controlled flow. Even though the power consumed in ventilating the tube was decreased by 28 percent, the air in the tunnel at the same time was 20 percent cleaner (Fig. 17).

Equipment

The system of flow control equipment tested in these experiments had two general components. First, devices to limit traffic entering the tunnel were designed so that entering flows could be set as low as 15 vehicles per minute through a range in 1 vehicle per minute increments to a maximum of 24 vehicles per minute. The proper setting for this first component was to be determined continually in peak periods by a man observing the speeds and flows of vehicles approaching and through the bottleneck. Equipment to provide the speed information formed the second component. The equipment used in this system is shown in Figure 18.

The first main component of the system—devices to automatically limit the number of vehicles entering the tunnel to any desired level—was installed at the entrance to the South Tube, and consisted of:

1. Overhead signals, flashing stop signs and a bell, located where traffic starts on the downgrade to enter the tunnel (Fig. 19).

2. These entrance signals were actuated by a traffic spacing computer located in the tolls sergeant's building (Fig. 20). This device consisted essentially of a timer and a stepping switch. The computer turned on the entrance signals whenever more vehicles than the preset amount entered the near lane in less than the proper time.

3. In this system the number of vehicles entering the near lane of the tunnel is provided to the computer by vehicle detectors located at the point where traffic has been merged into two lanes and starts its descent to the tunnel entrance portal.

4. Controls for the system were all installed at the New Jersey tolls sergeant's desk at a point where the entire entrance plaza was in view.

The second component of this system—devices to measure the speed and flow of traffic at critical points in the tunnel—consisted primarily of equipment loaned to the Port Authority for this test by the Automatic Signal Division of Eastern Industries, Inc., and was located inside this tunnel as follows:

5. A radar speed sensor was mounted over the near lane at the foot of the upgrade.

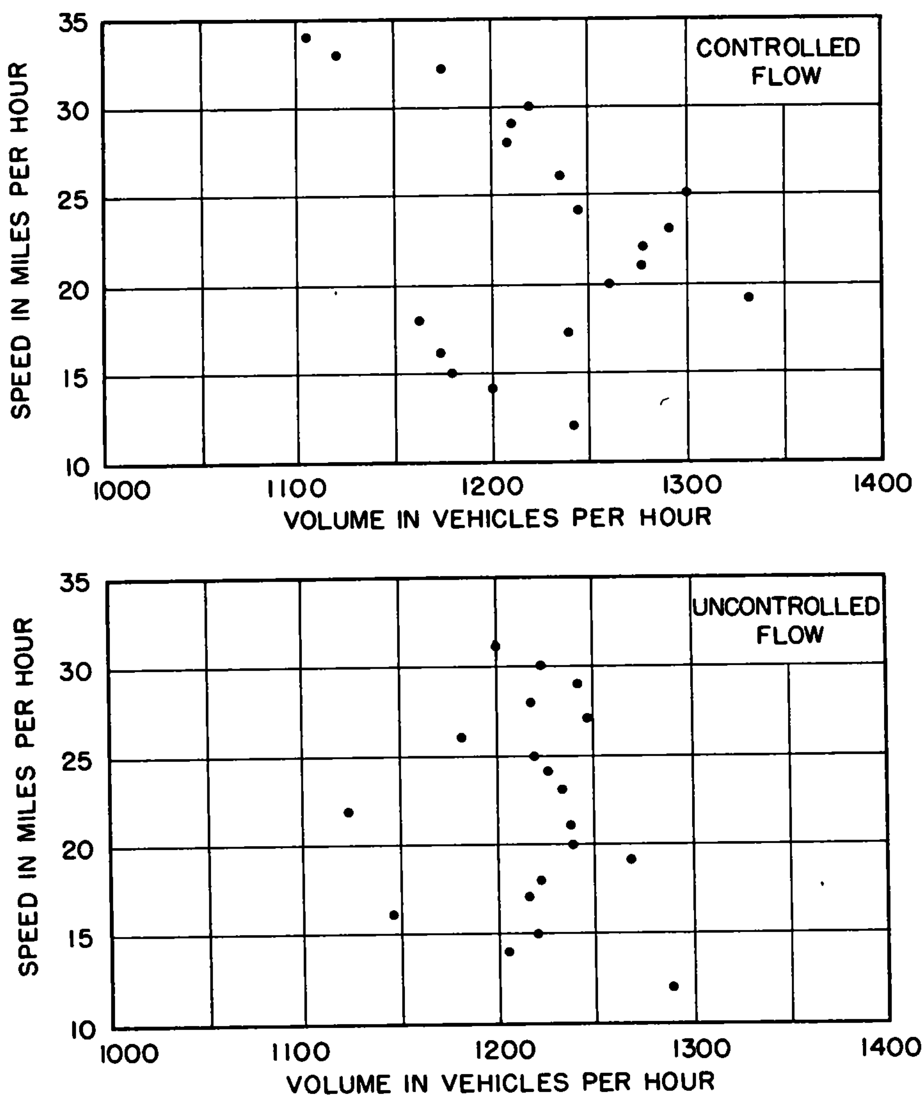


Figure 16. Bottleneck volume—speed relationship.

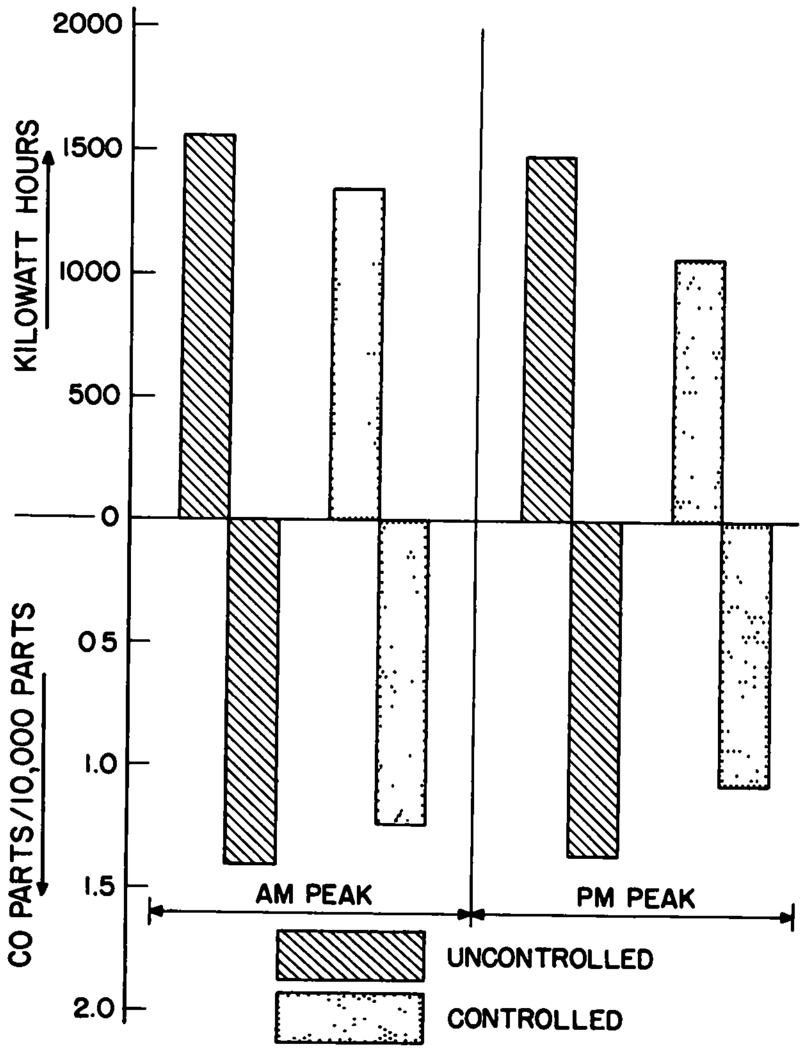


Figure 17. Power and carbon monoxide comparison.

6. A second radar speed sensor similar to the first was mounted 2,500 ft upstream (back toward the entrance) from the bottleneck.

7. Two systems of ASD Monitor equipment were provided (Fig. 21) in the Traffic Flow Monitor Center established for this test near the tunnel upgrade. This location is where the controller responsible for determining the proper setting for the input flow rate each minute was stationed. There, the staff member continuously "played" tunnel traffic, seeking to adjust the rate of traffic entering the tunnel to a level which would result in fluid, high rate flow through the bottleneck more than a mile inside the tunnel from the entrance portal. When the entering level was set too high, tunnel traffic became congested. When the entering level was set too low, the tunnel became starved for traffic and again, traffic production through the bottleneck would drop.

8. A one-camera, one-monitor closed circuit television system was installed to provide qualitative information on traffic flow conditions at the bottleneck.

Although this system did perform effectively during the test, it was evident to the experimenters that their control could be more effective. Analysis of the causes of

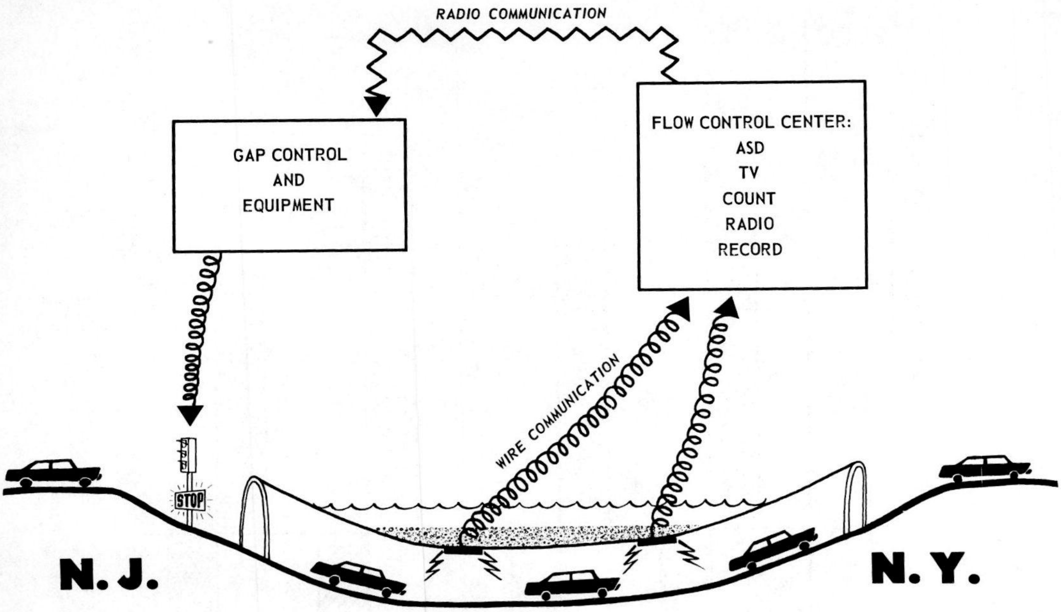


Figure 18. Traffic flow control cycle.

Figure 19. Entrance traffic signals.

production loss (Fig. 22) shows that in 39 percent of the cases when flow at the bottleneck was below average, the reason was inadequate control.

It became evident that measuring the speed and flow of traffic at the bottleneck alone was not enough to guarantee fully effective operation of the flow control system. As a general rule, the effectiveness of the flow control operation depended directly on the amount of information available to the controller. The more known about the current state of traffic flow through the tunnel, the more effective was the strategy followed by the controller in maintaining optimal flow.

The need for more information first arose as the effect of the time lag built into the initial system became clear. Observing flow and speed 6,000 ft downstream from the entrance portal, the controller acted on a situation over which he had no control until the traffic in the "pipeline" between the entrance and the bottleneck at the time of his decision passed through the bottleneck. This would require from 4 min to much longer times, depending on the amount of traffic in the pipeline. If the controller had observed traffic speeds below optimum at the bottleneck and decided that entering flow should be reduced, his decision would be wrong if there turned out to be relatively little traffic in the pipeline. In that case the bottleneck would soon lack traffic, and speeds would rise above optimum.

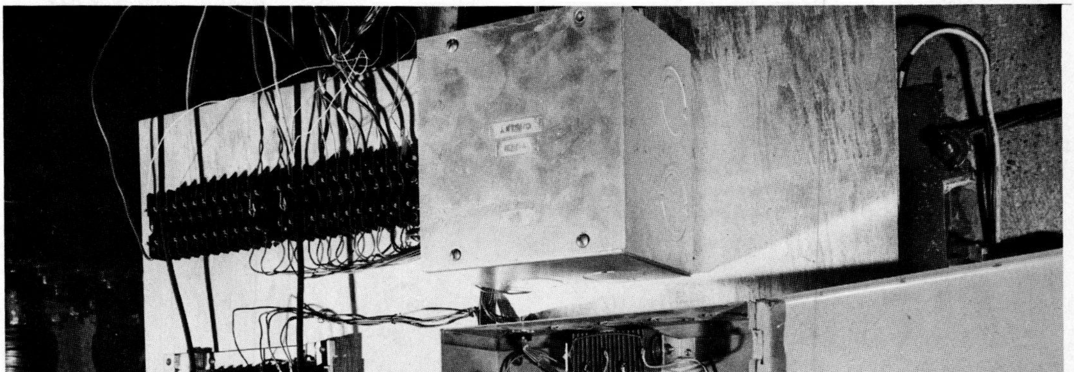


Figure 20. Traffic spacing computer.

Figure 21. Traffic flow monitoring center.

The principal conclusion drawn from this test was that the most effective system would probably be completely automatic. With an automatic system a large number of information points can be monitored continuously and consistent action strategies can be followed with immediate feedback of information on the effect of the strategy.

The test was not long under way before it became apparent that the least accurate piece of equipment in the system was the observer. Trying to exercise continuous control of stream flow for long periods of time was found to be extremely demanding on the individual. It required intense and uninterrupted concentration. The controller had to regularly evaluate information being received from several sources as well as consider the probable effects of his decisions on the traffic stream.

In view of the frequency and rapidity with which the state of traffic flow changes in the tunnel it was not possible for a man to provide the frequent and regular alterations in the entering flow needed to maintain a fluid high volume stream of traffic through the bottleneck.

Automatic Equipment

Based on these experiments, the Port Authority has now developed what might be

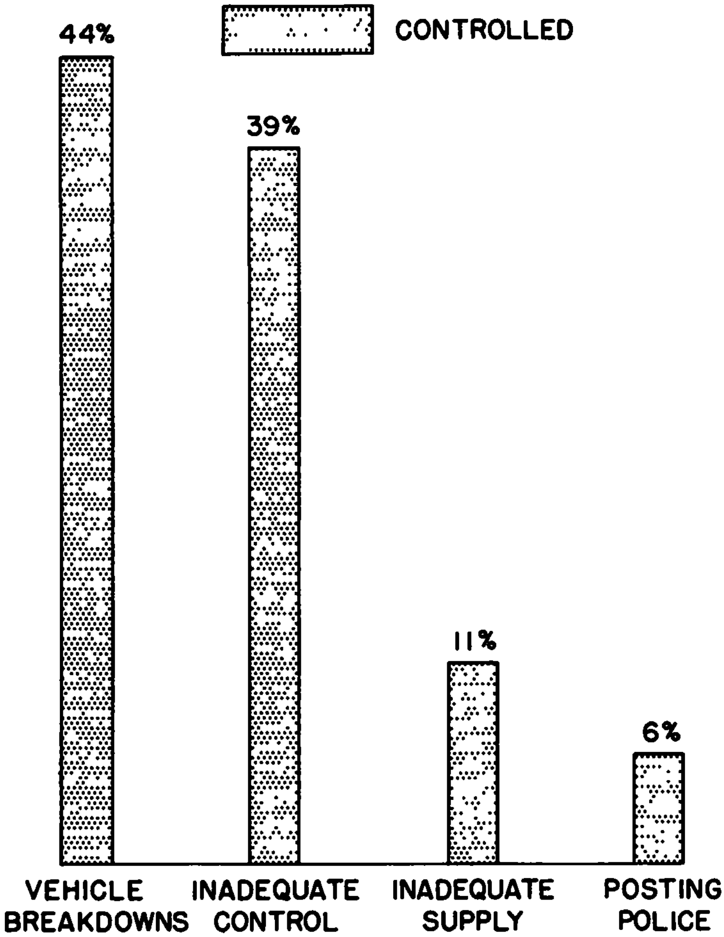


Figure 22. Causes of production loss.

described as a "first generation" prototype automatic system for controlling traffic flow.

The prototype system consists of three main elements:

1. Two sets of photocells—each set bounding a 13-ft zone for continually measuring the speed and flow of traffic through the zone. One set will be placed at the bottleneck (foot of the upgrade), and the second set will be 1,500 ft upstream. These two sets of photocells will be connected to the primary component of the new system, the flow computer.

2. The flow computer (Fig. 23)—located at the entrance to the Holland Tunnel South Tube. Each minute this computer will consider the number of vehicles which passed through the bottleneck in the preceding minute and then, by next considering the speeds of traffic approaching and at the bottleneck, establish a maximum number of vehicles which should enter the tunnel in the next minute. The prototype computer will not measure the speeds of vehicles exactly, but rather will determine when an excessive number of vehicles are going too slow or too fast to achieve the highest flow. This system has been entirely conceived, designed and built by Port Authority staff. Despite the relative simplicity of its design, the computer contains more than 70 relays. One of the most important features is the flexibility that has been built into the prototype computer, which will make it simple to change the action of the computer to improve its effectiveness as test operating experience is gained. After deciding each minute

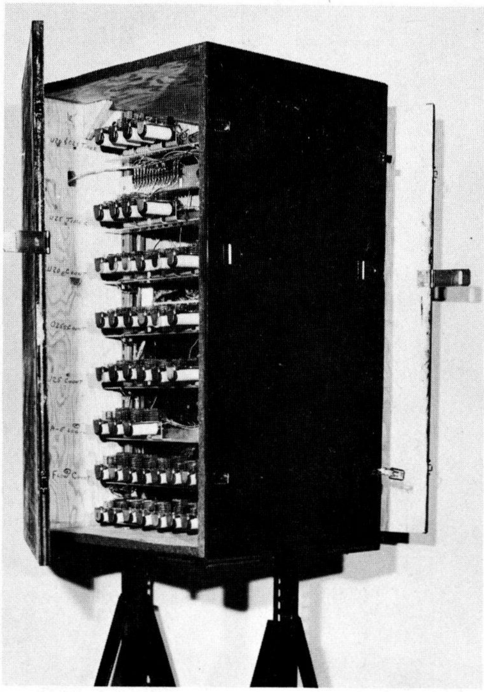


Figure 23. Prototype automatic flow computer.

the number of vehicles which should enter the tunnel in the next minute, the computer automatically adjusts a traffic spacer.

3. The traffic spacer—also located at the entrance to the Holland Tunnel South Tube. This element will perform essentially the same functions as the equipment shown in Figures 22 and 23, but will be compatible with the new flow computer. From an ultrasonic or induction vehicle detector placed at the tunnel near lane entrance, this traffic spacer will count the number of vehicles entering the tunnel. When the amount which has entered in less than a minute is equal to the amount predetermined by the flow computer, the traffic spacer will automatically turn the entrance signals red in both lanes for the remainder of that minute.

In testing this total system the Port Authority will use several recording devices to analyze the system operation. The main analytical tool is new Monitor equipment purchased from Automatic Signal Division.

It is most likely that this relatively straightforward flow control system will require a considerable amount of improvement both in functional design and in hardware. This system is the first prototype for accomplishing a job which is new in the field of traffic control, and one of the main values of this test will come in gaining new knowledge about traffic flow as the system is developed further.

Some of the specific points which will be considered are:

1. What is the effect on various types of traffic flow of various settings for the flow computer? What settings are best? What periodic or seasonal adjustments are desirable?

2. Are the very general speed divisions in this first prototype sensitive enough, or is it desirable to measure speeds in greater detail?

3. Should additional speed measurement points be added, and if so, where?

4. Are speed and flow the best traffic characteristics to measure, or should the system measure density, speed variance, or other characteristics? If density should

be measured, can this be done best by measuring flow at the entrance for a 3-, 4- or 5-min period?

5. Should the flow computer also consider the amount of traffic which actually entered the tunnel, and adjust its traffic spacing decisions up or down depending on the discrepancy between planned and actual traffic input?

6. How well does the far lane operate under this proposed system where input decisions are based entirely on the near lane? Should this near lane system be augmented with speed, flow or other sensors at critical locations in the far lane? Would a separate flow computer system for the far lane be desirable?

These questions show the extent of the study that will be needed to evaluate functional aspects of the flow control system. The answers will determine the design of a system likely to be most effective, and hence will strongly influence further development of the hardware aspects of this system. Here, the questions that may have to be considered are equipment reliability, whether relays are the best counting and timing devices for this type of circuitry (they probably are not, but at this stage they are easiest to work with), whether the Port Authority should construct a more advanced prototype, and when consultants and/or commercial manufacturers should be used to build more units.

Based on studies conducted to date, it seems likely that flow control systems of this type can be a significant help to engineers and roadway operators in gaining more effective traffic service from congested roadways. As these tests and developments are carried forward, the Port Authority will be pleased to make its findings available.

ACKNOWLEDGMENTS

The work reported in this paper is being carried forward through the active support and endeavors of many staff members of The Port of New York Authority. All major decisions relating to the development of the tunnel policing system are made by the Director of Tunnels and Bridges, Charles H. Taylor, after review by a special group of top Tunnel and Bridge management consisting of August Z. Schneider (Deputy Director of Tunnels and Bridges), Leslie C. Edie (Chief, Project and Planning Division), George E. Stickle (General Manager), J. Douglas Maynard (Manager of the Holland Tunnel) and Arthur Tate (Manager of the Lincoln Tunnel).

The traffic flow work is carried on under the primary influence and stimulus of Mr. Edie. Traffic flow experiments recorded in this paper were conducted under the immediate supervision of Kenneth W. Crowley and Alan T. Gonseth, who have also played a primary role in developing a functional specification for the automatic flow control system. Robert A. Hauslen and Carroll F. White have contributed to the design of this automatic flow control system. Tunnel management, especially Mr. Maynard and his staff at the Holland Tunnel, have been of great assistance in this work. Credit for successful performance of the flow control system is due Charles Kneeter and Daniel Cook under whose supervision the equipment has been assembled.

John D. Barbieri is of great assistance in coordinating the tunnel policing work, and Messrs. Hauslen and White have contributed much to the development of the electro-mechanical systems. Henry Wenson and Mr. Kneeter have played major roles in developing essential components for the tunnel policing system. The Port Authority's Traffic Engineer, Louis E. Bender, and his staff are participating in development of the traffic diversion system.

REFERENCES

1. Edie, L.C., and Foote, R.S., "Effect of Shock Waves on Tunnel Traffic Flow." HRB Proc. (1960).
2. Greenberg, H., and Daou, A., "The Control of Traffic Flow to Increase Flow." Journal of Operations Research, 8:4 (July-Aug., 1960).
3. Palmer, M.R., "The Development of Traffic Congestion." Yale University Bureau of Highway Traffic (May 1959).
4. Foote, R.S., Crowley, K.W., and Gonseth, A.T., "Instrumentation for Improved Traffic Flow." Proc., Institute of Traffic Engineers (1960).

Operational Study–Schuylkill Expressway

ROBERT H. PEARSON and MICHAEL G. FERRERI, Traffic Engineers, Simpson and Curtin, Philadelphia, Pennsylvania

In February and March of 1960, a study was made of traffic operations on the Schuylkill Expressway in Philadelphia to determine design deficiencies and the necessary remedies. Studies included mechanical and manual volume counts, lane distribution, vehicle classification, radar speed distribution, travel time, delays to ramp vehicles, gap acceptance and rejection at ramps, a review of accident experience, and a motion picture analysis of peak-hour Expressway conditions. Ramp capacity studies were made at several on-ramps which have little or no acceleration lanes (0 -150 ft) and are controlled by stop signs.

The data were recorded in sufficient detail to yield: (a) frequency and time length of gaps in the Expressway shoulder lane, (b) the time length of each accepted and rejected gap, (c) the time length of gaps accepted by a queue of vehicles, (d) the speed of each shoulder lane vehicle, (e) total delay to each ramp vehicle, and (f) the delay to each ramp vehicle while waiting as the first vehicle in line. Data were recorded in peak hours with ramps under constant pressure. Information was obtained manually using stop watches.

Using this information, a high coefficient of correlation was obtained for a curve of ramp capacity as a function of shoulder lane volume. Several other variables were examined to determine their effect on ramp capacity.

During peak hours, movies were taken at critical locations along the Expressway and at on-ramp merging areas. These movies showed the merging problems, build-ups of congestion, and the subsequent reductions in capacity. It was possible to measure speeds and volumes from these movies.

Mechanical counts of volumes on each ramp and between each interchange were made. These were supplemented by manual counts of vehicle classification and lane distribution.

Radar speed distribution measurements were made between interchanges at points corresponding to manual volume counts. Travel time studies were made in peak and off-peak hours, recording the elapsed distance every minute to precisely pinpoint areas of congestion and draw a speed-distance curve for the entire Expressway. Travel time studies were repeated in June to determine the effect of slightly higher volumes.

●THIS PAPER is a summary of a detailed report submitted to the Pennsylvania Department of Highways for determining the extent of existing and future deficiencies on the Schuylkill Expressway and the nature of improvements to correct these deficiencies.

Studies included manual and mechanical volume counts, radar speed surveys, time delay studies, lane distribution, vehicle classification, ramp studies, a review of accident experience, and motion pictures of peak-hour Expressway conditions.

These studies provided the factual data for analysis of Expressway deficiencies, traffic demands and operations, and guided determinations for immediate and future improvements designed to expedite traffic flow with increased safety.

The scope and intent of this paper is not to cover fully all the details of the report submitted to the Pennsylvania Department of Highways, but to summarize the types of studies made, and more specifically to point out the usefulness of the techniques used in the ramp studies for acquiring data to determine the need for ramp corrections.

The Schuylkill Expressway (Fig. 1) is Philadelphia's first radial limited-access highway and consequently is subjected to tremendous peak-hour volumes and subsequent congestion. Following its completion, the Expressway was made a part of the State's interstate highway system, designated as Interstate 80S and 680. Its termini are the Pennsylvania Turnpike on the west and the Walt Whitman Bridge to New Jersey on the east. It provides connections with all major arterial streets and highways between the Turnpike and Bridge and is one of the principal means of access to center city Philadelphia. Starting from its northern connection with the Pennsylvania Turnpike, the Expressway was constructed and opened in stages beginning in 1950.

The Expressway is adjacent, and substantially parallel, to the West River and East River Drives, between City Avenue and Spring Garden Street interchanges. It is the only limited-access facility in the area and, as a consequence, has diverted substantial traffic volumes from each of the two River Drives, as well as from other routes to and from, or through, center city. The generally superior character of the Expressway makes it particularly attractive to commuter traffic, traveling daily between center city and the suburbs. It, therefore, develops peak-hour morning and evening volumes, ranging from 2 to 3 times average midday volumes.

TRAFFIC

Approximately 165,000 vehicles use the Expressway on an average weekday in 1960. Highest volumes are in that section of the Expressway between Spring Garden Street and Girard Avenue. Average 1960 weekday volumes in this section are 88,666 vehicles.

The highest one-directional volume of traffic within a 60-min period during the study was recorded outbound between 4:30 and 5:30 p.m., between Spring Garden Street and Girard Avenue—5,720 vehicles or an average of slightly more than 1,900 vehicles per lane in this three-lane (directional) section of highway. A simultaneous record of the lane distribution of traffic at this location showed 2,489 vehicles westbound in the median lane.

The Expressway interchange with the highest volume of traffic is the connection to and from Vine Street—53,361 vehicles entered or left the Expressway at this point. Of this number, 38,492 were to and from the west, and 14,869 were to and from the east.

Between Vine Street and City Avenue interchanges, the 1960 peak-hour volumes of traffic exceed the practical capacity of the Expressway. During commuter periods, particularly the evening homebound rush hours, the pressure of traffic on this section of the Expressway is such that vehicle spacing forces operating speeds considerably below the 50-mph posted speed limit. During those times, the operating sensitivity is such that the slightest disruption of flow, caused by an incident such as a vehicle breakdown, results in stop-and-go operations over long distances and for considerable periods of time after the occurrence has been cleared.

INTERCHANGE CAPACITY

With few exceptions, interchange capacity is limited by mandatory stops at ramp entrances to the Expressway; made necessary because of inadequate acceleration lanes. Yield signs have replaced the stop signs since this study was completed. Off-ramp operations are often hindered by surface street traffic controls, inadequate storage, and lack of proper deceleration lanes. These conditions are incompatible with modern limited-access highway design.

Studies indicate that on-ramp capacity can be increased up to 60 percent by providing adequate acceleration lanes.

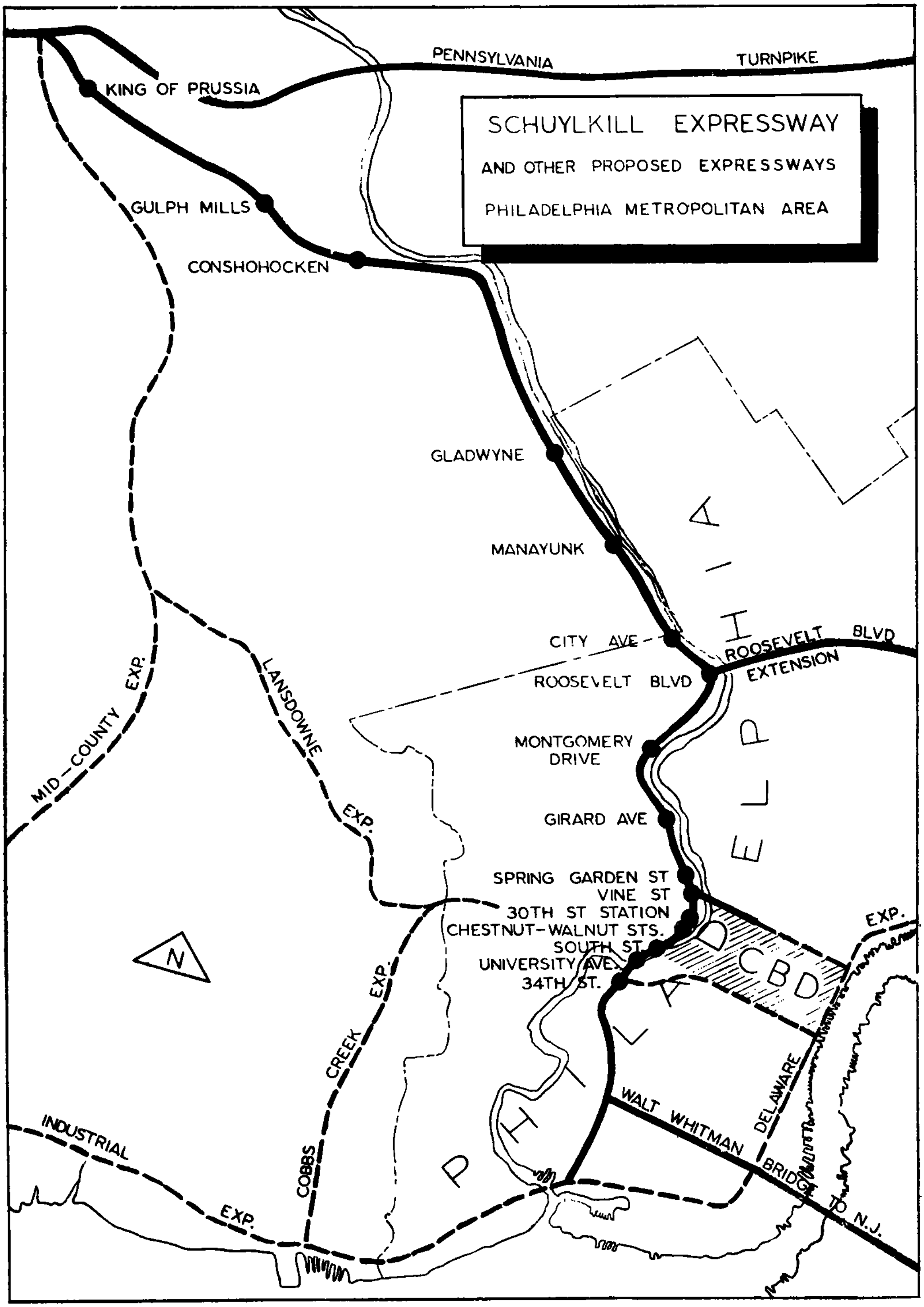

SCHUYLKILL EXPRESSWAY
AND OTHER PROPOSED EXPRESSWAYS
PHILADELPHIA METROPOLITAN AREA
PENNSYLVANIA TURNPIKE
KING OF PRUSSIA
GULPH MILLS
CONSHOHOCKEN
GLADWYNE
MANAYUNK
CITY AVE
ROOSEVELT BLVD
ROOSEVELT BLVD EXTENSION
MONTGOMERY DRIVE
GIRARD AVE
SPRING GARDEN ST
VINE ST
30TH ST STATION
CHESTNUT-WALNUT STS.
SOUTH ST.
UNIVERSITY AVE.
34TH ST.
PHILADELPHIA
CBD
MID-COUNTY EXP.
LANSDOWNE EXP.
COBBS CREEK EXP.
INDUSTRIAL EXP.
DELAWARE EXP.
WALT WHITMAN BRIDGE TO N.J.
N

Figure 1.

ACCIDENTS

Analysis of reports of accidents occurring on the Expressway indicate a fatality rate—based on exposure—greater than that of the Pennsylvania Turnpike and the New Jersey Turnpike, the two existing limited-access facilities transversing the Philadelphia metropolitan area.

	Year	Deaths Per 100 Million Vehicle-Miles
Schuylkill Expressway	1959	4.1
Pennsylvania Turnpike	1959	3.7
New Jersey Turnpike	1959	1.5

TRAFFIC DISTRIBUTION, PARALLEL ROADS

On an average weekday, 8,800 vehicles travel inbound along both sides of the Schuylkill River during the morning eastbound peak hour. In the maximum load area, this volume moves along seven eastbound lanes of roadway—three on the Schuylkill Expressway and two each on the East and West River Drives. The westbound evening peak hour is only slightly less—8,700.

Eastbound, the three Expressway lanes are used by 56.8 percent of this traffic and the four River Drive lanes the remainder. Westbound, the distribution is 65.5 percent on the Expressway and 34.5 percent on the River Drives.

FUTURE TRAFFIC

It is anticipated that traffic volume in 1975 will be 45 percent higher than at present.

In the maximum load section—between the Girard Avenue and Spring Garden Street Interchanges—1975 traffic will increase from 88,666 to an estimated average daily volume of 128,300.

This estimate gives effect to the influence of a fully interconnected Expressway system in Metropolitan Philadelphia. If this system is not completed by 1975, the demand on the Expressway will be even greater.

Design hour estimates are based on peak hour and major directional percentages of 10 percent and 55 percent west of the Gladwyne Interchange, 10 percent and 60 percent from Gladwyne to the City Avenue Interchange, 9 percent and 65 percent between City Avenue and the Vine Street Ramps and east of Vine Street, 10 percent and 55 percent, respectively.

TRAFFIC VOLUME STUDIES

Traffic volumes on the Expressway, ramps and feeder roads were determined by manual and machine counts during February and March 1960. Machine counts were made at 100 locations and manual counts at 19 locations, nine of which also served as a check on machine counts.

Surface street volumes and turning movements at several locations near the entrance to, or exit from, Expressway ramps were made to determine directional flow of traffic at those points.

Each machine count was made for a minimum of one week; thus the relationship of one weekday to another was established as well as the average weekday.

Vehicle classification and lane distribution manual counts were made at nine locations along the Expressway. These counts were conducted from 7:00 to 10:00 a.m., 11:00 a.m. to 3:00 p.m. and from 4:00 to 7:00 p.m. at each location. Checkers were assigned to each lane and recorded by ½-hr periods the volume in that lane segregated by vehicle type.

Checks on the reliability of machine counts, unadjusted for dual rear-axle vehicles, indicate a satisfactory degree of accuracy—less than 3 percent difference between simultaneous machine and manual counts.

The day-to-day variation in traffic on the Expressway, Monday through Thursday is slight. Friday is consistently the heaviest traveled weekday and normally presents the maximum traffic condition. Weekends show a substantial drop in volume from the average weekday.

Vehicle Classification

In the classification counts made at nine locations on the Expressway, vehicles were classified as passenger cars, light trucks (panel, pickup, etc.), heavy trucks, tractor-trailers and buses.

During the 10-hr period in which vehicle classification checks were made, heavy trucks and tractor-trailer combinations comprised approximately 12 percent to 16 percent of the total traffic west of the City Avenue Interchange, 6 percent to 8.5 percent between the City Avenue and Vine Street Interchanges, and from 9 percent to 14 percent east of the Vine Street Interchange.

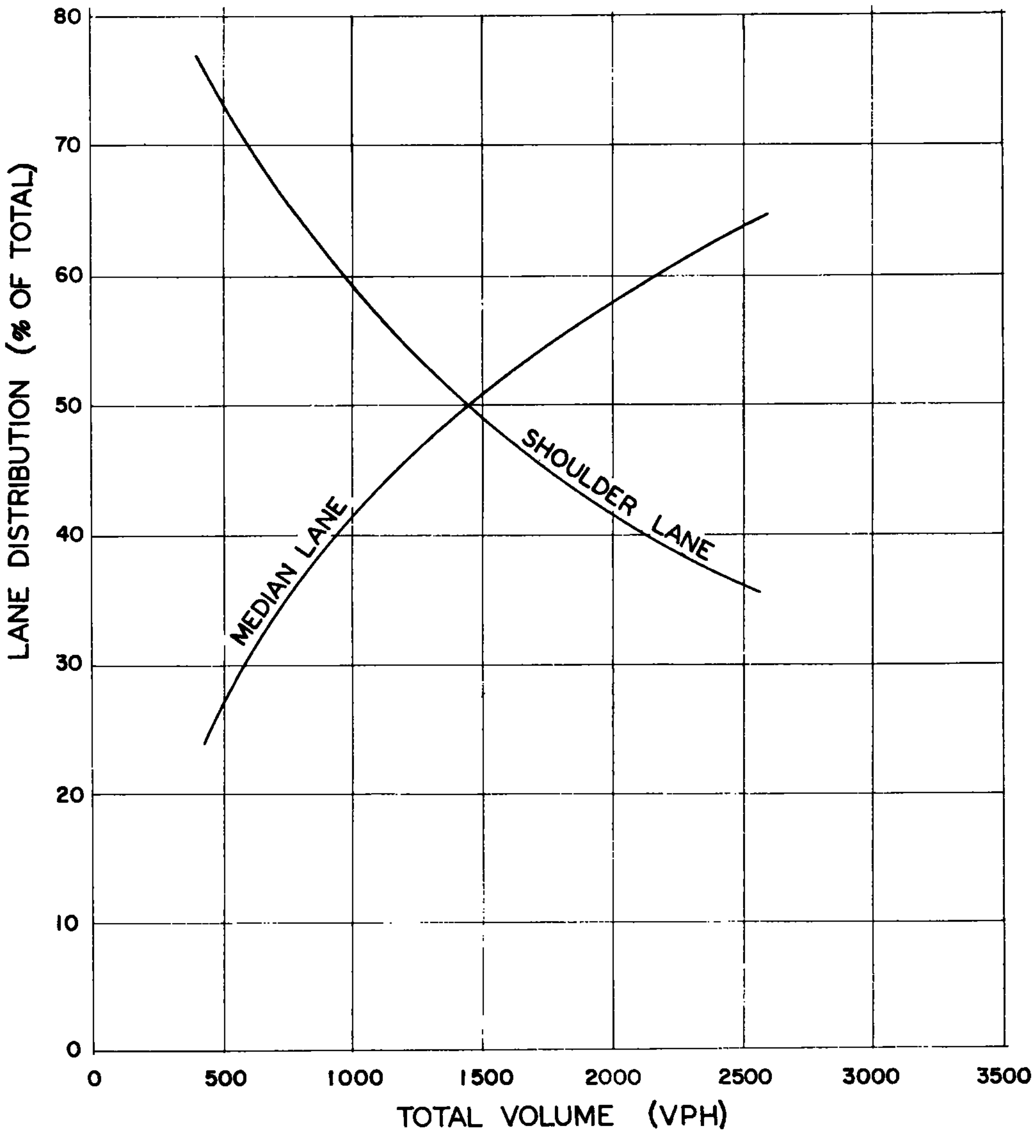


Figure 2. Schuylkill Expressway, lane distribution (2-lane section).

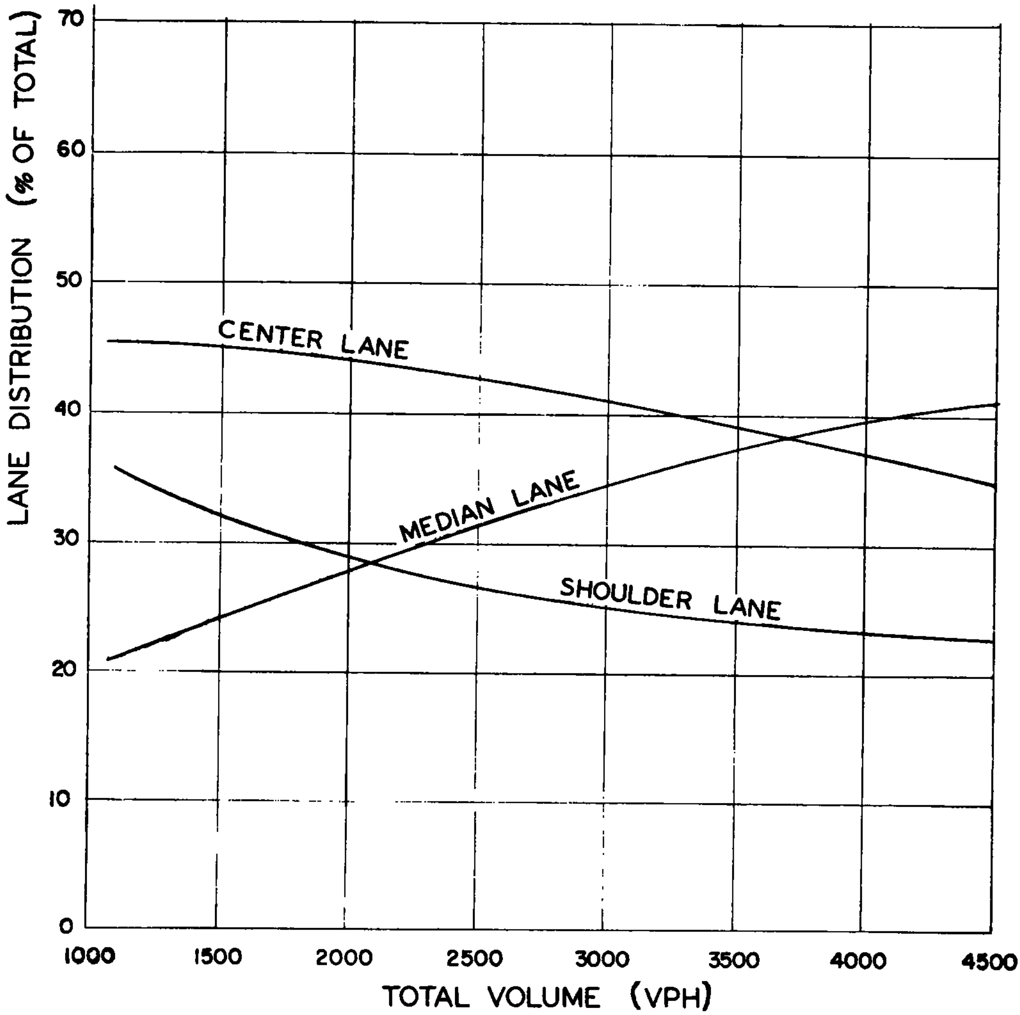


Figure 3. Schuylkill Expressway, lane distribution (3-lane section).

Lane Distribution

Figure 2 shows the average distribution of vehicles in the median and shoulder lanes in the two-lane sections of the Expressway in relation to total volume. Figure 3 shows comparable data for the three-lane sections.

Lane distribution in each direction was determined for heavy trucks and also for total vehicles during midday, a. m. and p. m. peak periods.

Radar Speed Studies

Vehicle speeds were measured by radar speed meters at 11 locations on the Expressway. Nine of these checks were made at the same locations as the vehicle classification and lane distribution counts. By measuring speeds and volumes simultaneously a close control of the speed-check sample was possible and major fluctuations in the speed-volume relationship could be recorded.

Samples were designed to yield average speeds which would be accurate within ± 1 mph.

At each location, speeds were measured by lane and vehicle type in the morning from 7:00 a. m. to 9:00 a. m., midday between 9:00 a. m. and 4:00 p. m. and in the evening from 4:00 p. m. to 6:00 p. m. Speeds were also recorded in both directions at all locations except those east of Vine Street where peak-hour speeds were recorded only in the direction of major flow.

Lane Speeds

Average speeds in the shoulder lane were found to be, in general 1 to 3 mph slower than the center or median lane. Speed observations of vehicles operating in the median lane of the three-lane sections could not be accurately recorded because of interference from vehicles in the other two lanes.

Speed Violations

The highest percentage of passenger car speed violators (13.9 percent above 50 mph and 2.1 percent above 55 mph was recorded for vehicles operating in both directions between City Avenue and Montgomery Drive, during the midday. This percentage reflects the change from 60 to 50 mph in the legal speed limit at City Avenue.

In general, automobile speeds are 5 to 7 miles faster than heavy trucks and tractor-trailer speeds west of City Avenue where the passenger vehicle limit is 60 mph and the truck limit 50 mph. Where the speed is the same for both vehicle types (50 mph), truck speeds are from 1 to 3 mph lower than passenger vehicles.

Operating Speeds

To measure speed variations and time consumed for single trips during commuter hours and midday, a series of operating speed runs were made by the average vehicle method.

METHOD OF STUDY

At the end of each 60-sec interval, a recorder noted the odometer reading. This method of recording resulted in an adequate number of readings per mile in free-flowing sections, and more importantly, increased the number of readings for each mile of travel in congested areas.

All runs were made on weekdays during the month of March and included both directions over the entire length of the Expressway.

Midday Studies

During off-peak hours, the only deterrent to high speeds is the enforcement of the posted speed limits. The average over-all speeds are 50.6 mph eastbound and 50.9 mph westbound. The average midday running time in either direction for the entire length of the Expressway is under 24 min.

Peak-Hour Studies

Average eastbound speeds between 8:00 and 9:00 a.m. are 40.3 mph; 28 percent below the weighted average posted speed limits, and 21 percent below average midday speeds. Westbound in the evening peak, speeds are of the same order, averaging 40.1 mph.

Speed and Delay

Average running time between King of Prussia and Passyunk Avenue is 24 min in either direction. During commuter hours, 30 min is required on the average. The maximum observed eastbound time was 35.6 min. The westbound maximum was 42.7 min, largely resulting from the "squeeze left" (3-lane to 2-lane transition section).

The critical section of the Expressway, between the Gladwyne and Vine Street Interchanges comprises 40 percent of the Expressway length, and accounts for 85 percent of the commuter hour lost time due to congestion. Time lost in commuter periods due to congestion is computed as the differential between midday and commuter period times.

RAMP CAPACITY STUDIES

Detailed studies were made at four heavily traveled on-ramps to establish quantitatively the extent to which present geometrics and regulatory devices tend to depress ramp capacities.

Specifically, the purpose of these studies was to determine: (a) capacities of on-ramps as presently designed; (b) delays to ramp vehicles; (c) relationship between ramp capacities and other operational variables such as Expressway speeds, shoulder lane volumes, frequency of shoulder lane gaps of various lengths and the characteristics of shoulder lane traffic; and, (d) criteria for the justification of improved ramp geometrics.

Procedure

The four ramps that were selected for study were among the most heavily traveled on the Expressway.

They were located at four contiguous interchanges, as follows, beginning at the west: (a) eastbound at Gladwyne Interchange; (b) eastbound at Manayunk Interchange; (c) westbound at City Avenue; and (d) eastbound at Montgomery Drive. The periods in which the studies were conducted were those during which the ramps were under maximum pressure. Thus, in most cases, vehicles were constantly stored on the ramps waiting acceptable Expressway gaps. At each ramp, data were collected for three successive periods approximating 15 to 20 min each, with breaks of 3 to 5 min between each period.

A team of five men was required, located as follows (Fig. 4): (a) two men at the ramp nose to record shoulder lane headways and merging vehicles; (b) two men on the Expressway approximately 200 to 300 ft in advance of the ramp nose to record by radar the speed of each shoulder lane vehicle; and, (c) one man "floating" on the ramp to record the time of the initial stop of each ramp vehicle and the number of vehicles then waiting on the ramp. Stop watches were used by the radar observers, by the men at the ramp nose and by the floating man, to enable accurate time observations. All watches were started simultaneously just before the beginning of each observation period.

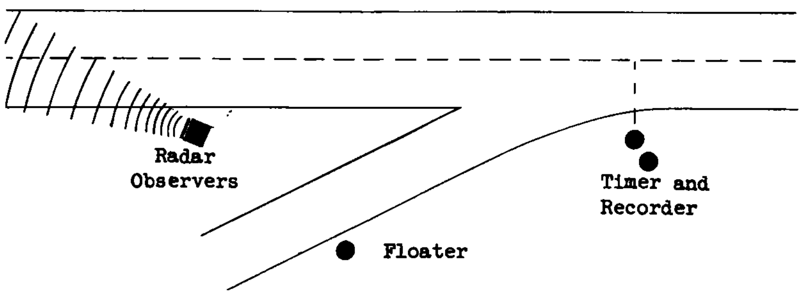


Figure 4.

One of the men at the nose of the ramp observed the exact second that each shoulder lane vehicle and each ramp vehicle reached the point where the ramp and Expressway merge; the other man recorded these data. Figure 5 shows the form used.

Radar speeds of each shoulder lane vehicle were recorded and classified, to be matched with vehicles recorded at the ramp nose. Figure 6 shows the form used for this purpose.

Figure 7 was used to record observations by the floating observer on the ramp.

Vehicle types were recorded at all locations to facilitate "matching" the entering and exiting recordings.

At the end of each study period, data sheets were collected and the stop watches were again synchronized for the start of the next period.

SCHUYLKILL EXPRESSWAY (PDH)

SHOULDER LANE HEADWAY - RAMP MERGING

Interchange ____________________

Shoulder Lane ________ Bound Grade ______

Ramp ON OFF Length __________

Grade _____ Date ____________

Weather ________________________

Vehicle Classification

S = Small Car (Exc. Rambler Size)
W = Station Wagon
C = Cars other than S or W
P = Pickup
T = Truck
I = Tractor Trailer
B = Bus

Time AM PM	Shoulder Lane Time Sec.	Merge Sec.	Head-way Sec.	Ramp Veh. Wait-ing as 1st Veh. Sec.	Total Wait-ing Time Sec.

Time AM PM	Shoulder Lane Time Sec.	Merge Sec.	Head-way Sec.	Ramp Veh. Wait-ing as 1st Veh. Sec.	Total Wait-ing Time Sec.

Figure 5.

SCHUYLKILL EXPRESSWAY (PDH)

SHOULDER LANE VEHICULAR SPEED - BEFORE RAMP

Interchange ______________________________

Shoulder Lane ________ Bound Grade ____________

Ramp On Off

Date ______________________

Weather _____________________________

Vehicle Classification

T = Truck

I = Tractor Trailer

B = Bus

Time AM PM	Time Sec.	Speed MPH	Time AM PM	Time Sec.	Speed MPH	Time AM PM	Time Sec.	Speed MPH	Time AM PM	Time Sec.	Speed MPH

Figure 6.

SCHUYLKILL EXPRESSWAY (PDH)

RAMP VEHICLE DELAY

Interchange ______________________

Shoulder Lane __________ Bound Grade _________

Ramp On Off Length

Grade Stop Sign No Sign

Vehicle Classification

S = Small Car (Excl. Rambler Size)
W = Station Wagon
C = Cars Other Than S or W
P = Pickup
T = Truck
B = Bus
I = Tractor Trailer

Time AM PM	Time of First Stop Sec.	No. of Veh. in Front	Time AM PM	Time of First Stop Sec.	No. of Veh. in Front	Time AM PM	Time of First Stop Sec.	No. of Veh. in Front	Time AM PM	Time of First Stop Sec.	No. of Veh. in Front

Figure 7.

Preliminary summarization of field sheets yielded these data: (a) shoulder lane gap distribution; (b) distribution (by time length) of the accepted and rejected gaps; (c) distribution of gaps accepted by a queue of cars (for example, three cars accepting a single gap); (d) waiting time on ramp before merging; (e) waiting time on ramp as first vehicle in lane; and (f) speeds of lead and trail vehicles for each gap.

Delays to Ramp Vehicles

Figure 8 shows the delay to ramp vehicles as a function of their place in line on arrival at the ramp. The average delay ranged from 16.4 sec at the Gladwyne ramp to 33.3 sec per vehicle at the City Avenue ramp.

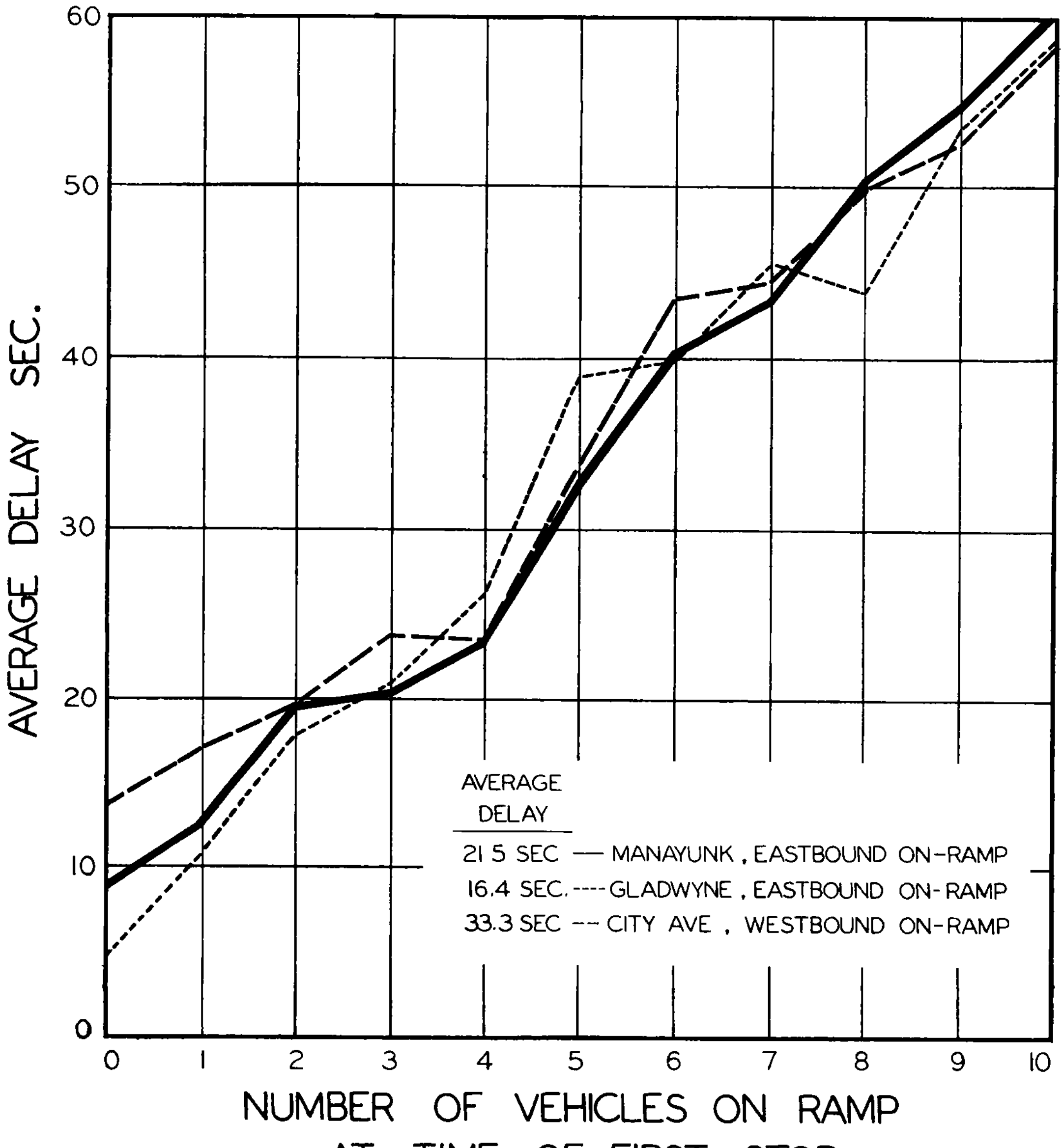


Figure 8. Average vehicle delay.

The delay curves of the on-ramps are similar; the maximum deviation being approximately 8 sec with only one vehicle on the ramp. Very few observations were obtained of delays over 60 sec.

Data for the Montgomery Drive on-ramp are not plotted on Figure 8 because delays were so long that the line of waiting vehicles stretched out of sight and made it impossible to record data properly. However, several cars were "clocked" from the time they first came into line until they moved into the shoulder lane. An average delay of 2.1 min was computed from eight observations. This condition is continuous for approximately 45 min to 1 hr each morning.

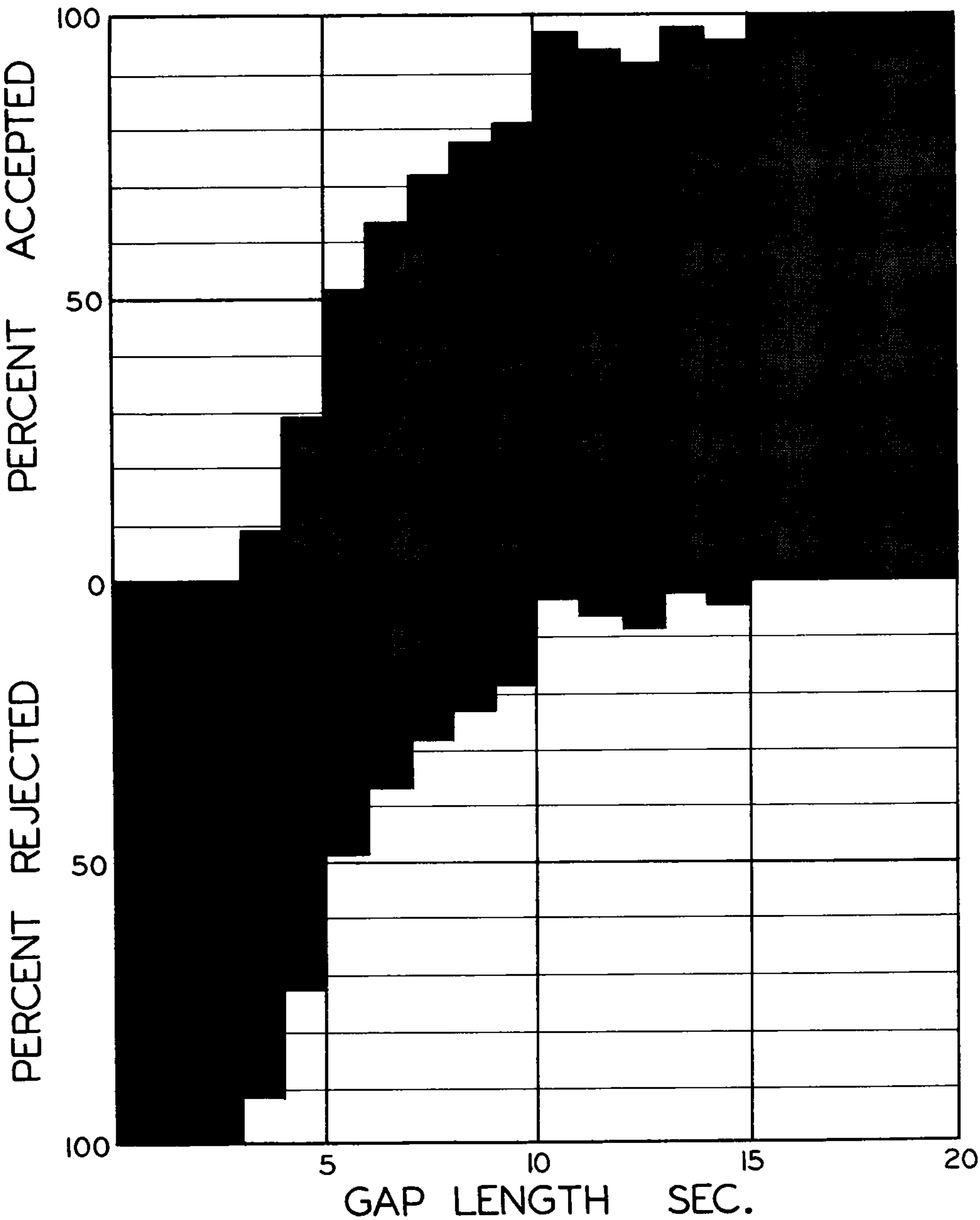


Figure 9. Gap acceptance and rejection composite on-ramps with stop signs (1,100 observations).

Ramp Capacities

The studies yielded 1,100 observations of gaps which were accepted or rejected by ramp vehicles. Figure 9 shows a composite summary of gap acceptance at these ramps. The plotted points demonstrate a close fit to parabolic curves. A correlation analysis of the percent of rejected gaps yielded a coefficient of correlation of -0.99 with the equation:

$$\log R = 2.377 - 0.114T$$

in which

R = the percent of rejected gaps
T = the gap length in seconds

The 50th percentile of accepted gaps is at 6 sec. This corresponds closely to a previous finding of 5 sec for stop sign locations (1).

Figure 10 illustrates the degree of acceptance of gaps by a queue of vehicles.

The data from Figure 10 are used as the basis for Figure 11, which illustrates the relationship between the length of gap and the number of vehicles per gap. The coefficient of correlation for Figure 11 is 0.94.

After establishing the number of vehicles that will accept each gap, only the frequency of occurrence of each gap length in a given time interval need be known to predict the number of ramp vehicles which will enter the Expressway. Because the hourly rate of vehicle flow in the shoulder lane ranged from 300 to 800 when gap distributions were determined, it was decided to use the Poisson distribution to compute gap distributions in and beyond this range. Several chi-square tests of computed versus actual gap distributions showed very close fits. No gap distributions for hourly flows above 1,100 vehicles per hour were computed inasmuch as randomness ceases to function at about this volume. As an example, consider the shoulder lane flowing at 800 vehicles per hour. Then, by a computation similar to that in Table 1 ramp capacities were computed for various shoulder lane volumes and plotted in Figure 12.

Data for the on-ramp with adequate acceleration lane curve are based primarily on studies made in California (2).

Table 2 compares Expressway ramp capacities—vehicles per hour—of an inadequate acceleration lane having a stop sign at the Expressway entrance with capacities of ramps having an adequate acceleration lane and no stop sign, for various shoulder lane volumes.

Off-Ramps

Specific problems exist at several off-ramps, due primarily to conditions at the local street end of the ramps, in addition to insufficient deceleration lanes. During the p.m. peak hour, westbound traffic leaving the Expressway at City Avenue and Manayunk often are stopped on the Expressway as the ramps at these locations are full.

Recommendations to improve these conditions were made at each critical location.

As an example of the technique used to apply the ramp study curves, the following is a typical interchange analysis.

Manayunk Interchange Recommendation

One of the most congested interchanges is the Manayunk Interchange. The present and future peak-hour volumes for Expressway lanes and the eastbound on-ramp are given in Table 3. The practical ramp capacity under present conditions (inadequate acceleration lane with stop sign) and with adequate acceleration lane based on the ramp study data is shown in Figure 12.

On-Ramp C is at capacity in 1960 and by providing an adequate acceleration lane, the ramp will not reach capacity again until 1973. If a third lane is added to the Expressway at that time, the ramp capacity will extend well beyond 1975.

Immediate lengthening of the acceleration and deceleration lanes at this interchange (Fig. 13) was recommended. In addition, it was recommended that Ramp D be widened to two lanes.

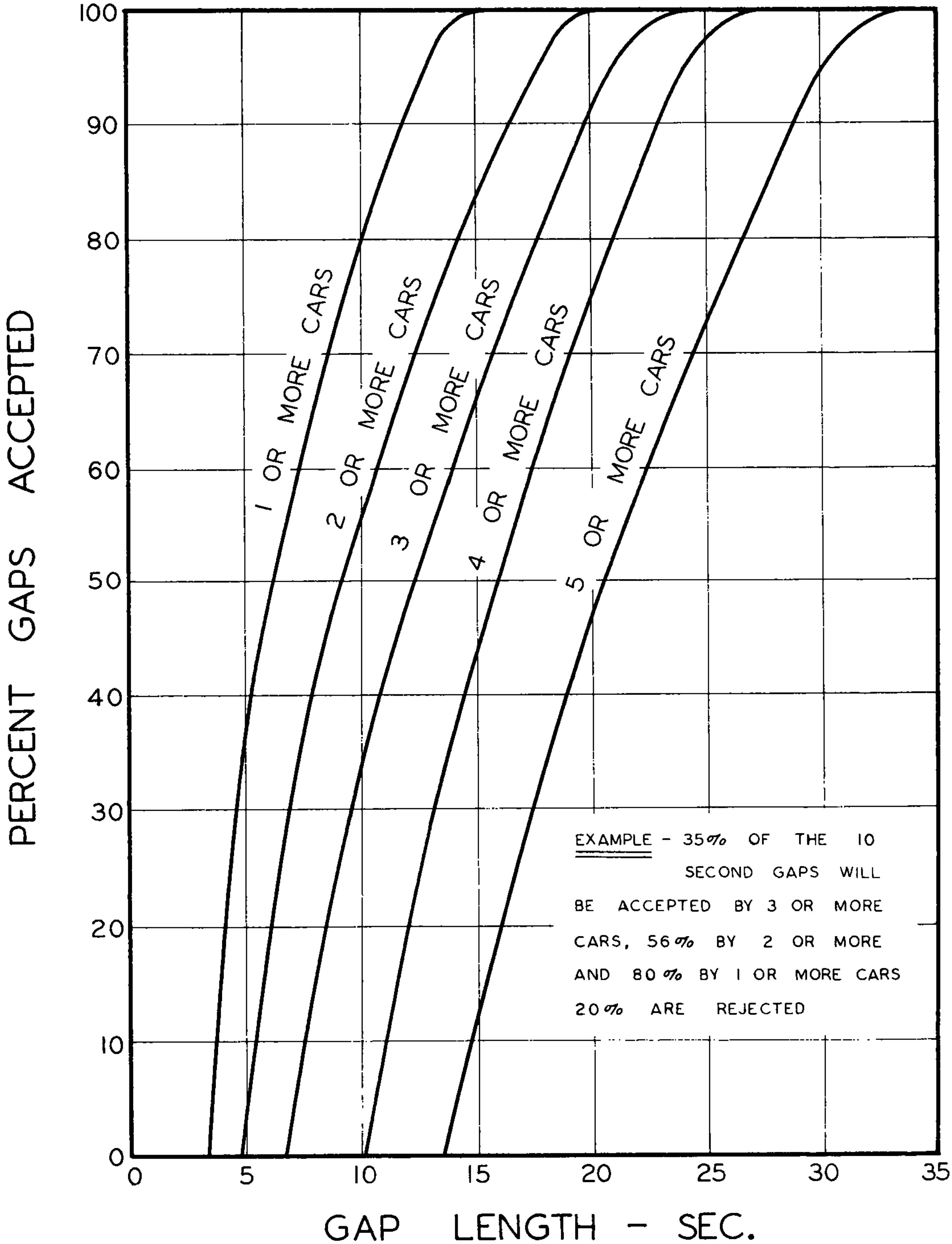


Figure 10. Gap acceptance by queues of vehicles.

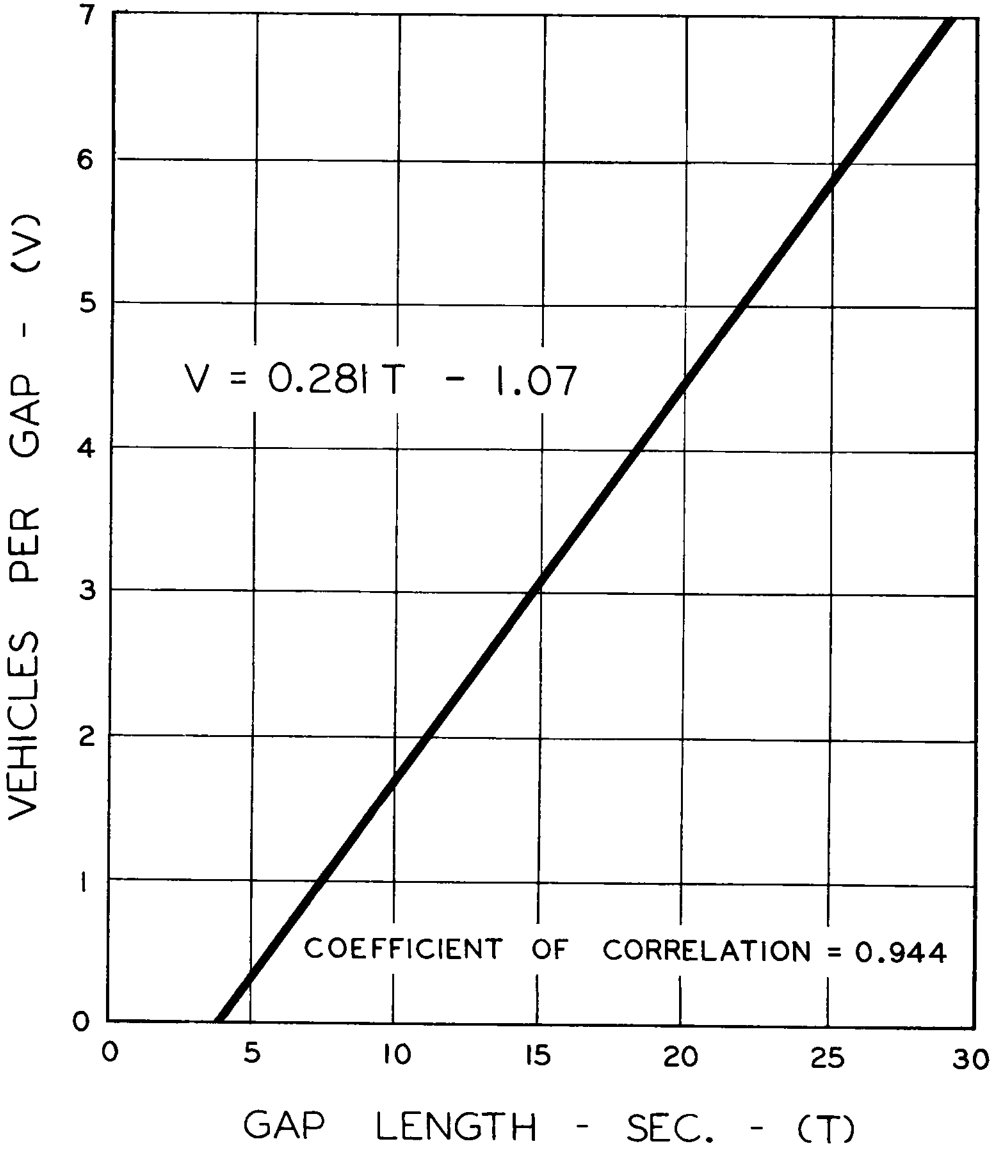


Figure 11. Vehicles per gap.

TABLE 1

Gap Length, T (sec)	Calc. No. of Gaps of T sec	Veh. (no.) Per Gap*	Total
0	-	-	0
1	159	-	0
2	127	-	0
3	103	-	0
4	82	0.05	4
5	65	0.33	21
6	53	0.61	32
7	42	0.89	37
8	34	1.17	40
9	27	1.45	39
10	22	1.73	38
11	17	2.01	34
12	14	2.30	32
13	10	2.58	26
14	10	2.86	29
15	7	3.14	22
16	6	3.42	21
17	5	3.70	19
18	4	3.98	16
19	3	4.27	13
20	2	4.55	9
21	2	4.83	10
22	1	5.11	5
23	1	5.39	5
24	1	5.67	6
25	1	5.95	6
26	1	6.24	6
27	0	6.52	0
28	0	6.80	0
29	0	7.08	0
30	0	7.36	0
30	1	8.77	9
	800	Ramp Capacity=	479 VPH

*Data from Figure 11.

TABLE 2

Shoulder Lane Volume	Ramp Capacity: Inadequate Acceleration Lane with Stop Sign	Ramp Capacity: With Adequate Acceleration Lane	% Increase
200	820	1,330	62
500	660	1,060	61
800	500	800	60
1,100	340	540	59
1,400	175	280	60

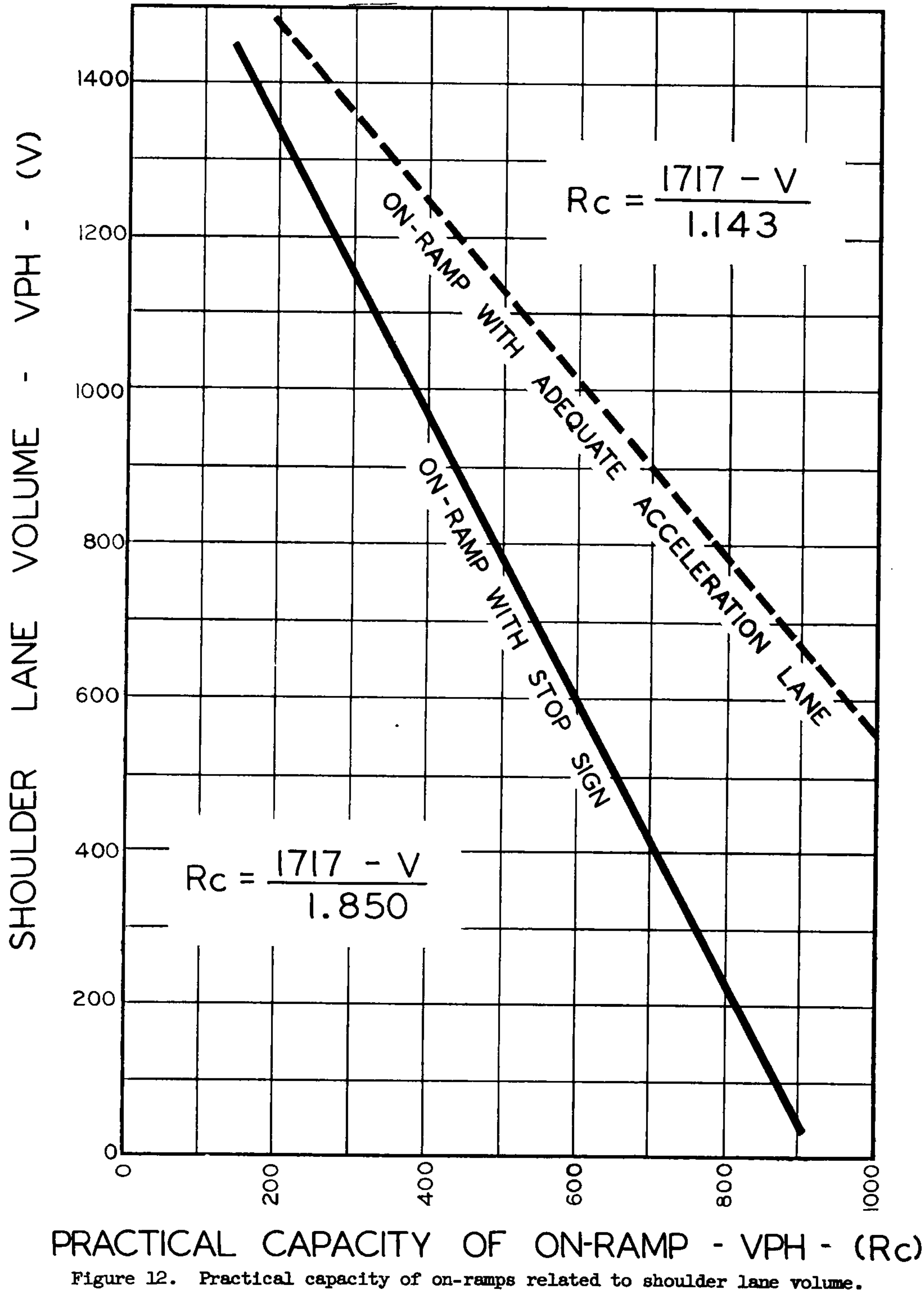


Figure 12. Practical capacity of on-ramps related to shoulder lane volume.

TABLE 3

No. of Exp. Lanes	Year	Expressway DHV Total	Expressway DHV Median	Expressway DHV Center	Expressway DHV Shoulder	Ramp DHV	Practical Ramp Capacity With Stop Sign	Practical Ramp Capacity With Acc. Lane
2	1960	2,286	1,606	-	680	561	560	910
2	1975	2,680	1,720	-	960	700	410	665
3	1975	2,680	800	1,100	700	700	540	870

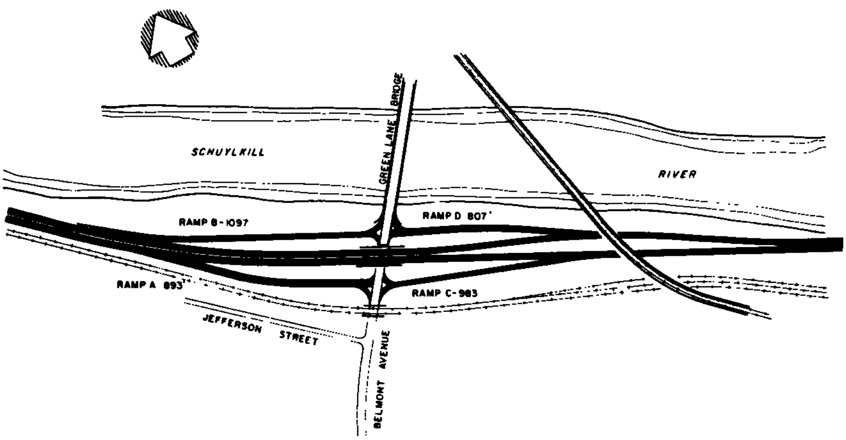


Figure 13. Manayunk Interchange.

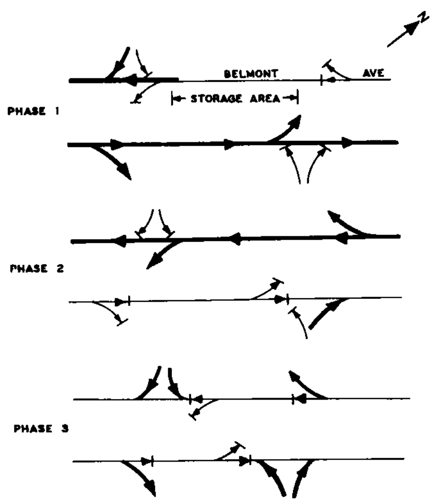


Figure 14. Phasing diagram of Manayunk Interchange.

To provide needed capacity including left turn storage, the widening of Belmont Avenue from four to six lanes was recommended.

To provide increased capacity on Green Lane Bridge, it was recommended that the bridge roadway be widened to five lanes, three of which would operate in the major direction. This may be done by eliminating the sidewalk on one side of the bridge and reducing the sidewalk on the other side to 4 or 5 ft.

Left turn channels from Ramps A and D would be widened sufficiently to permit turns from these ramps to be made in two lanes.

A traffic actuated progressive signal system would be installed to control traffic movements between the interchange ramps and Belmont Avenue, including Jefferson Street, and at Green Lane and Main Street in Manayunk.

It was recommended that the signals at

the ramp ends and Belmont Avenue be three-phase (Fig. 14). Three-phase operation will make possible the clearing of the limited storage area between the ramps before through movement begins and will increase the capacity for right turns from the ramps by permitting this movement during two of the three phases.

The studies and analyses summarized in this paper led to the following additional recommendations: (a) additional lanes, (b) adequate acceleration and deceleration lanes, (c) median barrier, (d) fencing to prevent pedestrian crossings, (e) lighting at least at all interchanges where such does not now exist, (f) a 55-mph "transition" speed limit between existing 60-mph and 50-mph sections, (g) minimum 10-ft shoulder width, and (h) restricting traffic by ramp closings when necessary.

REFERENCES

1. Bureau of Highway Traffic, Yale University, Technical Report 4, p. 69, Table XII.
2. Moskowitz, K., "Research on Operating Characteristics of Freeways." ITE Proc. (1956).

Accident and Operating Experience At Interchanges

R. L. FISHER, Assistant Supervising Engineer, New Jersey State Highway Department, Trenton

Geometric highway design can be evaluated only if the relative merits of the design features can be measured in some fashion. A yardstick is needed that will measure the various design features. Studies have been under way for the past two years to evaluate interchanges by three means—accident records, operation, and capacity.

Interchanges can be broken down into three basic elements: (a) the exit terminal, (b) the main portion of the ramp, and (c) the entrance terminal. This paper attempts to show how a variation in the design of these basic elements affects operation and safety.

● DURING the past two years New Jersey has undertaken a comprehensive study of safety, operation and capacity at 50 interchanges.

The first step was to review what had already been done by others. Experience and personal preference largely influences design. No published data could be found in which interchanges or their basic elements had been evaluated. For example, suppose it is decided that a certain ramp terminal design is satisfactory for an inner loop of a radius of 150 ft. What changes, if any, should be made if the radius of the loop was changed to 200 ft? What would be the effect on capacity, traffic behavior and safety?

Geometric highway design can be evaluated only if the relative merits of the design features can be measured in some fashion. In other words, a yardstick is needed that will measure the various design elements.

In this study an effort was made to evaluate interchanges by three means—accident records, operation, and capacity. The first two phases are almost completed, but much work remains to be done on the third item.

In such a study there is always a temptation to select the more complicated interchanges. However the most elaborate interchange is composed only of ramps which have three basic elements: (a) the exit terminal, (b) the main portion of the ramp, and (c) the entrance terminal. This discussion is primarily concerned with the entrance and exits at the main roadway.

In selecting interchanges for study, many of the older ones have been included that today might be considered substandard. This was done to give a range of values so that determination of desirable standards for safe and efficient operation could be made.

The accident records in general are for the 4-yr period from January 1, 1955 to January 1, 1959. A 4-yr period would tend to level out any unusual seasonal or temporary conditions. As, in addition, there is generally a 1- to 2-yr construction and a 1-yr design period, this means that most of the interchanges were designed at least 7 years ago. Many of them are 15 to 20 years old and one of them has been in operation for more than 30 years. At these older interchanges figures show a large number of accidents.

The number of accidents at each interchange varied from 1 to more than 130. In general, the same types of accidents occur at all interchanges; higher type design simply reduces the number of accidents. The older type interchanges with their larger accident experience were thus helpful in determining the real cause of accidents.

As the accident data came in for each interchange, the location of the accident was spotted on a plan and the accident reports and collision diagrams were studied.

The second phase of this study was then started. This consisted of field investigation of the roadway elements, including signs and pavement markings, in an effort to determine what design element was inadequate and what changes would improve the accident record and operating conditions.

All design must be based on assumed traffic behavior. After construction, this assumption can be checked by actual traffic performance. Near misses, variation in speed or lane changing may indicate some shortcoming in design.

The next phase of this study consisted of observation of traffic behavior during peak and off-peak periods.

This field evaluation of design elements calls for a trained observer who has an open mind and a thorough knowledge of both design and traffic behavior. It also requires a special ability to be able to analyze and determine what features are inadequate and what changes are required to provide for most efficient and safe operation.

An inadequate interchange does not have constant traffic friction or near accidents. For example, a highway with an average daily volume of 60,000, carries 22,000,000 vehicles a year. If only one driver out of one or two million gets in trouble, the interchange will have a poor accident record. This is one of the reasons why untrained observers, after a short inspection, do not recognize inadequate design features. It also explains (particularly for low volume roads) why poor design elements can remain in the standards for year after year.

It has been stated that a modern freeway is nothing but a number of interchanges connected with the proper number of lanes.

The AASHO Special Freeway Study and Analysis Committee recognized this importance of interchanges. Its members concluded that most of the traffic difficulties on freeways involve driver hesitancy, erratic driving, and resulting turbulent flows produced by improperly designed ramp terminals, short weaving sections, too close spacing of entrances and exits, abrupt decrease in number of lanes and inefficient marking and signing.

All of these items are connected with interchanges. Further inspection shows that only the ramp terminals are involved.

ENTRANCE RAMP TERMINALS

US 22 at Vaux Hall Road (Fig. 1) carries 66,000 vehicles a day. The ramp has an inside radius of 40 ft.

At first glance, it would appear that most of the accidents at the entrance ramps would be merging accidents and would occur on US 22. This is not true. At the two entrance ramps there were 29 accidents, seven caused by improper merging, whereas 22 occurred at the ramp terminal. These ramp accidents occur at interchanges which do not have adequate acceleration lanes.

At such interchanges vehicles have to slow down or stop before entering the main roadway. This places following drivers on the ramp in the difficult, if not impossible, position of (a) trying to watch the car in front as it merges with traffic on the main roadway, and (b) trying to look to the left at oncoming main roadway traffic.

Most drivers try to solve this problem by watching the car in front until it approaches the main roadway. Then they look to left at traffic on the main route. If at that time the car in front should suddenly stop because the driver at the last second found out that he had misjudged merging conditions, there is almost bound to be an accident.

This interchange also illustrates the importance of curb treatment at the ramp terminal. Unless the left-hand curb is placed so that vehicles are guided into a flat merging path, many of them will cut across one or more streams of traffic. At this location 55 percent of the entering traffic was observed to cut over into the center lanes instead of using the right-hand lane.

The ramp on the 8-lane section of US 1 (Fig. 2) has a 500-ft entrance radius and a

300-ft black top acceleration lane of the parallel type. (Figure 2 shows only the eastbound lanes of US 1.)

One might expect much better accident record and operating experience for this 500-ft radius ramp than for the ramp with the 40-ft entrance radius shown in Figure 1.

However, operating conditions have not been improved. Just before entering US 1, drivers are in the position of trying to watch the car in front and also the ones to the left and rear on the main roadway. There were 12 accidents on the ramp at this location due to rear-end collisions. There were not any merging accidents.

The ramp terminal in Figure 3 is at the junction of N.J. 3 and 17. Two design elements create most of the difficulty at this location: (1) four lanes merge into two in a very short distance, and (2) the left-hand curb line of the ramp does not guide traffic into a proper path of entry. This curb location encourages traffic on the ramp to cross to the center of N.J. 17.

There were 8 rear-end accidents on the ramp due to lack of an acceleration lane and 5 accidents due to improper merging.

Figure 4 shows the terminal of N.J. 62 at US 46. The ramp has a minimum radius of 500 ft with no acceleration lane. All accidents were caused by vehicles slowing or stopping before entering US 46 and being hit in the rear by a following ramp vehicle.

A special study was made of all entrances without acceleration lanes. The radii of the ramps varied from 40 to 1,000 ft.

It was found that increasing the radius from 40 ft all the way to 1,000 ft was of little benefit. The important factor is not the radius of the ramp but whether or not traffic slows down or stops before entering. As long as this occurs, there will be rear-end accidents on the ramps.

Figure 5 is a good example of the foregoing statement. The ramp radius is only 75 ft. The 825-ft acceleration lane eliminated most of the stops before entering and there was only one rear-end accident on the ramp.

The left-hand curb does not guide traffic into a flat merging path. There were two merging accidents.

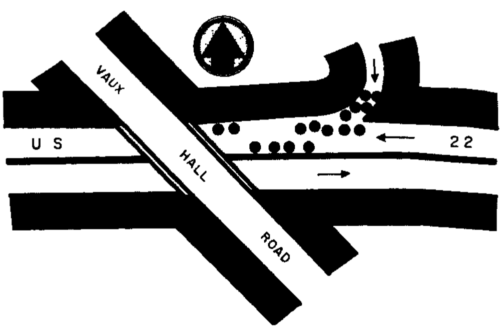


Figure 1. Ramp from Vaux Hall Road to US 22.

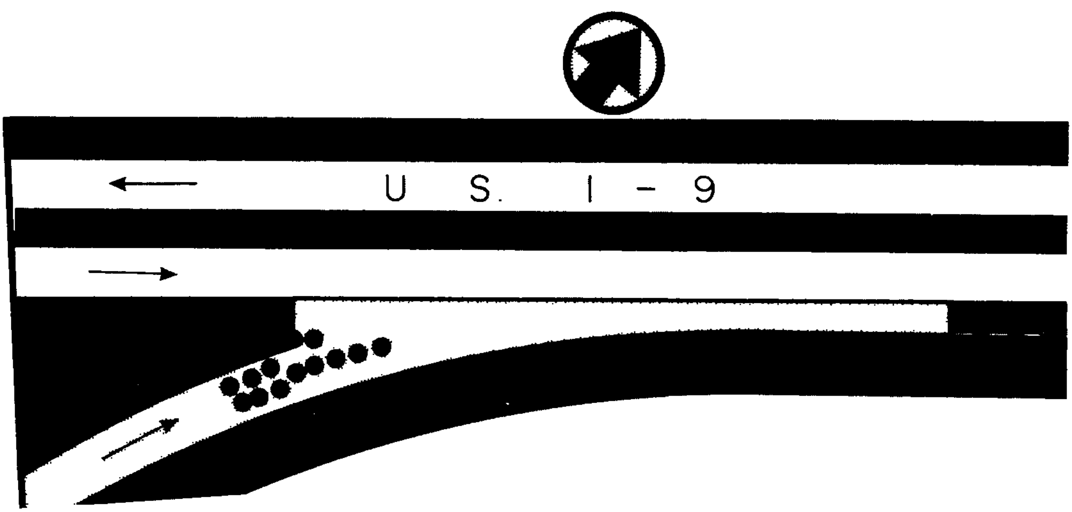


Figure 2. Ramp from Carnegie Avenue to US 1.

The ramp from US 1 to the Garden State Parkway (Fig. 6) shows a good treatment for the left-hand curb line. This connection also has an 800-ft acceleration lane. However, sight distance for entering traffic is restricted and there were three accidents at the entrance terminal.

Figure 7 shows the connection from Route 18 to the New Jersey Turnpike. This terminal design is standard for all interchanges on the turnpike. The left-hand curb line guides traffic into a flat merging path and the tapered acceleration lane is 1,200 ft long. There were no accidents at this location.

This type of design creates two operating conditions that tend to eliminate accidents at entrance ramps. First, drivers do not slow down or stop before entering the highway. This eliminates the rear-end accidents on the ramps that were shown in Figures 1 through 5.

Second, with the long tapered type of acceleration lane, drivers on the main roadway yield the right-of-way to entering ramp traffic. Observations show that this is accomplished either by a slight adjustment in speed or by moving over to the left-hand lanes. This type of driver behavior eliminates merging accidents and provides maximum capacity.

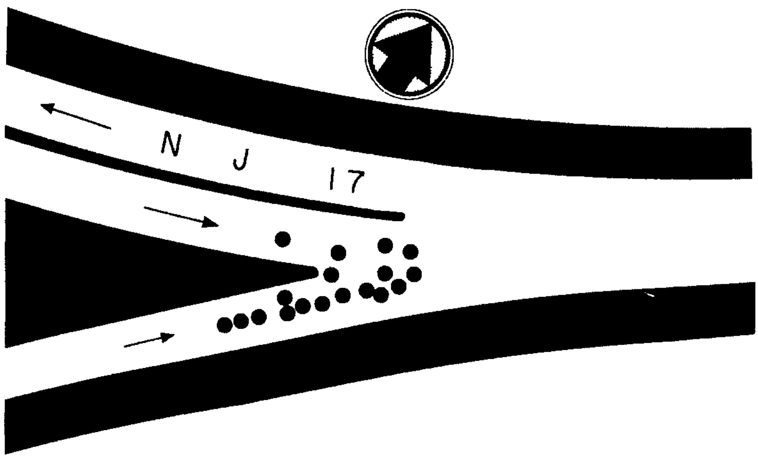


Figure 3. Ramp from N.J. 3 to N.J. 17.

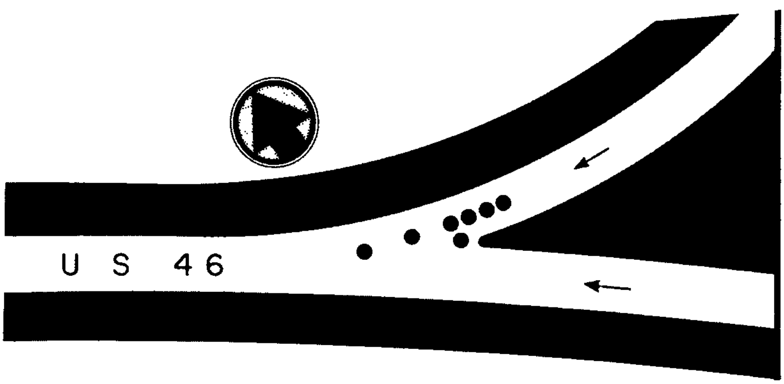


Figure 4. Ramp from Union Avenue to US 46.

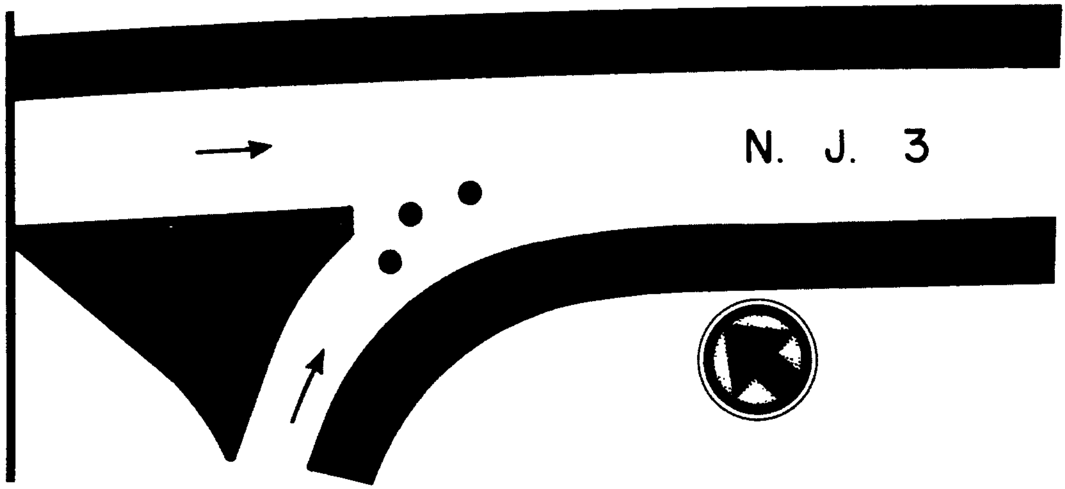


Figure 5. Ramp from Grove Street to N.J. 3.

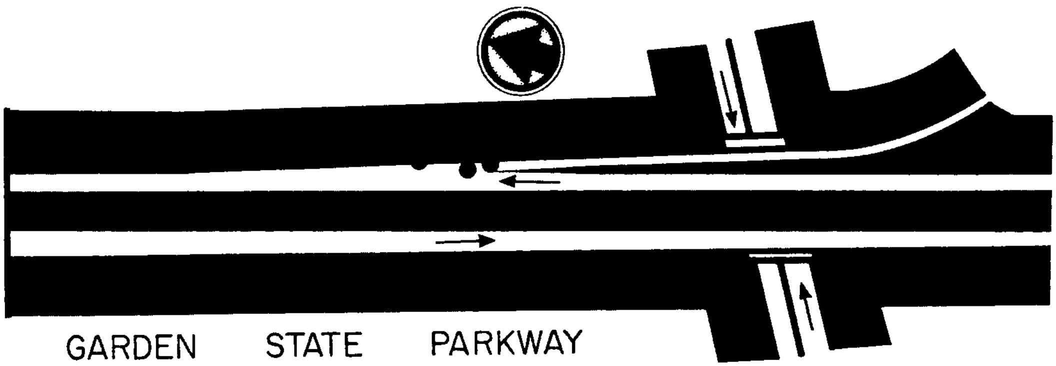


Figure 6. Ramp from US 1 to the Garden State Parkway.

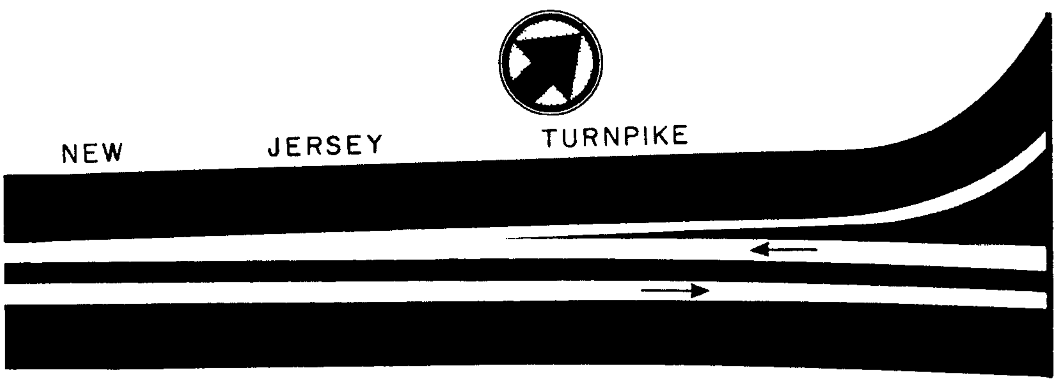


Figure 7. Interchange 9, New Brunswick.

EXIT RAMP TERMINALS

Route 22 at Vaux Hall Road (Fig. 8) is a four-lane land service road operating at capacity, carrying 66,000 vehicles a day.

This exit has a 300-ft radius with no deceleration lane. For these conditions the accidents have been about what would normally be expected.

Nine drivers slowed for the exit and were hit in the rear, 4 tried to turn from the wrong lane, 3 missed the exit and stopped, 3 hit the island nose, and 15 were due to rear ends caused by sudden stops in traffic.

The exit on Route 3 at Main Avenue (Fig. 9) has a 1,000-ft radius compounded to a 75-ft radius.

There were 8 accidents. One involved a vehicle that stopped after passing the exit; 3 were rear ends caused by drivers slowing for the turnout, one vehicle hit the island nose and one was an exit from the left-hand lane.

It is apparent that a higher type of exit would have prevented most of these accidents.

This exit on N.J. 3 at N.J. 17 (Fig. 10) has a 2,000-ft radius with no deceleration lane. This design permits a high speed exit only for the driver who follows the right-hand curb line.

Other vehicles, particularly trucks, tend to hide this type of exit and a driver may not see it in time to make a proper turnout.

The actual length of opening (parallel to the main roadway) through which he may turn with reasonable radius, is very short.

At a speed of 60 mph, a driver who delays 2 sec in making the turnout has the turning radius reduced from 2,000 to 300 ft. The greatly reduced radius decreases the possible speed of the turnout and creates traffic friction on the main roadway. This is reflected in the number of accidents.

The exit from the Garden State Parkway to US 1 (Fig. 11) has an 800-ft radius and an 800-ft deceleration lane. This length is about the absolute minimum. Shorter ones do not receive proper use.

One accident was caused by a driver slowing for the exit, the other one might have been due to inattentive driving.

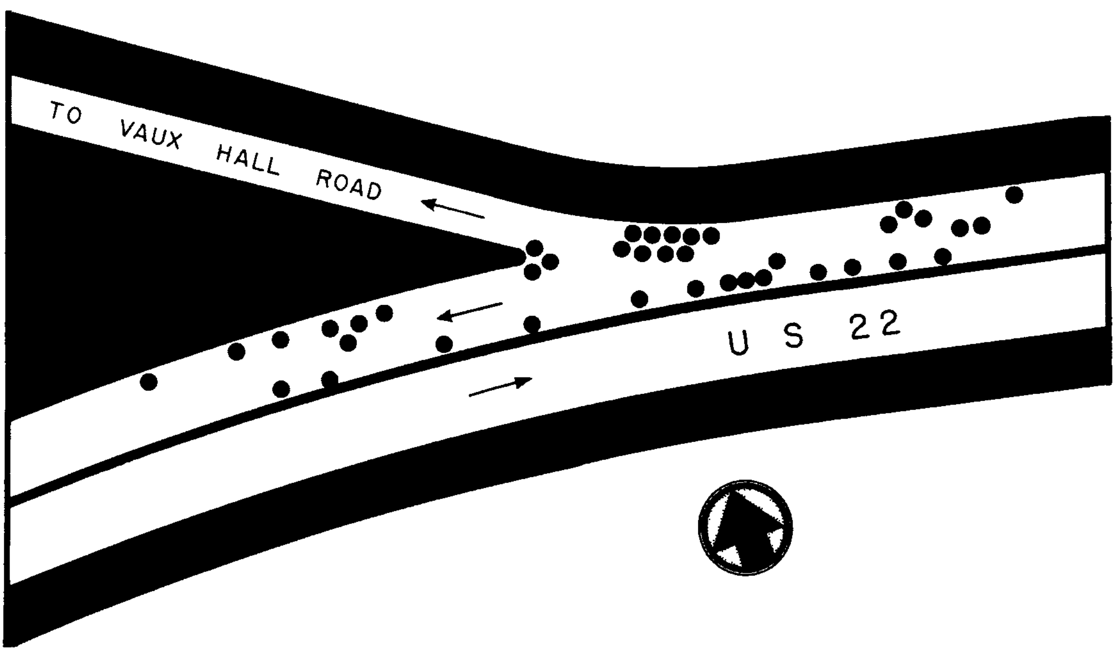


Figure 8. Ramp to Vaux Hall Road from US 22.

The exits on the New Jersey Turnpike (Fig. 12) have deceleration lanes of the parallel type—1,200 ft long.

The added deceleration lane permits traffic to thin out, giving the driver a better chance to see the exit in advance and provides the necessary time and distance to make a safe turnout.

Field observations show that nearly all drivers will use a deceleration lane of the parallel type if it is 1,200 ft long. When the length is decreased to less than 800 ft, some drivers will not use them and the accident rate is increased.

The abrupt beginning of the added lane also prevents drivers (particularly in bad weather) from drifting over into the exit by mistake.

There were no accidents at this location.

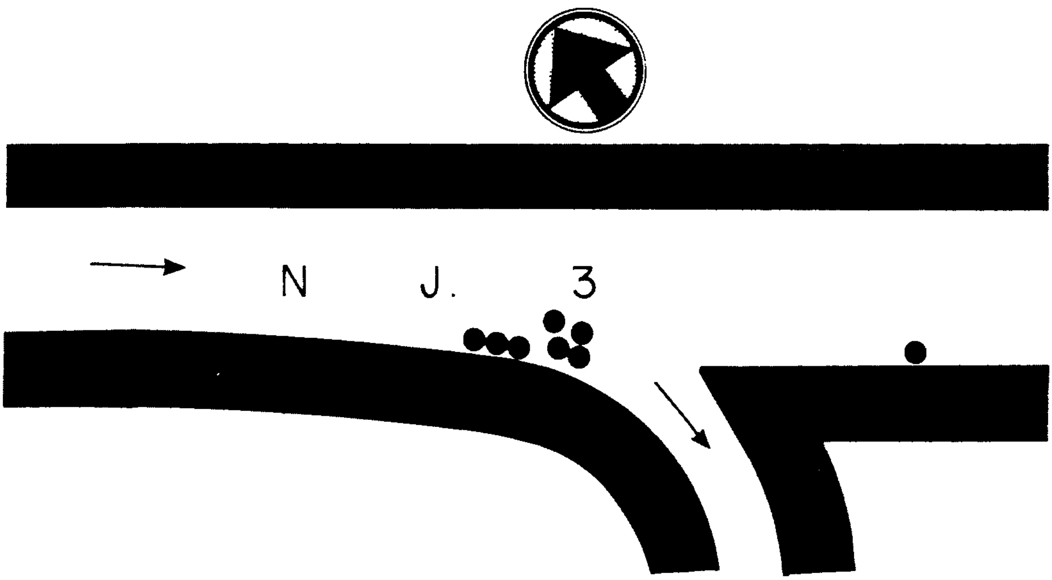


Figure 9. Ramp to Main Avenue from N.J. 3.

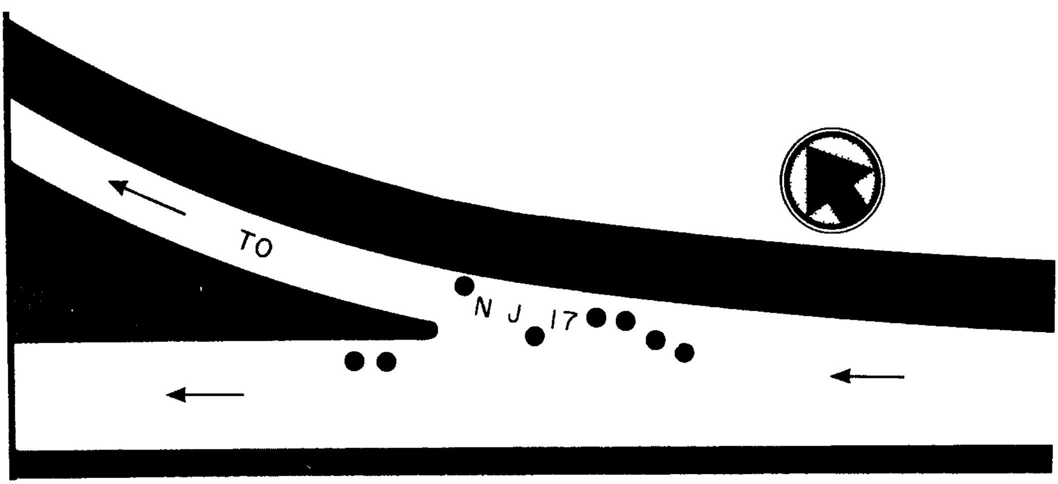


Figure 10. Ramp to N.J. 17 from N.J. 3.

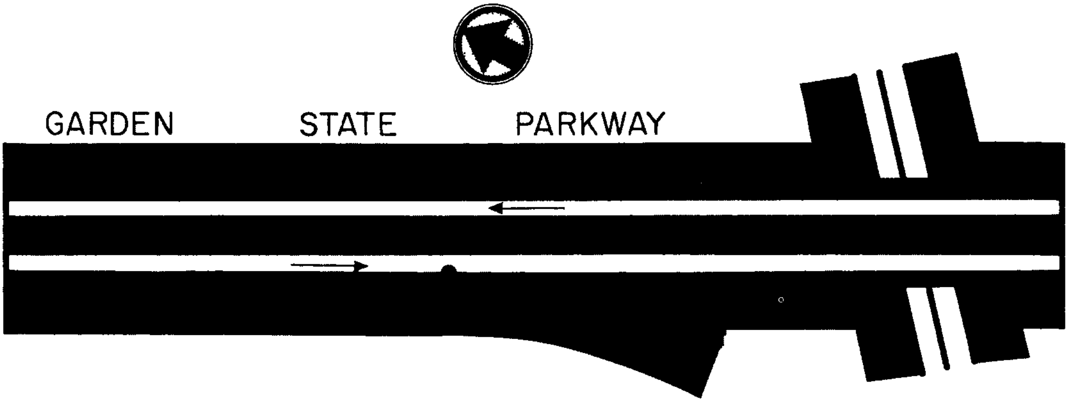


Figure 11. Ramp from the Garden State Parkway to US 1.

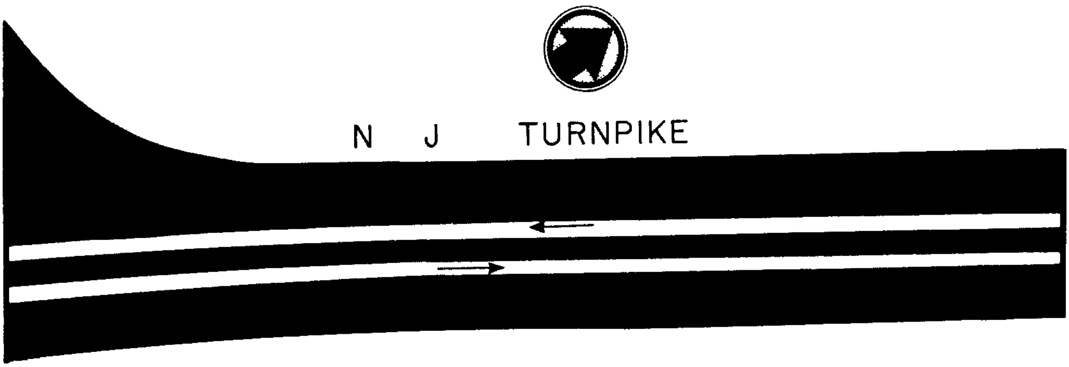


Figure 12. Interchange 9, New Brunswick.

Figure 13 shows the total number of accidents at each terminal compared with length of deceleration lane.

Even a short deceleration lane will help to reduce accidents. The greatest benefit is obtained when they are more than 800 ft in length. The greater length also eliminates much of the slowing down and traffic friction on the through lanes.

In Figure 14 the number of accidents at each terminal are compared with the length of acceleration lane.

Acceleration lanes less than about 900 ft in length have poor accident records. The reason for this is that many drivers will slow down or stop before entering the main roadway. Nearly all drivers will cross over into the main lanes in the first 200 to 400 ft.

When acceleration lanes are of the tapered type and over 900 ft long, an entirely different method of operation takes place. The full length of acceleration lane is used and drivers do not slow down before entering the main roadway. This eliminates the rear-end accidents on the ramp. Main roadway traffic also yields the right-of-way to entering traffic. This is done either by a slight adjustment in speed or by moving to a gap in the left-hand lanes.

The ability of ramp traffic to merge with main roadway traffic at about the operating speed of the highway not only reduces the number of accidents but also creates smooth operating conditions and increases the capacity.

Without adequate acceleration lanes, drivers on the ramp have to find a gap in

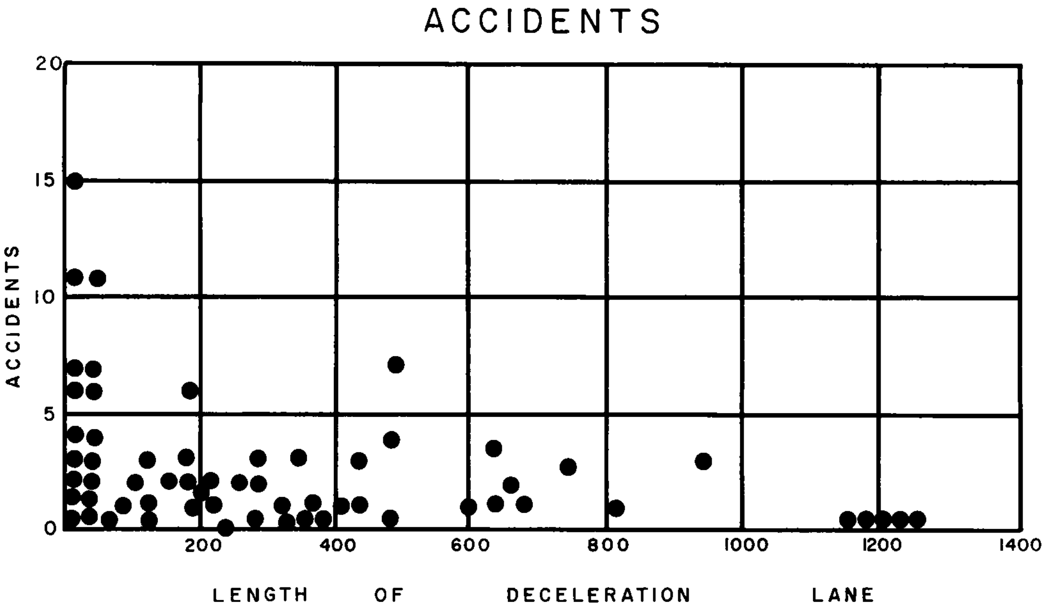


Figure 13.

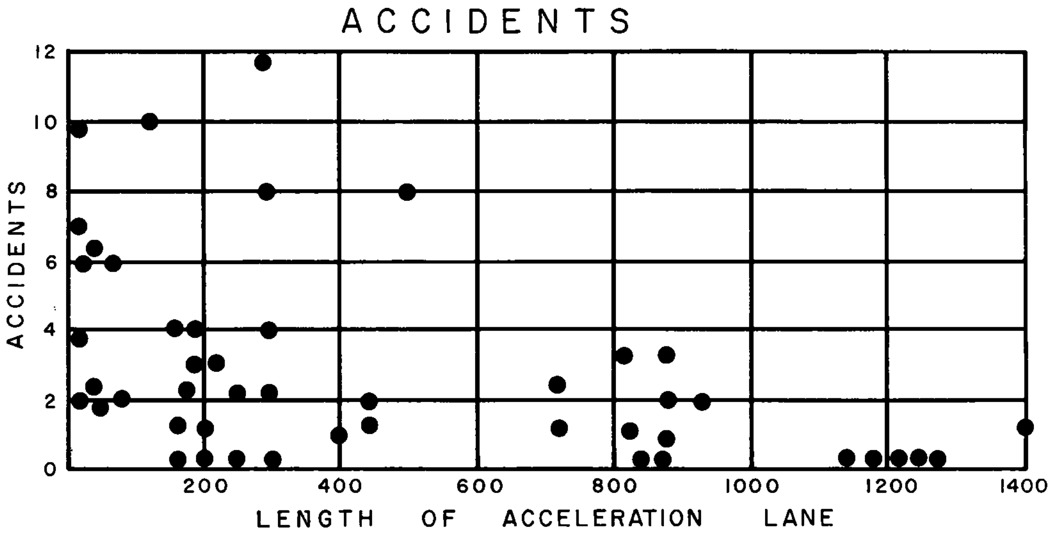


Figure 14.

traffic. Gaps sufficiently long for a vehicle to enter the highway and pick up speed to that of traffic on the main route are generally available only when volumes are low.

This means that when a vehicle enters from the ramp, it frequently forces the first vehicle to the rear on the main roadway to slow down. The second vehicle on the main roadway may have to brake harder and the third or fourth vehicle may have to stop. This reduces the capacity of the route and in many cases discourages use of the right-hand lane.

MAIN PORTION OF RAMP

In general there were very few accidents or any operating difficulties on any of the main portions of the ramps. Problems, if any, occurred at the ramp terminals.

On two-way ramps, there is a tendency for vehicles to swing wide at the beginning of the ramp, thus creating head-on conflicts. For this reason, it is believed that opposite direction traffic on two-way ramps should be separated by a curb.

For inner loops, a series of compound curves has been found to be effective in providing a means for traffic to safely reduce speed. New Jersey for some time has used radii of 750, 500, 250 and 170 ft.

There were not any accidents that could be ascribed to the curvature on inner loops that had radii of over 100 ft. The only ramp that had a poor record was one located past several underpasses, on a long downgrade and with a 70-ft ramp radius with no deceleration lane.

LEFT-HAND FACILITIES

Figure 15 shows part of an older type of interchange with a left-hand entrance ramp.

There were 14 accidents at this ramp. Six were merging accidents; four vehicles were cut off and forced into the guardrail and four were rear ends caused when drivers slowed before merging.

Note that in this design, the slow right-hand lane of the entrance ramp from Bloomfield Avenue has to merge with the left hand fast lane of US 46.

In this example four lanes have to merge into two in a short distance. A better design would be to drop only one lane and have the four lanes merge into three. The problem then becomes which lane to drop. There is considerable difference of opinion on this matter. One method would be to adjust the interchange design so that the three left-hand lanes could be carried straight through. The right-hand lane could be tapered out as a long acceleration lane. This would be a conventional treatment to which the motorist was accustomed and would probably be much less confusing than dropping one of the left-hand lanes. In any case the lane lining should be studied as part of the preliminary design and not something to be added as an afterthought when construction is completed.

Figures 16-21 show left-hand exits. These create different operating conditions than the conventional right-hand exit. Left-hand ramps are few in number compared with the usual right-hand and therefore can create an element of surprise.

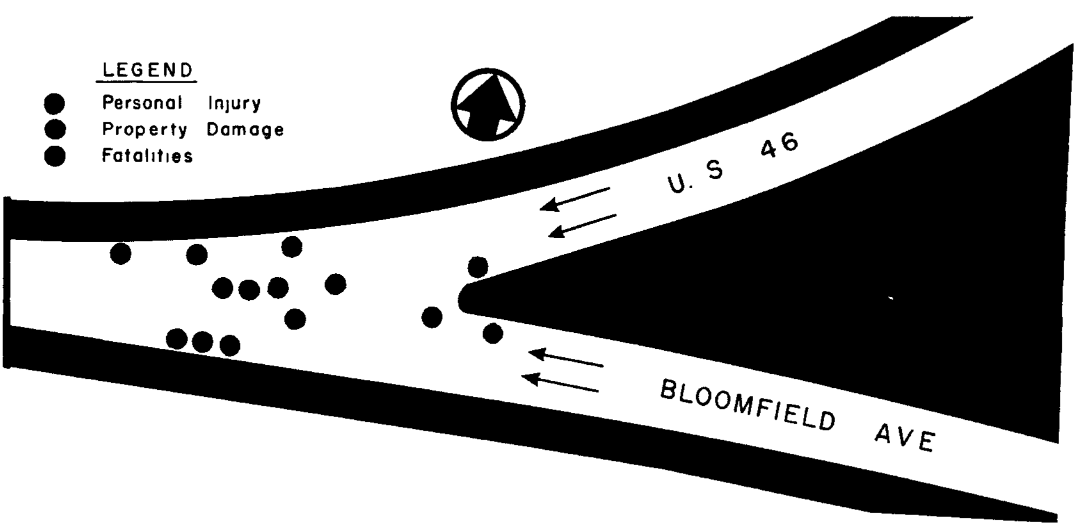


Figure 15. Ramp from Bloomfield Avenue to US 46.

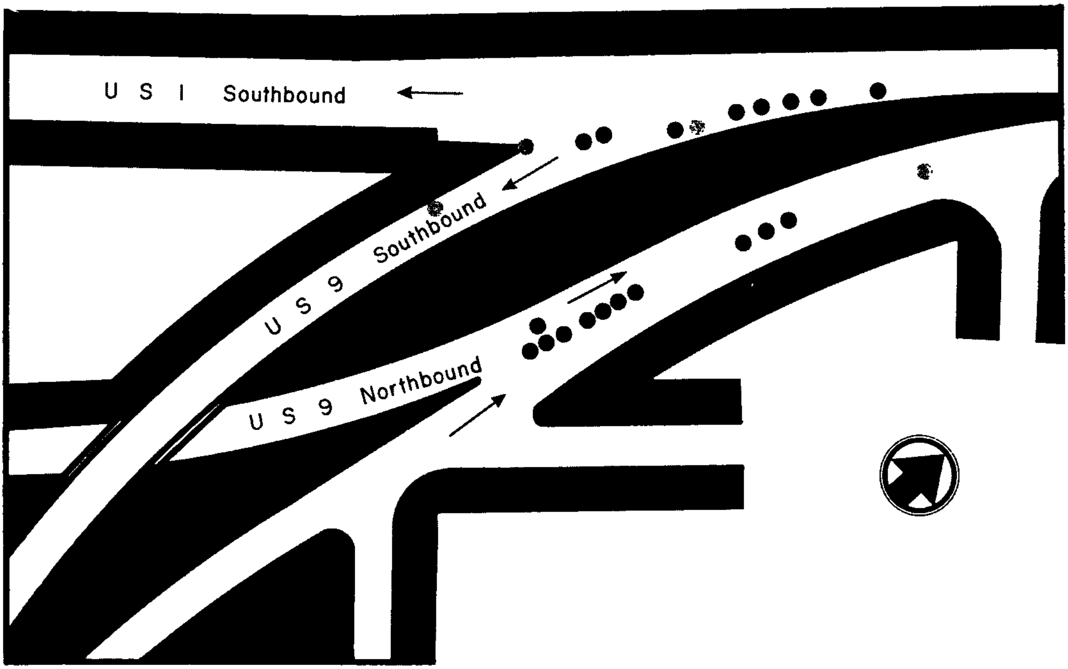


Figure 16. US 1 and 9 Interchange.

In spite of the best of signing and lane lining, some drivers will slow down or even stop before making an exit. This slowing down in the left-hand lane is not normally expected by a following driver in that lane.

A left-hand exit also forces slow-moving traffic in the right-hand lane, to cross over to left, in order to use the exit ramp.

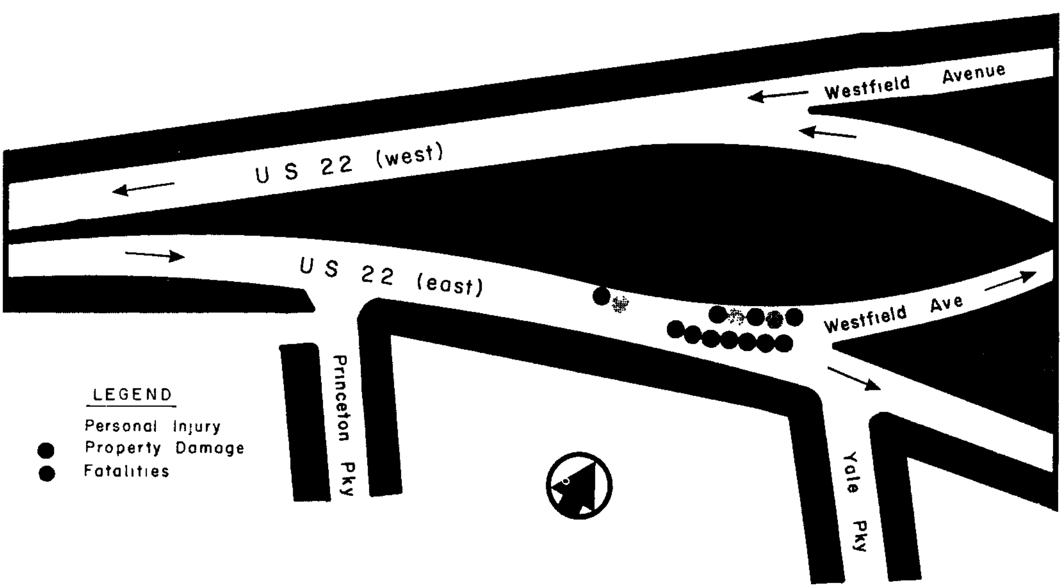


Figure 17.

On most highways the right-hand lane carries the lowest percentage of traffic and the left-hand lane the highest. Placing the exit on the left forces traffic into what may be an already over-crowded lane and makes thru traffic in that lane move to the right.

At the US 1 and 9 Interchange (Fig. 16) traffic makes an even split, 1,200 an hour

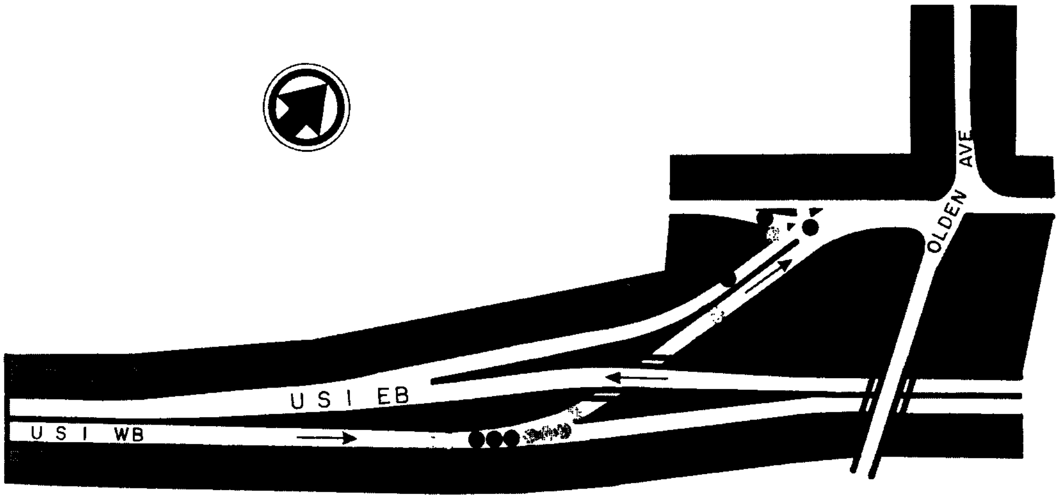


Figure 18.

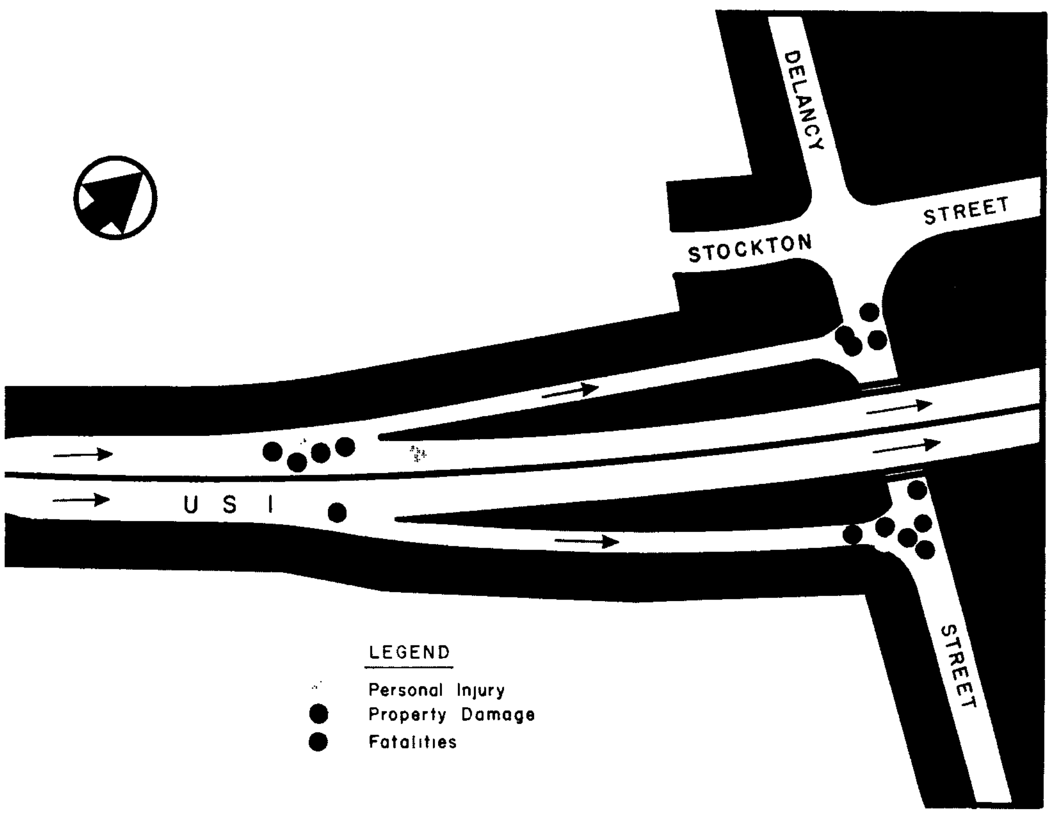


Figure 19.

using each roadway. Both ramps have a 950-ft radius. Particular attention has been directed to the signing at this location. There is a sign bridge with electric signs about 1,500 ft in advance of the exit; large reflectorized signs in the center island and a high mounted electric sign at the gore.

For southbound traffic, there were 12 accidents, two occurring during peak volumes. In seven cases vehicles were in the wrong lane. There were two accidents after midnight in which the drivers lost control of the vehicle and there was one sideswipe. This interchange is quite different from some of the others in that only one accident involved a driver slowing for the turn and being hit in the rear.

Northbound there were 12 accidents. Five were merging accidents, two were rear ends and five were due to lane changing.

US 22 carries 50,000 vehicles a day and Westfield Avenue about 7,000 a day. Eastbound US 22 is lane lined for two lanes and the left-hand exit has a radius of 700 ft (Fig. 17). There are two advance signs with 8-in. letters. These signs are located in the center island. There is also a sign at the gore.

For eastbound traffic, there were 14 accidents. Five were caused when vehicles slowed for the turn and were hit in the rear and seven were due to turns from the wrong lane. Five accidents occurred during peak volumes.

A complete interchange has 16 traffic movements. There are four offbound from the main roadway and four onbound. The same number occur at the local roadway.

This ratio of 16 traffic movements to 1 should be kept in mind in comparing accidents

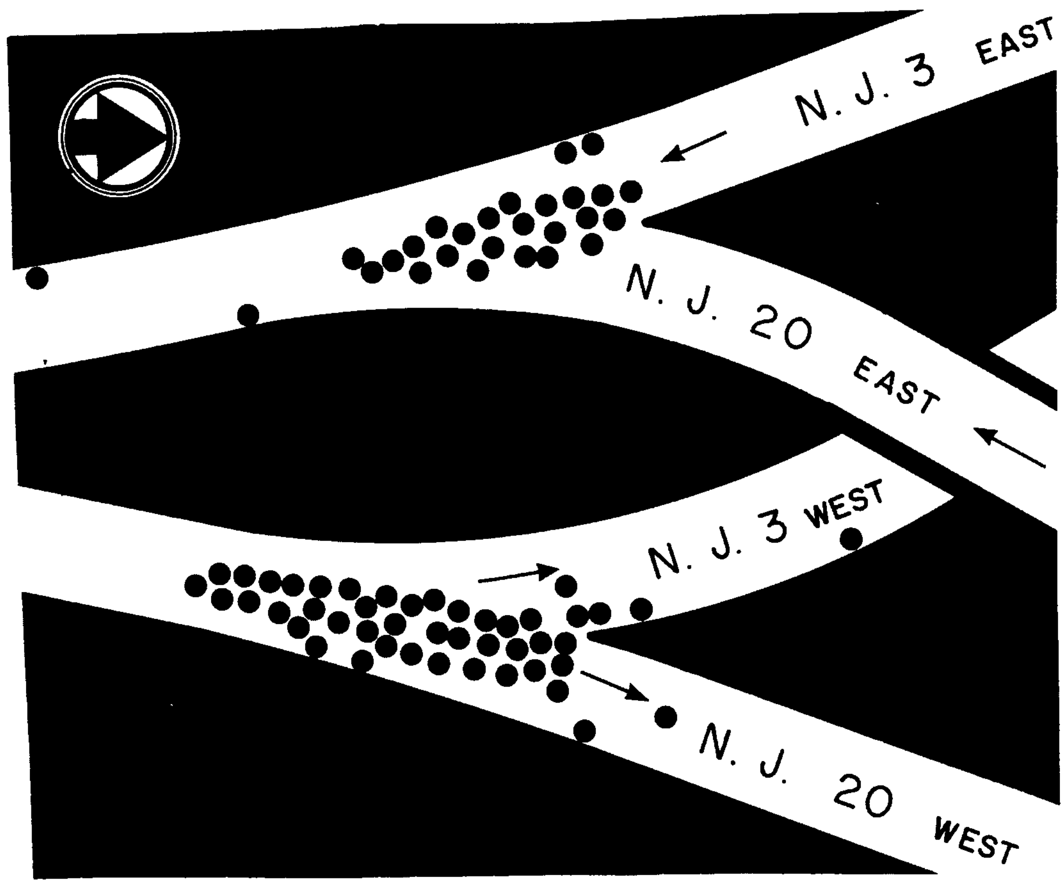


Figure 20.

at a conventional interchange with that of a single left-hand exit or entrance which has only one traffic movement.

At the US 1 Freeway and Olden Avenue (Fig. 18), traffic volumes are rather low. Four-hundred fifty vehicles an hour make the left turn and 950 continue on US 1.

There are overhead electric signs with 15-in. letters. The signs are located near the gore.

There were nine accidents at the left-hand exit. Six accidents were due to left turns from the wrong lane. One driver slowed for the exit and was hit in the rear. This accident resulted in seven injuries. At 1 a.m., when it was clear and dry, one motorcycle ran into another.

This interchange shown in Figure 19 has two exits of nearly identical design—one is a left-hand exit—the other is on the right.

US 1 in this area is of the dual-dual type. Each roadway has two lanes. The northbound outer roadway carries 2,800 vehicles an hour and the ramp 810 an hour. The inner roadway (which is restricted to passenger cars) carries 1,800 an hour and its ramp 560.

The right-hand exit had one accident—a driver made the mistake of slowing for the exit.

The left-hand exit had six accidents. Three were caused by turning from the wrong lane, one was due to lane changing and one driver missed the exit and tried to back up.

Figure 20 shows the intersection of N.J. 3 and 20. N.J. 3 westbound carries 3,900 vehicles an hour in advance of this interchange. Two-thousand six-hundred vehicles an hour use the left-hand exit.

Both the eastbound and westbound ramps have radii of 720 ft.

For westbound traffic, in addition to the route marker signs, there is a sign with 8-in. letters 600 ft in advance of the exit and located on the right; there is one on the left, 250 ft in advance, and one in the gore.

Eastbound there were 31 accidents. Seventeen were merging accidents and 10 were rear ends.

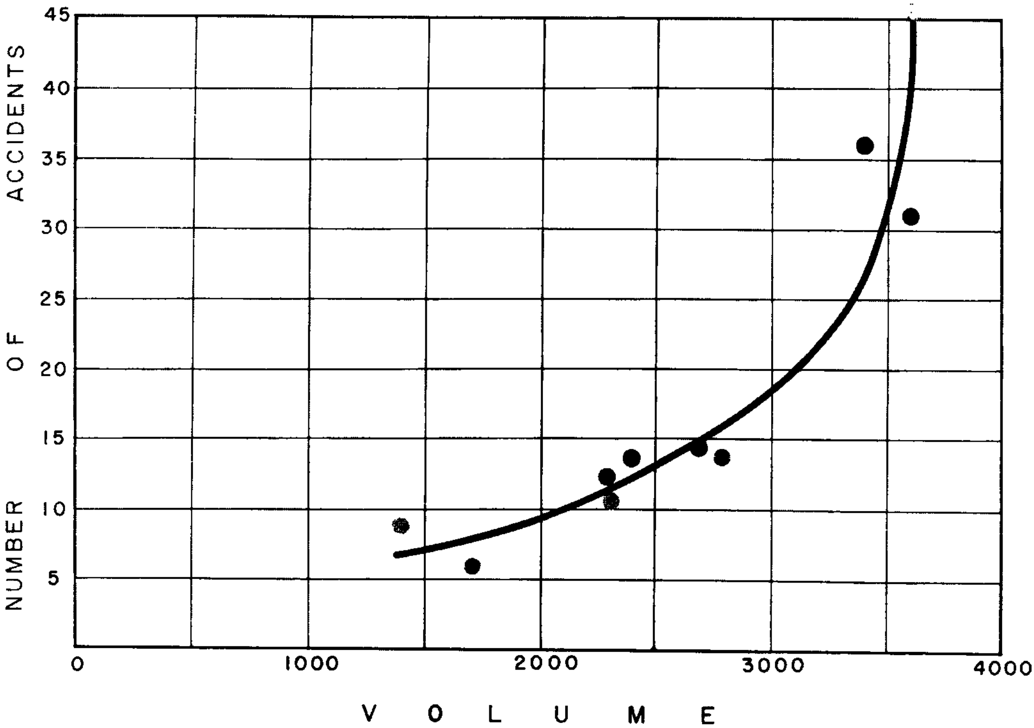


Figure 21.

Westbound there were 45 accidents. Seventeen were left turns from the wrong lane; 16 by traffic slowing for the turn and 5 were rear ends.

Figure 21 shows the number of accidents in comparison to traffic volumes for left-hand facilities. It is believed that the relationship is remarkable, especially when it is considered that some of the interchanges were for land service roads, others for parkway or freeway conditions.

CONCLUSIONS

From this study it is believed that the following conclusions can be reached:

1. There are not any accidents that can be ascribed to the curvature, on inner loops which have radii of over 100 ft. As ramps which have large radii take considerably more property and require more travel time, the use of one-lane inner loops with radii greater than 150 ft would appear to be questionable.
2. When the main roadway has concrete pavement, black top acceleration and deceleration lanes do not receive full use. Most drivers cut over into the main roadway within the first 200 to 400 ft.
3. Unless shoulders are provided, many drivers will not make proper use of acceleration and deceleration lanes.
4. Adequate length acceleration and deceleration lanes together with careful treatment of the terminals, and control of access, can practically eliminate accidents at interchanges.
5. All of the left-hand entrances and exits in this study had poor accident records.

Application of Computer Simulation Techniques To Interchange Design Problems

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A study of the operating characteristics of the driver-vehicle combination has yielded a general digital simulation model. This simulation model, which can duplicate traffic flow on a 17,000-ft section of a freeway including two on-ramps and two off-ramps, can be used to economically evaluate alternate design criteria.

The basis for this simulation model is the statistical analysis of actual traffic data collected at a number of freeway locations by the Bureau of Public Roads and The Texas Transportation Institute. Significant information on the complex problem of gap acceptance by a merging vehicle was developed through the analysis of the collected data. Study of the collected data resulted in the development of the relationship between (a) average velocity and total traffic volume, (b) vehicle distribution to lanes and total traffic, and (c) exiting vehicle distribution to lanes and distance from the off-ramp. Furthermore, information was developed on velocity distributions in each lane at various total traffic volumes. The detailed information which describes actual traffic flow and driver behavior on a section of a freeway is reported in the section on "Freeway Operating Characteristics."

The simulated vehicle in the model, following decision rules based on the actual collected traffic data, is allowed to maneuver through the section of freeway under study. The effects of changes in traffic volume, velocity, freeway configuration, etc., can then be evaluated by noting changes in the computer output of traverse time, waiting time on ramp, volume-velocity relationship, weaving complexity, etc.

The computer simulation thus creates a duplication of the real situation at a small fraction of the cost of studying the real system. Furthermore, the simulation allows: (1) the evaluation of various freeway configurations without the expense of their construction, and (2) the performance of controlled experiments impossible to perform with the actual traffic.

Controlled experiments with a section of freeway 3,400 ft long and containing one on- and off-ramp sequence were performed. The experiments were conducted with the on- and off-ramps located at two different positions on the freeway section. The results of the experiment showed no significant difference between the traffic flow under the two different configurations for the traffic volumes used. It is possible that at on-ramp volumes higher than those simulated, one configuration may prove more satisfactory than the others. Detailed reports on the measures of effectiveness measured are presented in the section on "Experimental Results."

●IN 1959 Midwest Research Institute developed a digital simulation model which could be used to duplicate a 1,700-ft section of a freeway. Based on the success of this model in duplicating actual freeway conditions, a study was undertaken to expand the simulation model so that a complete interchange could be duplicated. The new simulation model was to be used in developing efficient design criteria for highways. Work on this project was undertaken in three phases.

Phase 1

One phase of the project was devoted to the study and analysis of traffic data gathered at locations in Detroit and Houston. The purpose of this portion of the study was to develop input data, describing the freeway operating characteristics of the driver-vehicle combination, which could be used with the simulation model. The results of this portion of the study are reported in the section on "Freeway Operating Characteristics."

Phase 2

A second phase of the project consisted of developing the logic and program for the digital simulation model. Emphasis was placed on making the model as simple as possible without sacrificing any reality. This phase of the project, which was carried out concurrently with Phase 1, is discussed in the section on "Simulation Model."

Phase 3

The third phase of the project consisted of testing the simulation model developed during the previous phases. Two controlled experiments were carried out, comparing the effects on traffic flow of two different freeway configurations. The two experiments are outlined in the section on "Design of Experiments." The section on "Experimental Results" is devoted to a comparison of the traffic characteristics of the two configurations. The section on "Conclusions and Recommendations" is devoted to some conclusions and recommendations based on the experimental results.

FREEWAY OPERATING CHARACTERISTICS

Data were gathered at two intersections in Detroit by the Bureau of Public Roads, and at intersections in Houston by the Texas Transportation Institute.

Detroit

The Detroit data on 25,000 punched cards were obtained from the Bureau of Public Roads. Each card represented a vehicle crossing one of the recording mechanisms at each site (Fig. 1). Each card contained information on the velocity, lateral placement, type and maneuver of the vehicle. The time of day to the nearest 0.0001 of an hour that the vehicle passed the recorder was also available. The format of these data made it suitable for obtaining information on (a) velocity distributions, (b) volume-velocity relations, and (c) vehicle distribution to lanes.

Velocity Distributions.—The distributions of through vehicle velocities at Sites 7 and 8 are shown in Figure 2. The distributions are slightly skewed in appearance. Some parameters of the distributions are given in Table 1. Analyses were also made of the velocity distribution at various traffic volumes. Figure 3 shows the velocity distributions at 6-min volumes of 91, 128, and 150 vehicles. These distributions are for Lane 1 at Site 8. Figure 4 shows additional velocity distribution for Lanes 1 and 2.

Volume-Velocity Relations.—Regression analyses were made for each lane of the two Detroit locations in order to determine the effect on average velocity of an increase in traffic volume. The input data consisted of the 6-min average velocity and the 6-min vehicle volumes. Graphs of velocity versus volume in Lane 2 for Sites 7 and 8 are shown in Figure 5.

A linear relationship between velocity and volume of the type shown in Equation 1 was assumed.

$$Y = a + b X \tag{1}$$

in which

Y = 6-min average velocity,

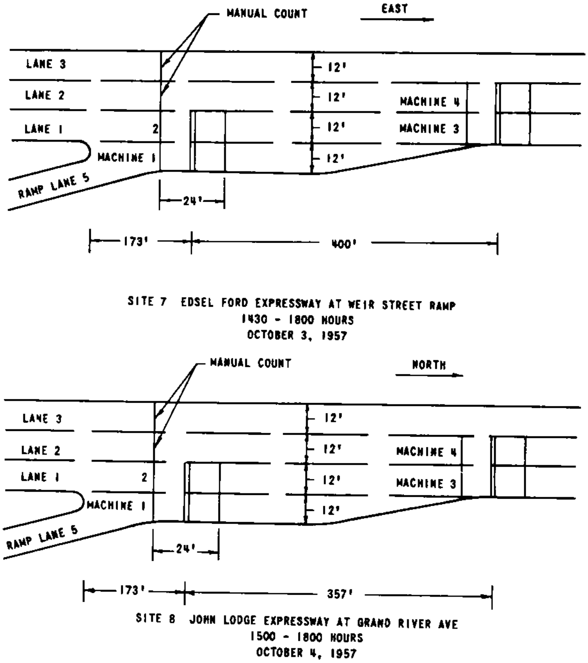


Figure 1. Detroit Study Sites, October 1957.

X = 6-min volume,
a, b = constants

The results of the analyses are shown in Table 2.

Vehicle Distribution to Lanes.—The distribution of vehicles to the three lanes was obtained at Sites 7 and 8 (four recording locations in all). The relationship between the distribution and the total freeway volume was determined. A plot of the data (Fig. 6) indicated that a linear relationship existed. The relationships were described by equations of the following type:

$$P_i = a + b\alpha \qquad (2)$$

in which

P_i = percent of total volume in i^{th} lane
α = total freeway volume per 6-min increments

The results of the linear regression at all locations are given in Table 3. As expected the entering vehicles prevent significant correlation between through volumes and Lane 1 utilization in the proximity of the on-ramp.

Houston

The Houston data were obtained by the analysis of motion pictures taken at on- and off-ramp sites. These data were suitable for obtaining information about: (1) gap acceptance of merging vehicles, (2) path of exiting vehicles, and (3) distribution to lanes of exiting vehicles.

Gap Acceptance of Merging Vehicles.—It is difficult to determine the size of a gap accepted by a merging vehicle for the following reasons:

1. It is impossible to determine what the size of the gap was at the moment the merging driver made his decision.
2. It is impossible to determine how the merging driver's choice was affected by the size of other gaps being presented to him either in advance or following the gap accepted.

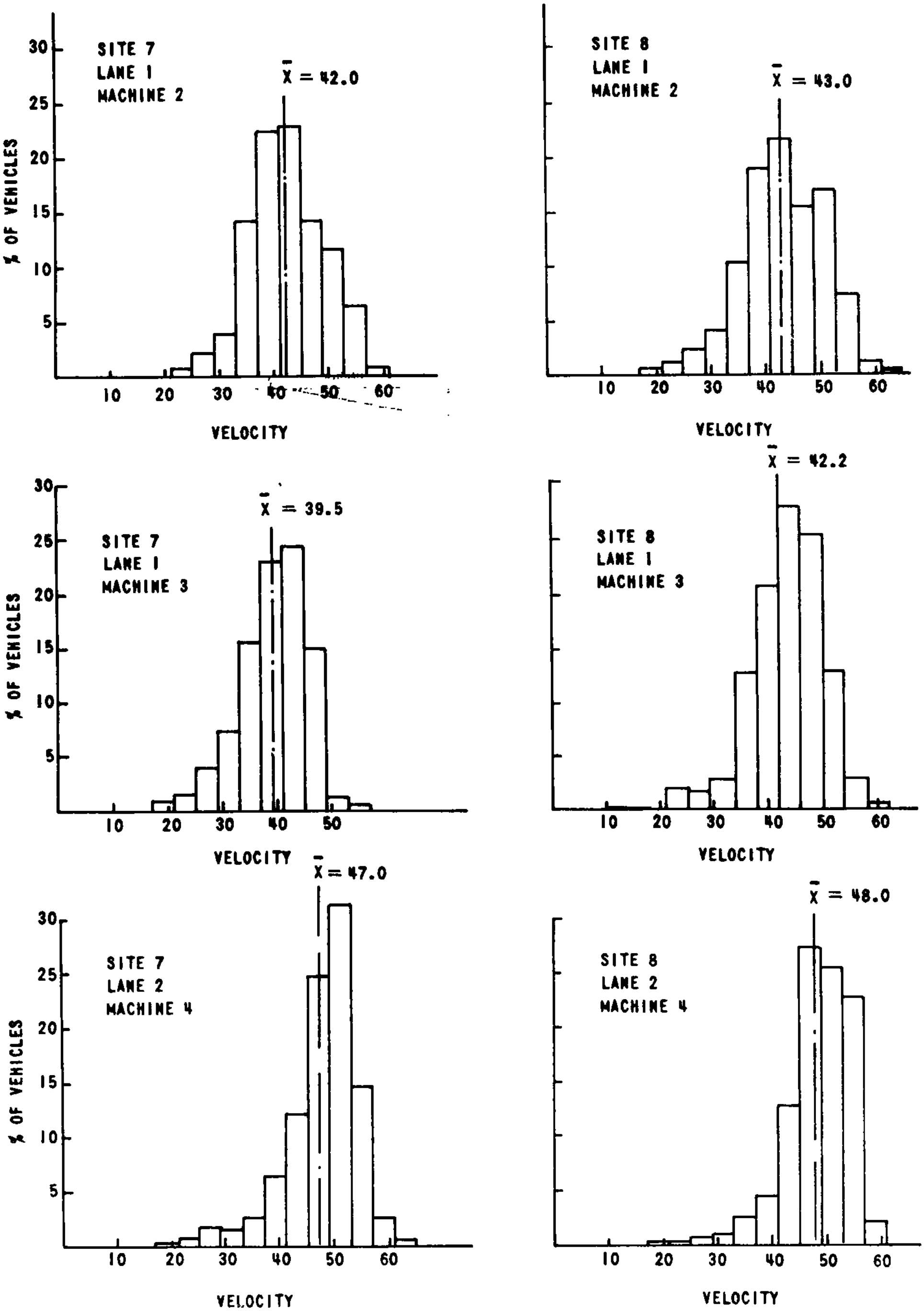


Figure 2. Through vehicle velocity distributions at Sites 7 and 8 in Detroit.

TABLE 1

THE MEAN, STANDARD DEVIATION, AND COEFFICIENT OF VARIATION OF THE VELOCITY DISTRIBUTIONS

Site	Lane	Machine	Mean Velocity ($\bar{x}$)	Standard Deviation of Velocity (s)	Coefficient of Variation ($s/\bar{x}$)
7	1	2	42.05	6.85	0.16
	1	3	39.54	6.83	0.18
	2	4	46.98	6.76	0.14
8	1	2	42.99	7.83	0.18
	1	3	42.16	6.90	0.16
	2	4	47.94	6.52	0.14

3. It is difficult to determine the rate at which the gap size is changing as it approaches the merging vehicle.

However, for the purpose of obtaining some meaningful data on gap acceptance, the following procedure for analyzing the film was adopted:

1. The film was projected and the merging, lead, and trailing vehicles were observed.
2. The film was projected back and the time the lead vehicle (No. 1) passed a given point 0 (Fig. 7) was observed.
3. The film was projected forward and the time that the trailing vehicle (No. 3) passed point 0 was observed (Fig. 7).
4. The time difference between Steps 2 and 3 was calculated. This value was used as a measure of the gap accepted.

A similar procedure was used to determine the size of gaps rejected. Gap acceptance data on passenger vehicles only were obtained. Multiple vehicle entries were not considered. The data were subdivided into merging behavior of stopped vehicles and merging behavior of moving vehicles.

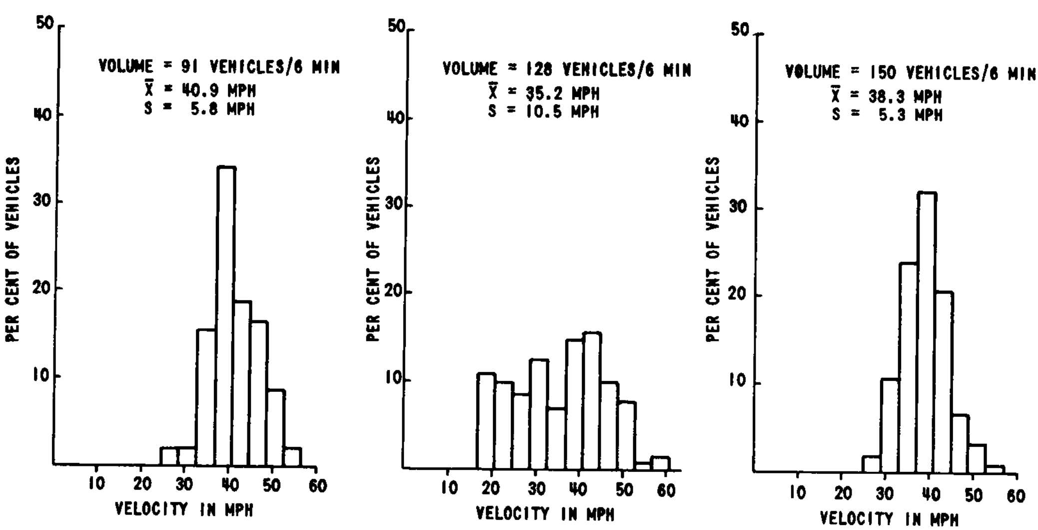


Figure 3. Velocity distributions for Lane 1, Site 8, at given vehicle volumes.

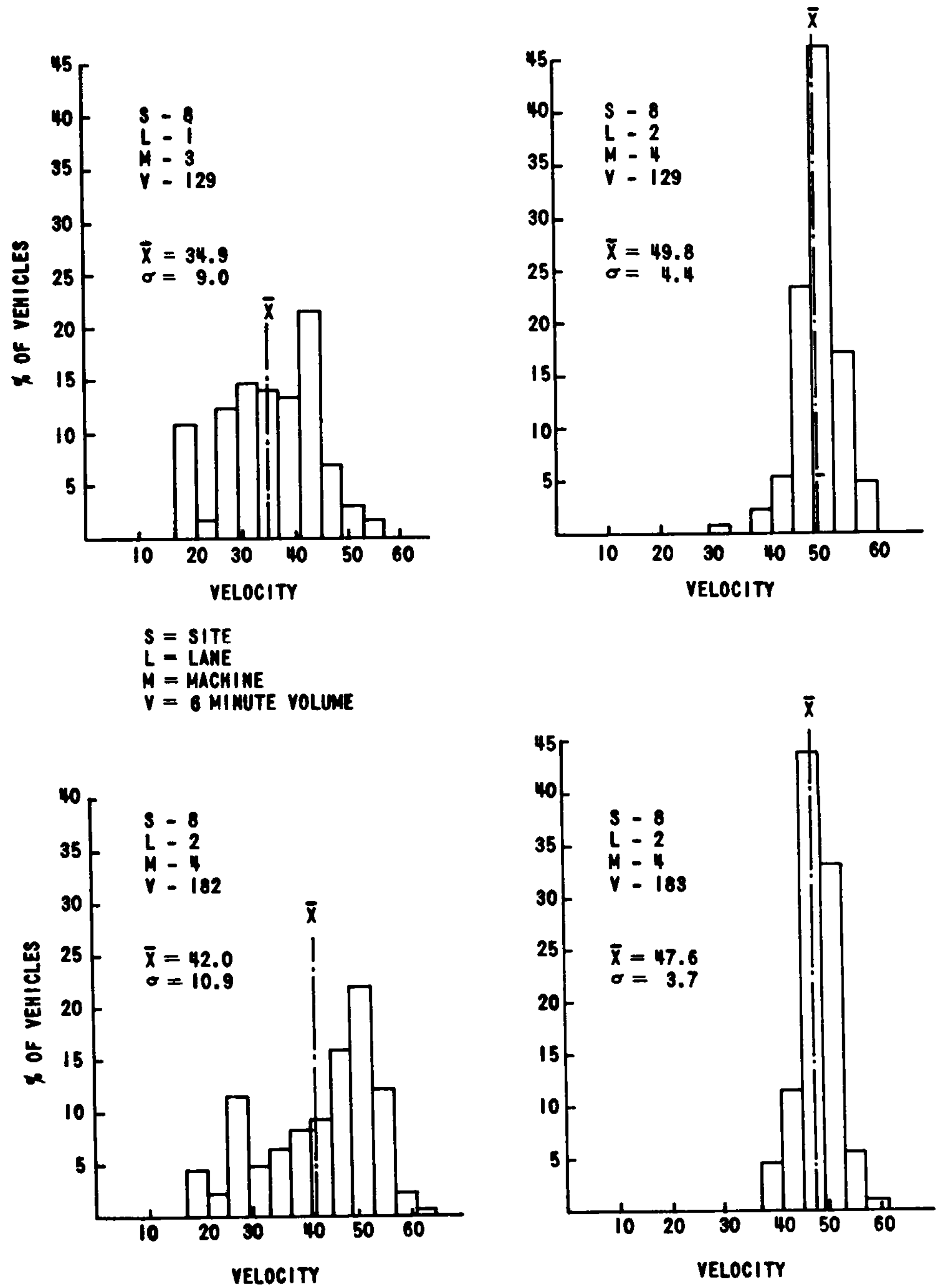
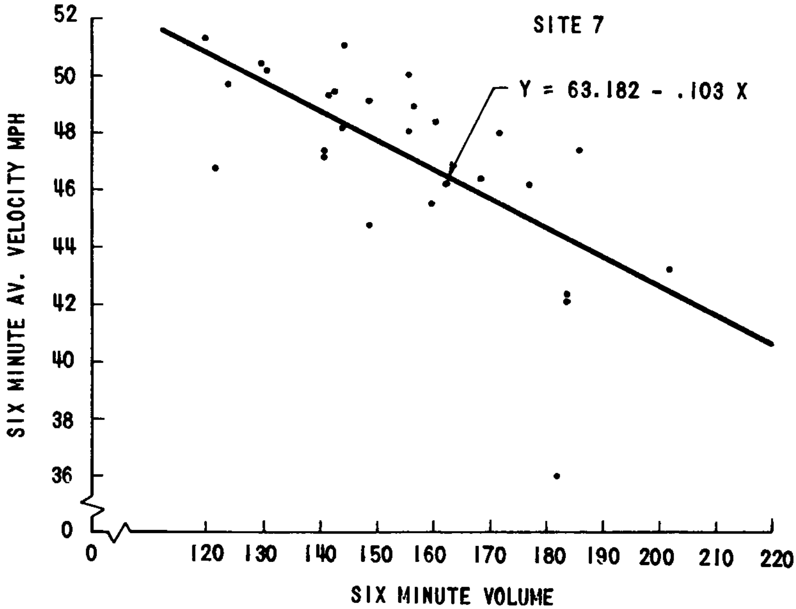


Figure 4. Velocity distributions for Lanes 1 and 2 at Site 8 for given vehicle volumes.



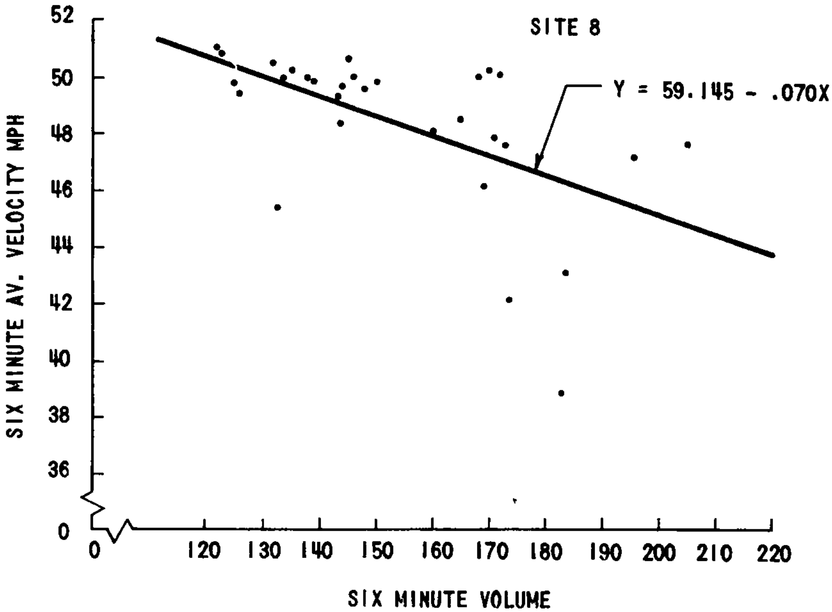


Figure 5. Relationship between average velocity and volume in Lane 2.

An analysis was made of film taken during the peak morning and afternoon traffic periods. The results (Table 4) show that vehicles merging after they have stopped require a longer time gap than those which merge without stopping.

Path of Exiting Vehicle.—The deceleration lane at the Cullens Interchange on the Gulf Freeway in Houston was studied to determine the path of the exiting vehicles. The lane was divided into four sectors (Fig. 8) and the number of cars moving into the deceleration lane in each sector was recorded (Table 5). Observations were carried out for 1 hr and 20 min during the peak morning volume and for 50 min during the peak evening hours.

TABLE 2

LINEAR EQUATIONS FOR VELOCITY-VOLUME RELATIONSHIP

Site	Lane	Machine	Equation	Correlation Coefficient (r)	Test of Significance at 0.05 Level
7	1	2	Y = 47.630 - 0.015X	-0.758	Significant
7	1	3	Y = 50.548 - 0.097X	-0.622	Significant
7	2	4	Y = 63.182 - 0.103X	-0.683	Significant
8	1	2	Y = 51.062 - 0.085X	-0.580	Significant
8	1	3	Y = 53.703 - 0.077X	-0.738	Significant
8	2	4	Y = 59.145 - 0.070X	-0.585	Significant

Distribution to Lanes of Exiting Vehicles. —To determine the effect of the flow pattern of vehicles in an interchange area on the efficiency of a freeway, the Cullens Interchange on the Gulf Freeway in Houston was analyzed. During the period of analysis, the total volume of through traffic varied from 2,100 to 3,900 vehicles per hour. The exiting vehicles made up, on the average, 6.6 percent of the through traffic. Analysis showed that for the range of volumes under investigation, there was no relationship between the total volume and the distribution to lanes of exiting vehicles. On the average, vehicles were distributed as shown in Figure 9.

SIMULATION MODEL

The purpose of the simulation model is to duplicate in a digital computer the real life vehicle-driver combination passing through a section of a freeway in order to permit controlled experiments. The purpose of these experiments is to obtain knowledge of the relationships among the various factors that control the smooth and efficient flow of traffic. On the basis of this knowledge of traffic behavior improved highway design criteria can be developed. Digital simulation is an effective tool in highway traffic study as it permits the performance of experiments which cannot be performed, or which are too costly to perform, on a real freeway.

Input Factors for Simulation Model

In order to obtain a true duplication of actual traffic behavior on the freeway the simulation model should contain all factors which influence traffic behavior. In this model the following factors are considered:

1. The volume of entering and exiting vehicles.
2. The distribution of vehicles to lanes.
3. The velocity distributions of vehicles.
4. The gap acceptance distribution of merging and weaving vehicles.
5. The acceleration of entering vehicles.
6. The deceleration of exiting vehicles.
7. The distribution to lanes of exiting vehicles.

In addition, all vehicles are allowed to shift lanes in order to pass slower moving vehicles in front of them.

The Study Area

The study area is set up in a 4 x N matrix, ($N \leq 999$), (Fig. 10). The four rows represent: (1) the three through Lanes 1, 2, and 3, and (2) the ramp, acceleration and deceleration Lane 5. Each of the N blocks represents a 17-ft section of freeway, the approximate length of an automobile. For the simulation runs on the computer any

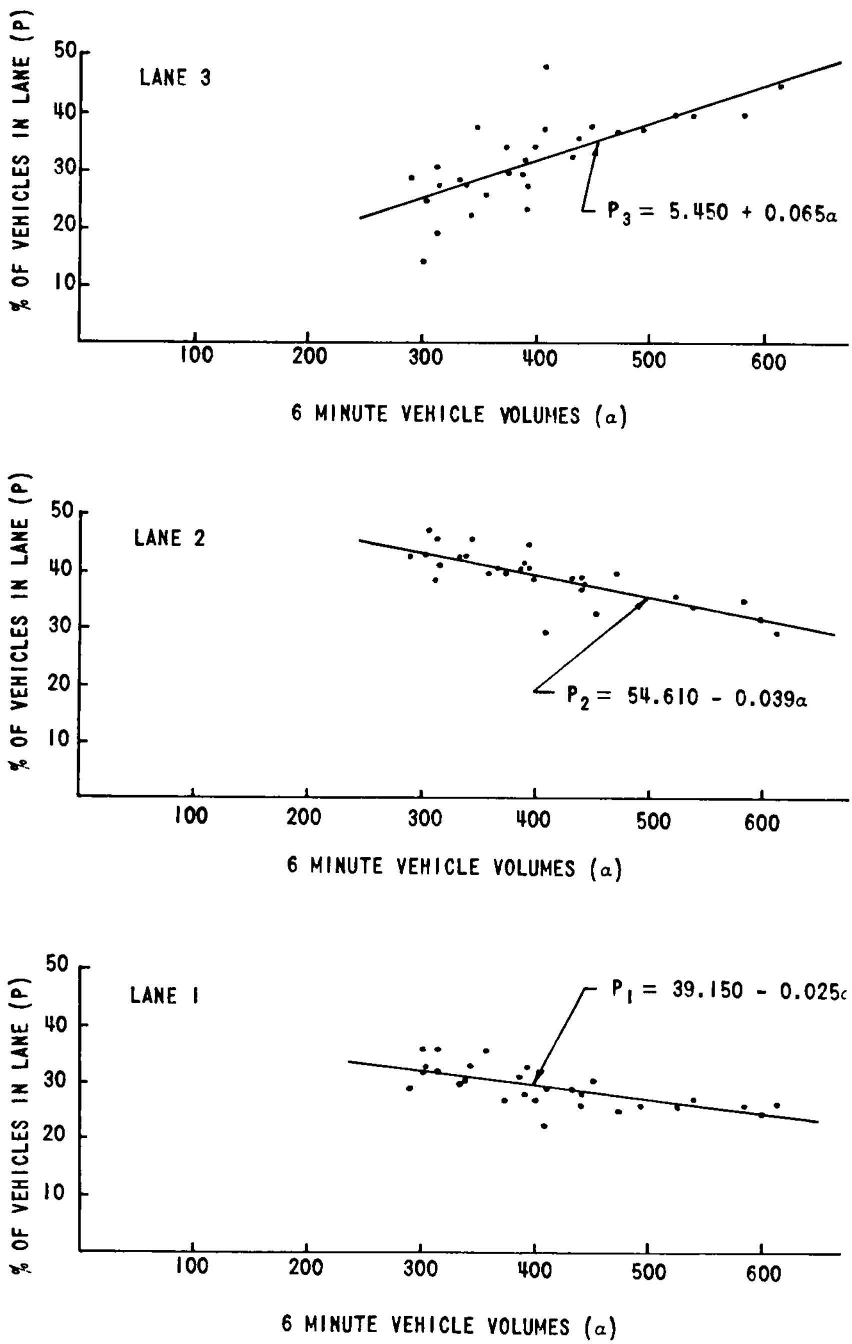


Figure 6. Distribution of traffic volumes to the three lanes.

TABLE 3

LINEAR EQUATIONS FOR DISTRIBUTION TO LANES—VOLUME RELATIONSHIPS

Site	Distance From Nose of Ramp (ft)	Lane	Equation	Correlation Coefficient (r)	Test of Significance at 0.05 Level
7	179	1	$P_1 = 27.78 - 0.010\ \alpha$	-0.337	Not significant
		2	$P_2 = 53.48 - 0.031\ \alpha$	-0.615	Significant
		3	$P_3 = 19.16 + 0.041\ \alpha$	+0.670	Significant
	579	1	$P_1 = 39.15 - 0.025\ \alpha$	-0.619	Significant
		2	$P_2 = 54.61 - 0.039\ \alpha$	-0.734	Significant
		3	$P_3 = 5.45 + 0.065\ \alpha$	+0.735	Significant
8	179	1	$P_1 = 26.12 - 0.006\ \alpha$	-0.202	Not significant
		2	$P_2 = 47.01 - 0.014\ \alpha$	-0.350	Significant
		3	$P_3 = 24.61 + 0.025\ \alpha$	+0.519	Significant
	536	1	$P_1 = 39.25 - 0.015\ \alpha$	-0.532	Significant
		2	$P_2 = 45.16 - 0.026\ \alpha$	-0.751	Significant
		3	$P_3 = 15.59 + 0.041\ \alpha$	+0.745	Significant

TABLE 4

GAP ACCEPTANCE OF MERGING VEHICLES

	Moving Merging Vehicles			Stopped Merging Vehicles		
Size of Gap Presented (sec)	No. of Veh Accepting Gap	No. of Veh Rejecting Gap	% of Total No. of Veh Accepting Gap	No. of Veh Accepting Gap	No. of Veh Rejecting Gap	% of Total No. of Veh Accepting Gap
0 -1.00	1	1	12.5	0	31	0.00
1.01-2.00	58	44	56.8	8	270	2.8
2.01-3.00	111	33	77.0	21	100	17.4
3.01-4.00	120	6	95.1	32	57	36.0
4.01-5.00	102	3	97.0	24	13	64.9
5.01-6.00	101	2	98.0	19	1	95.0
6.01-7.00	70	0	100.0	16	0	100.0
7.01-8.00	46	0	100.0	5	1	-
8.01-9.00	36	1	98.0	-	-	-
9.01-10.00	40	0	100.0	-	-	-

value of N (≤ 999) can be utilized. Locations of interchanges are designated as follows:

A = ramp input location
B = nose of on-ramp
C = end of acceleration lane
D = beginning of deceleration lane
E = nose of off-ramp
F = off-ramp output location

The program is very flexible and permits the on- and off-ramps to be located at any point on the section of freeway under investigation.

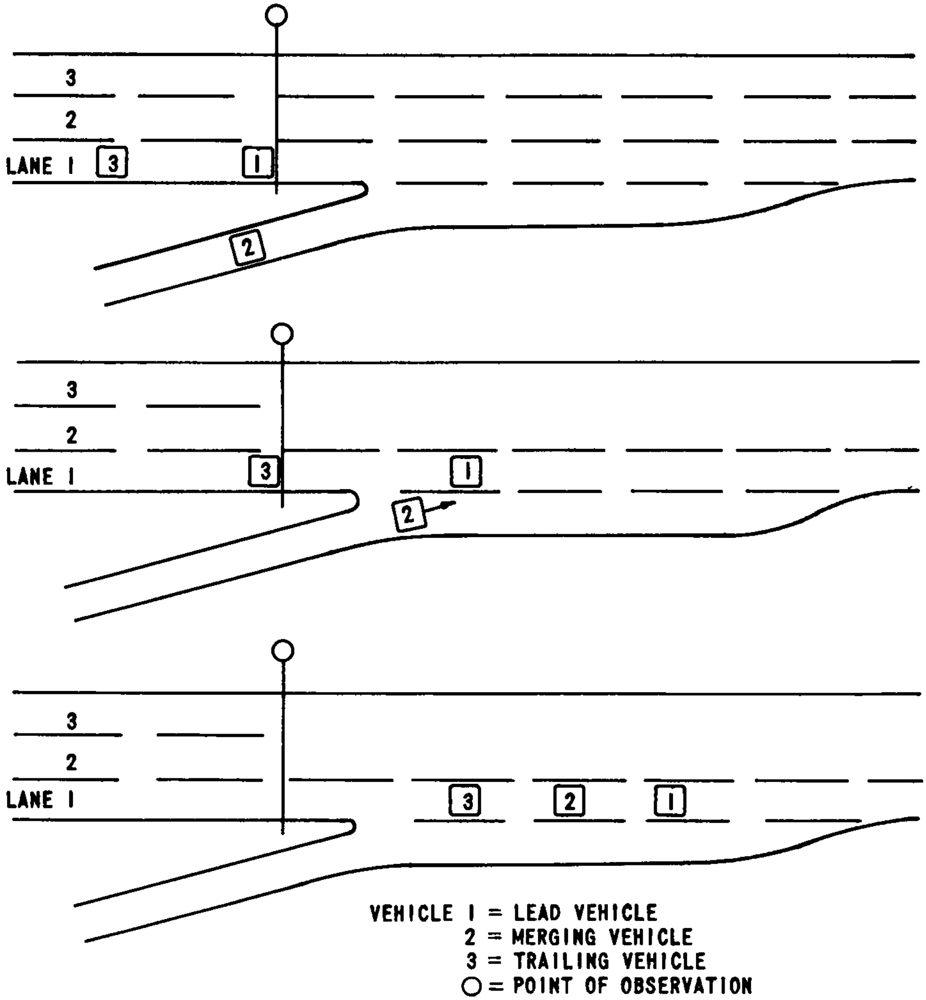


Figure 7. Observation point for the collection of gap acceptance data.

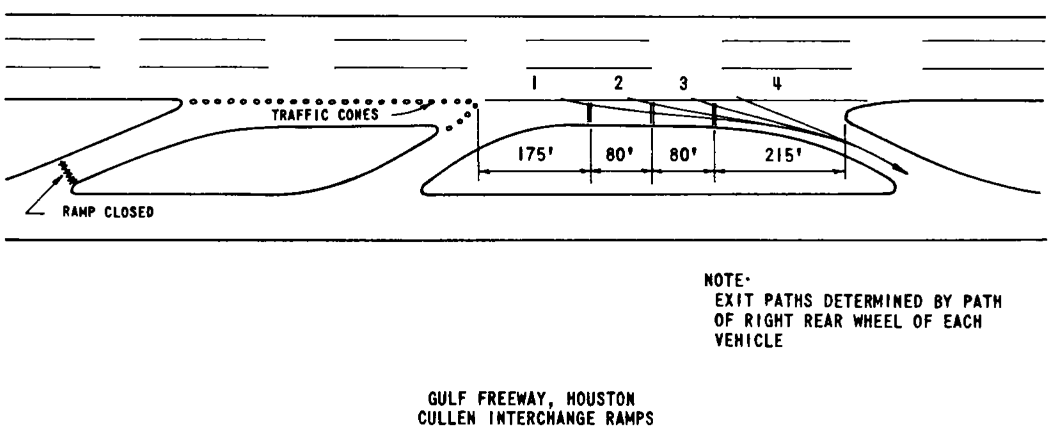


Figure 8. Site for the collection of data on the path of an exiting vehicle.

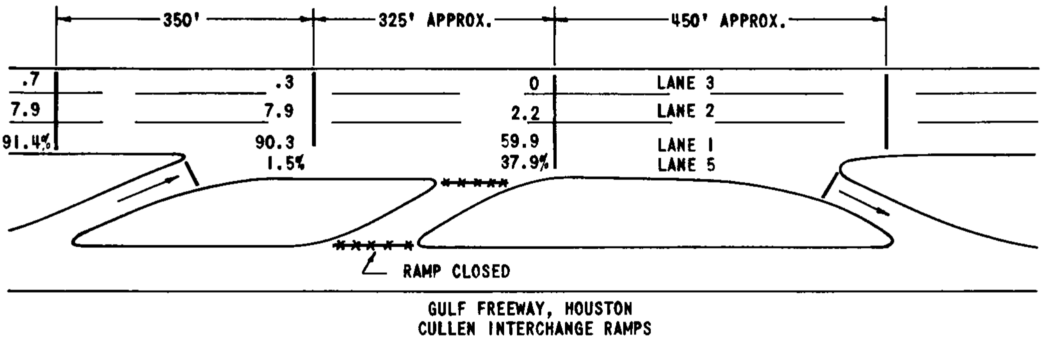


Figure 9. The distribution of exiting vehicles to lanes at various distances from the off-ramp.

TABLE 5
PATH OF EXITING VEHICLES

Exit Sector	Morning Peak Hours No. of Veh	Morning Peak Hours % of Total Exiting in Sector	Evening Peak Hours No. of Veh	Evening Peak Hours % of Total Exiting in Sector
1	112	32.6	44	22.2
2	148	43.0	86	43.4
3	74	21.5	66	33.3
4	10	2.9	2	1.1

Simulation Procedure

Basically, the procedure consists of simulating the arrivals of cars into the section of highway under consideration and then controlling the action of the vehicle by a set of decision processes. During each second of real time each vehicle in the matrix is examined. The vehicle is allowed to advance, weave, merge, accelerate, decelerate, or exit according to logical rules describing the behavior of actual vehicle-driver

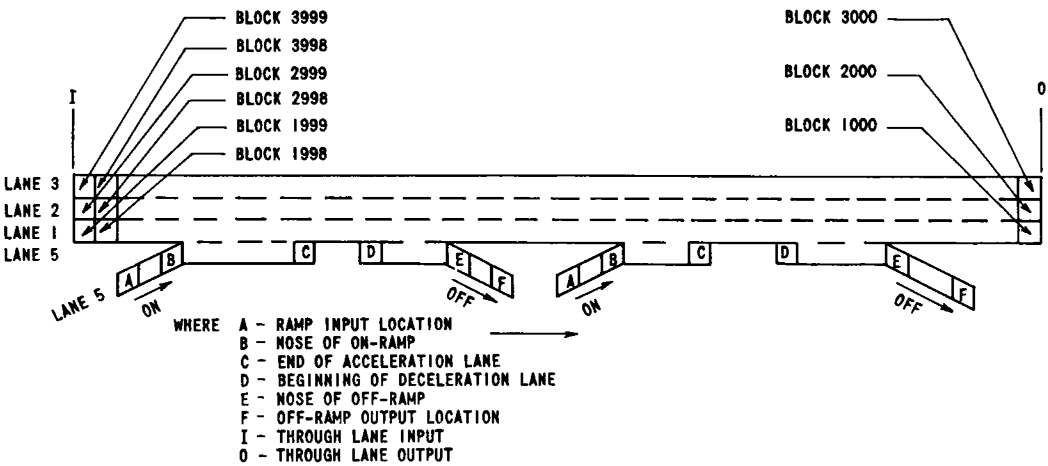


Figure 10. Study area matrix for computer simulation of an interchange area.

combinations. Just prior to examining all vehicles at each second, each of the input locations is evaluated. Inspection starts at vehicles closest to the end of the section of highway under examination and proceeds to vehicles in the input location.

An over-all description of the logic involved in processing a vehicle through the system is given in the flow diagram (Fig. 11). Detailed descriptions can be obtained in MRI Report No. 2349P.

Simulation Output

The present model was programed so that the following information can be obtained about each simulation run.

1. The volume of vehicles traversing the system in each lane.
2. The volume of vehicles entering the freeway through each on-ramp.
3. The volume of vehicles exiting the system at each off-ramp.
4. The number of vehicles which stop on the acceleration lane.
5. The length of the queue on the acceleration lane.
6. The number of vehicles that desire to exit but cannot.
7. The distribution of through-vehicle traverse times.
8. The distribution of ramp-vehicle traverse times.
9. The average vehicle velocity in each lane.
10. The number of weaves from:
    (a) Lane 1 to 2;
    (b) Lane 2 to 1;
    (c) Lane 2 to 3; and
    (d) Lane 3 to 2.

DESIGN OF EXPERIMENTS

Various types of controlled experiments can be carried out using the simulation model described in the section on "Simulation Model." For example, experiments on the effects on traffic flow of (a) various on-ramp vehicle volumes; (b) various accera-

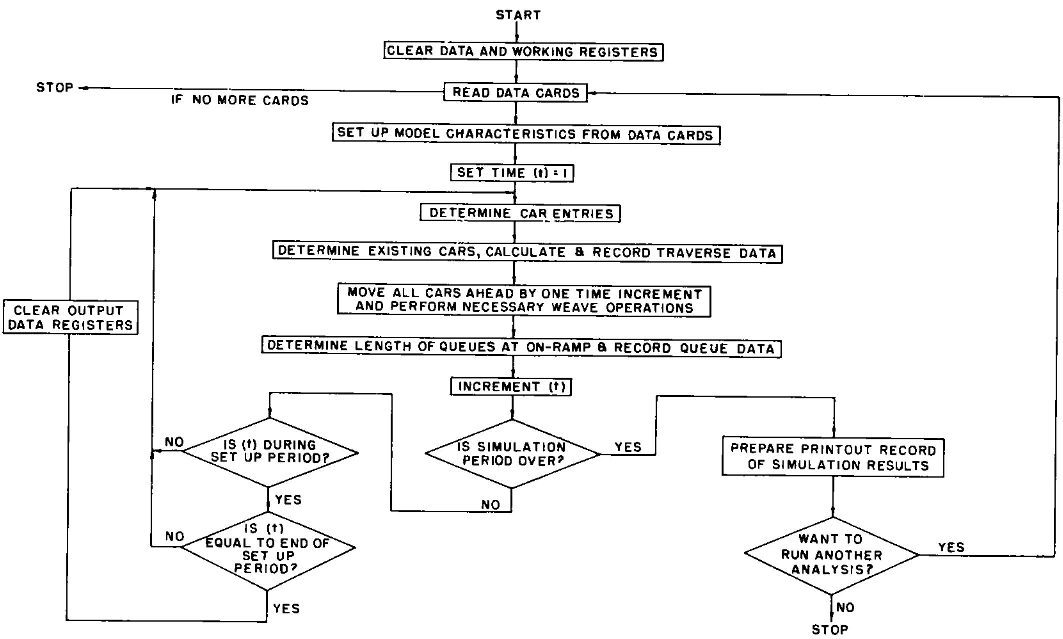


Figure 11. Flow diagram of over-all computer logic.

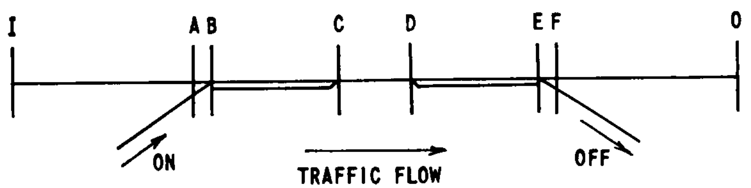
tion lane lengths; (c) various velocity distributions; (d) combinations of the above; and (e) various geometric configurations, etc., can be carried out with this model.

With the limited machine time available, it was decided to carry out experiments on the effect on traffic flow of spacing between an on-ramp and an off-ramp, under varying traffic volumes. The geometric configurations which were utilized, and the simulation input data, are described in this section.

Interchange Configurations

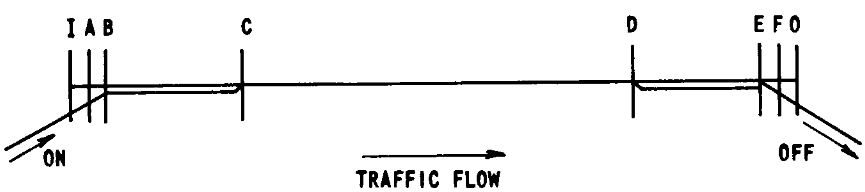
Two interchange configurations (Fig. 12) were examined. Each configuration was 200 blocks or 3,400 ft long and contained one on- and off-ramp combination. Each acceleration and deceleration lane was 595 ft long. In Configuration I exiting vehicles have 2,465 ft to travel to the nose of the off-ramp while in Configuration II the distance is 3,230 ft. In Configuration I the distance between the acceleration and deceleration lane is 340 ft and 1,870 ft in Configuration II. All exiting vehicles were designated at block number 199 (that is, as they enter the system).

CONFIGURATION I



	BLOCK	DISTANCE
I =	200	3400 FT
A =	150	2550 FT
B =	145	2465 FT
C =	110	1870 FT
D =	90	1530 FT
E =	55	935 FT
F =	50	850 FT
O =	00	000 FT

CONFIGURATION II



	BLOCK	DISTANCE
I =	200	3400 FT
A =	195	3315 FT
B =	190	3230 FT
C =	155	2635 FT
D =	45	765 FT
E =	10	170 FT
F =	5	85 FT
O =	00	00 FT

Figure 12. The configuration of the two sections of freeway being examined.

Input Data

Volume of Traffic.—For both configurations the input to the simulation was for 750 ramp vehicles per hour. For Configuration I experiments with through-lane volumes of 2,000, 3,000, 4,000, 5,000 and 6,000 vehicles per hour were conducted. For Configuration II experiments with through-lane volumes of 1,000, 2,000, 3,000, 4,000, 5,000 and 6,000 were conducted. Two tests were conducted at each volume.

Distribution of Volume to Lanes.—In all experiments carried out, the traffic volumes were assigned to the three lanes according to the following relationships:

$$P_1 = 0.43693 - 0.22183\alpha + 0.05730\alpha^2 - 0.00046\alpha^3 \quad (3)$$

$$P_2 = 0.48820 - 0.03136\alpha + 0.00006\alpha^2 + 0.00024\alpha^3 \quad (4)$$

$$P_3 = 0.07487 + 0.25319\alpha - 0.05736\alpha^2 + 0.00382\alpha^3 \quad (5)$$

in which

p_i = proportion of total volume in the i^{th} lane; and

α = total freeway volume in thousands of vehicles per hour.

These relationships are based on data observed in Chicago (1), and were utilized, as the digital program for them was available from the previous study.

Velocity Distributions.—The velocity distributions used with both highway configurations are based on the analysis of the Detroit data. Three different velocity distributions were used in the simulations. One velocity distribution was used for the on-ramp vehicles, a second for the vehicles in Lane 1 and a third for vehicles in Lanes 2 and 3 (Table 6).

Gap Acceptance Distributions.—Three different gap acceptance distributions were used in the simulation tests. The distributions for merging vehicles (both stopped and moving) are given in Table 7. The gap acceptance data for vehicles weaving between lanes are given in Table 7 (1).

TABLE 6

INPUT VELOCITY DISTRIBUTIONS

Vehicle Velocity (blocks/sec)	Cumulative Percent		
	Ramp Lane	Lane 1	Lanes 2 and 3
1.50	0.026	0.001	0.003
2.00	0.061	0.010	0.015
2.38	0.116	0.040	0.034
2.81	0.314	0.180	0.064
3.25	0.683	0.435	0.138
3.69	0.888	0.737	0.322
4.13	0.979	0.899	0.759
4.56	0.994	0.989	0.970
5.00	0.998	0.999	0.998
5.44	1.000	1.000	1.000

Exiting Vehicles.—For both freeway configurations 10 percent of the through vehicles were designated as exiting vehicles. These exiting vehicles were further allocated to the three lanes according to the following schedule:

1. Ninety percent of exiting vehicles in Lane 1 at Block No. 199;
2. Nine percent of exiting vehicles in Lane 2 at Block No. 199; and
3. One percent of exiting vehicles in Lane 3 at Block No. 199.

TABLE 7

GAP ACCEPTANCE DISTRIBUTION FOR WEAVING VEHICLES

Length of Gap (w) (sec)	Cumulative Probability of Acceptance
0.00-0.25	0.00
0.26-0.50	0.00
0.51-0.75	0.00
0.76-1.00	0.00
1.01-1.25	0.10
1.26-1.50	0.30
1.51-1.75	0.60
1.76-2.00	1.00

EXPERIMENTAL RESULTS

The controlled experiments described in the section on "Design of Experiments," were carried out on a 704 digital computer. The ratio of computer time to real time varied from 1 to 5 for low traffic volumes, to almost 1 to 1 for the higher volumes.

The over-all results of the experiments indicated that there were no significant effects on the traffic flow patterns of the freeway as a result of this change in geometric configuration.

The remainder of this section is devoted to the examination of a number of the simulation outputs in order to obtain a quantitative view of the traffic performance. The following outputs will be examined:

1. The distribution of through-vehicle traverse times;
2. The average velocity-volume relationships;
3. The vehicle distribution to lanes;
4. The number of vehicles stopping on the acceleration lane;
5. The number of exiting vehicles that cannot exit; and
6. The number of weaving movements on the freeway.

The Distribution of Through-Vehicle Traverse Times

Distributions of the through-vehicle traverse times were obtained for each through-volume simulated for both configurations. In each case the distributions were skewed to the right. There did not appear to be any difference in traverse time between the

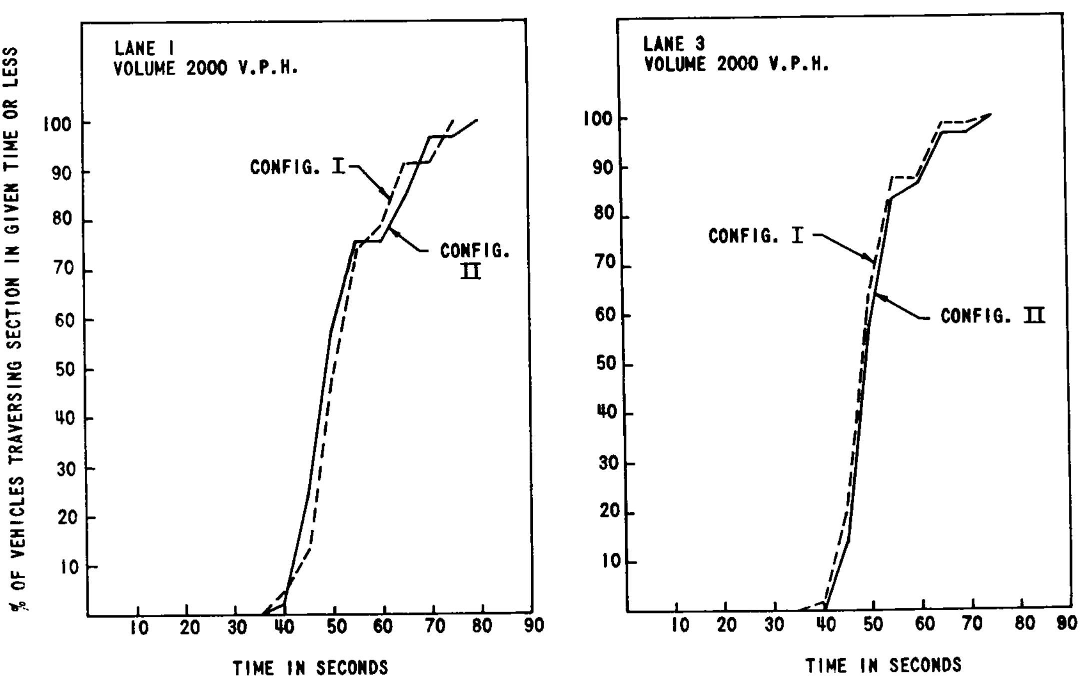


Figure 13. Cumulative distribution of through vehicle traverse times at a freeway volume of 2,000 vph.

two freeway geometric configurations. Cumulative distributions for Lanes 1 and 3 for each configuration at traffic volumes of 2,000 and 6,000 vehicles per hour are shown in Figures 13 and 14. For example (Fig. 14), in Lane 1 with a total freeway through-volume of 6,000 vehicles, 88.6 percent of the vehicles had a traverse time of 75 sec or less in Configuration II, while in Configuration I, 84.1 percent of the vehicles had a traverse time of 75 sec. From these cumulative distributions it is apparent that the difference in configuration does not affect the traverse time to any great extent. The total through-volume, however, does have an influence on the traverse time. An increase in through-volume causes an increase in average traverse time.

Average Velocity-Volume Relationships

The relationship between the average velocity and the volume of traffic in the lane was examined for each of the two configurations. The purpose of this examination was to evaluate the effect that distance between adjacent on- and off-ramps had on the average velocity. There was no apparent average velocity difference for the two freeway configurations under investigation. Linear relationships between average velocity and vehicle volumes of the form in Equation 6 were determined.

$$Y = a + bX \tag{6}$$

in which

Y = average velocity;

X = volume of traffic in lane (veh/hr); and

a, b = constants.

The relationship holding in each configuration in Lane 1 are shown in Figure 15. Ninety-five percent confidence limits were established around each regression line. These confidence limits indicate the area within which the true values indicated by the regression line will lie 95 percent of the time. Because of the large overlap of areas between the two configurations, one cannot assume any significant difference between the average velocity-volume relationship of the two configurations. A complete tabulation of the average velocity-volume relationship for each lane is given in Table 8. In each case correlation coefficients $r > 0.80$ were found.

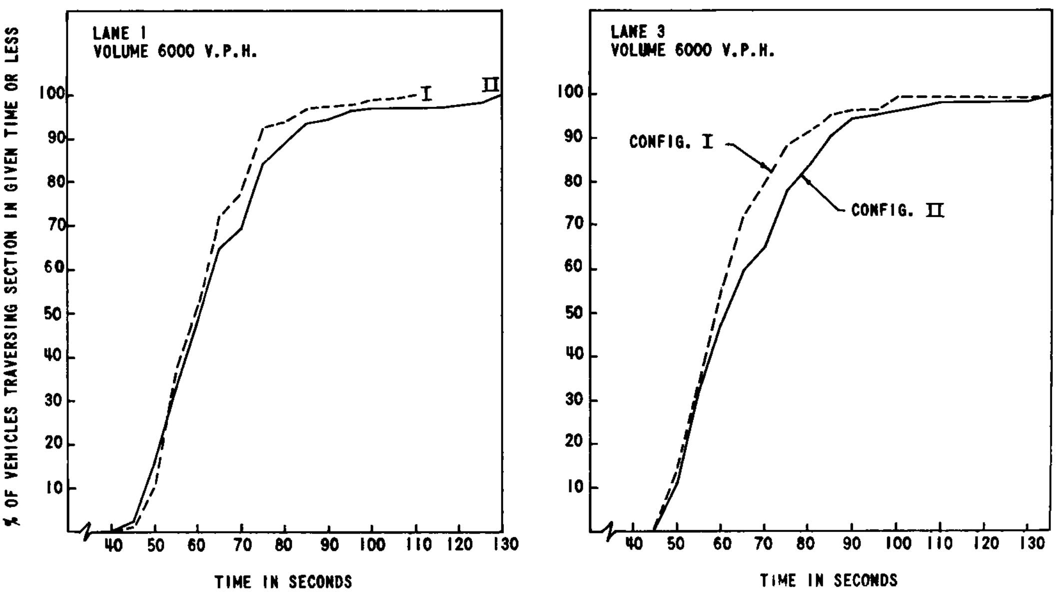


Figure 14. Cumulative distribution of through vehicle traverse times at a freeway volume of 6,000 vph.

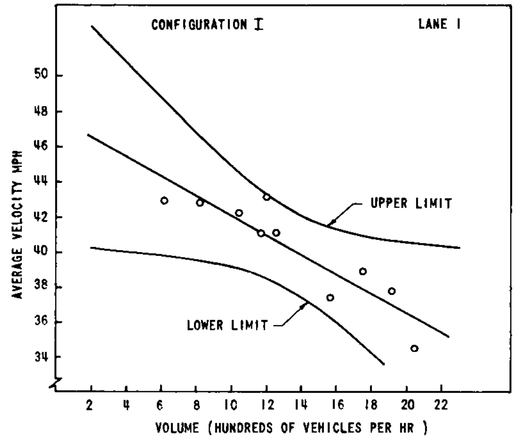
Vehicle Distribution to Lanes

The distributions of the vehicles to the three lanes were examined for each configuration. The output was considerably different from the input. For example, the input relationship between the percent of traffic in Lane 1 and the total volume of traffic was given by Equation 7.

$$P_1 = 0.43693 - 0.22183\alpha + 0.05730\alpha^2 - 0.00046\alpha^3 \quad (7)$$

Chi square tests were used to compare the output and the input, and for each configuration one had to reject the hypothesis that the output had the same form as the input distribution. This change of distribution may be caused by the great number of weaving movements which occur in an interchange area.

Plots of the distribution of traffic to lanes at the various volumes are shown in Figure 16. Regression analysis was used to relate the distribution to lanes and total volume of traffic. The results, along with the correlation coefficients, are given in Table 9. These results may indicate that in Configuration II, where the output is measured at a greater distance from the on-ramp than in Configuration I, the traffic has had an opportunity to reach a steady-state condition. In Configuration I, a number of ramp vehicles may still be weaving between lanes in order to reach a lane in which the traffic moves at the vehicle's desired rate.



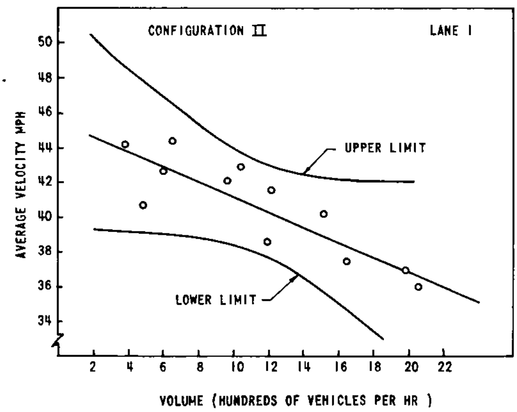


Figure 15. Relationship between average velocity and the number of vehicles in the lane.

TABLE 8

LINEAR EQUATIONS FOR AVERAGE VELOCITY-VOLUME RELATIONSHIPS

Configuration No.	Lane No.	Equation	Coefficient Correlation (r)	Test of Significance at 0.05 Level
I	1	Y = 47.698 - 0.0056X	-0.912	Significant
	2	Y = 49.918 - 0.0049X	-0.847	Significant
	3	Y = 51.516 - 0.0062X	-0.865	Significant
II	1	Y = 45.515 - 0.0043X	-0.865	Significant
	2	Y = 48.940 - 0.0046X	-0.860	Significant
	3	Y = 49.253 - 0.0058X	-0.853	Significant
I and II data grouped	1	Y = 46.180 - 0.0047X	-0.876	Significant
	2	Y = 49.160 - 0.0046X	-0.850	Significant
	3	Y = 49.958 - 0.0054X	-0.848	Significant

Number of Vehicles Stopping on Acceleration Lane

For each traffic-volume input, information was obtained about the number of vehicles which were forced to stop on the acceleration lane because they could not find a suitable gap for merging. The relationship between the percent of ramp vehicles stopping and the total volume of through traffic for both configurations is shown in Figure 17. Ninety-five percent confidence limits about each regression line are shown. The overlap of the confidence regions indicates that there is no difference between the two configurations. This result is to be expected because this difference in interchange configuration should have no effect on the gap arrivals. The regression line for the grouped data is shown in Figure 18 with its confidence region. This output indicates that with traffic volumes as high as 6,000 vehicles per hour, there are still sufficient gaps in the traffic for merging operations. This finding agrees with the findings of Webb and Moskowitz in their study of California freeways (2).

Number of Exiting Vehicles that Cannot Exit

The simulation output includes a count of the number of vehicles that have been tagged as exiting vehicles which cannot weave to the off-ramp in the distance allotted. In Configuration I, exiting vehicles have 2,465 ft in which to move to the nose of the off-ramp. In Configuration II, the distance available is 3,230 ft. A

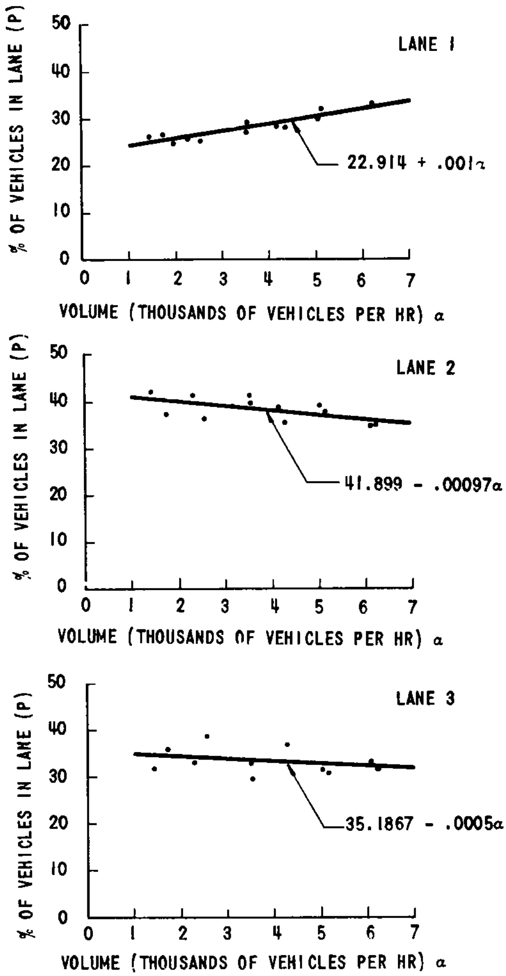
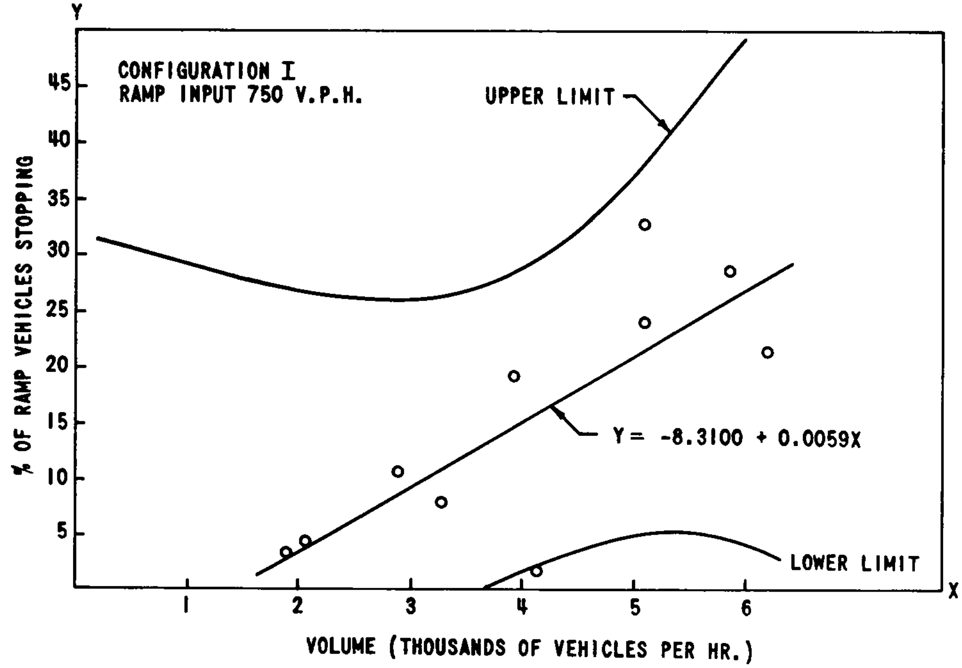


Figure 16. Distribution of traffic to the three lanes on configuration II.

TABLE 9

LINEAR EQUATIONS FOR THE VOLUME AND LANE DISTRIBUTION RELATIONSHIP

Configuration No.	Lane No.	Equation	Coefficient Correlation (r)	Test of Significance at 0.05 Level
I	1	$P_1 = 25.8081 + 0.00083a$	+0.450	Not Significant
	2	$P_2 = 40.5669 - 0.00062a$	-0.308	Not Significant
	3	$P_3 = 33.4049 - 0.00004a$	-0.019	Not Significant
II	1	$P_1 = 22.9138 + 0.00149a$	+0.905	Significant
	2	$P_2 = 41.8995 - 0.00097a$	-0.623	Significant
	3	$P_3 = 35.1867 - 0.00052a$	-0.313	Not Significant



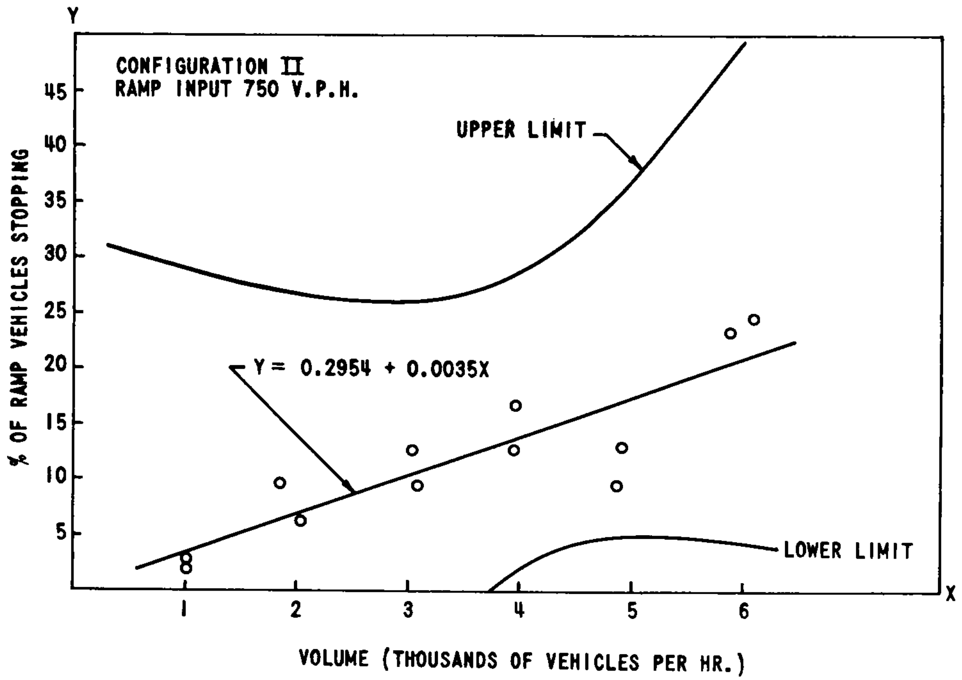


Figure 17. Relationship between the percent of vehicles stopping on acceleration lane (Y) and the total through volume (X).

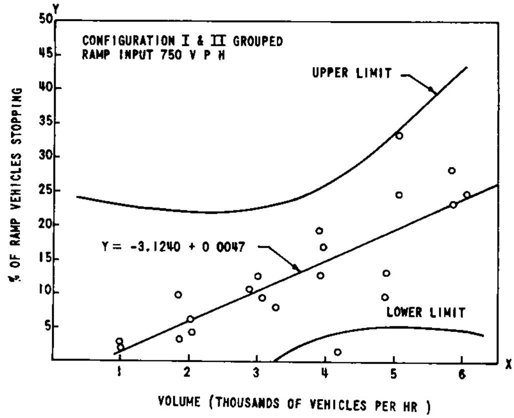


Figure 18. Percent of vehicles stopping on ramp vs the total through volume (grouped data).

scatter diagram of the number of vehicles which cannot exit at various traffic volumes is shown in Figure 19. The variation is such that it is not possible to determine whether one configuration is superior to the other.

With 10 percent exiting vehicles (90 percent of which are in Lane 1, 9 percent in Lane 2, and 1 percent in Lane 3), it is necessary to start the exiting behavior more than 3,230 ft before the nose of the off-ramp to insure that all vehicles can complete their maneuver. This information may be helpful in determining where to post exit signs.

To examine the effect of a lower percentage of exiting vehicles, simulations were carried out for a 6 percent value. A comparison of results for Configuration II with 6 and 10 percent exiting vehicles are given in Table 10.

Weaving Movements on Freeway

During a 5-min time interval the total number of weaves (Y) between the lanes occurring on the 3,400-ft section of freeway increased linearly with the volume of traffic (X). A comparison between the number of weaves from Lane 1 to 2 for the two highway

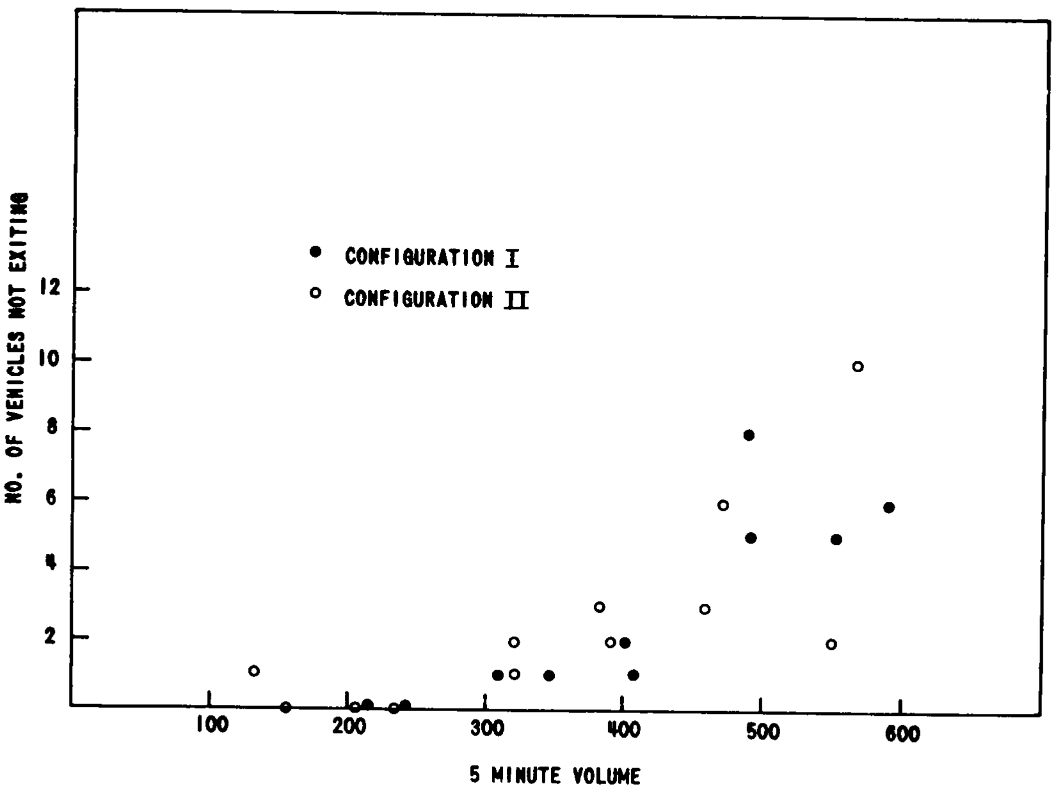


Figure 19. Scatter diagram of the number of exiting vehicles not able to reach the off-ramp.

TABLE 10

COMPARISON OF NUMBER OF VEHICLES NOT ABLE TO EXIT

Percent of All Vehicles Exiting	5-Min Volume of Traffic	Number of Vehicles (Not Able to Exit)
10	384	3
	391	2
6	382	1
	398	0

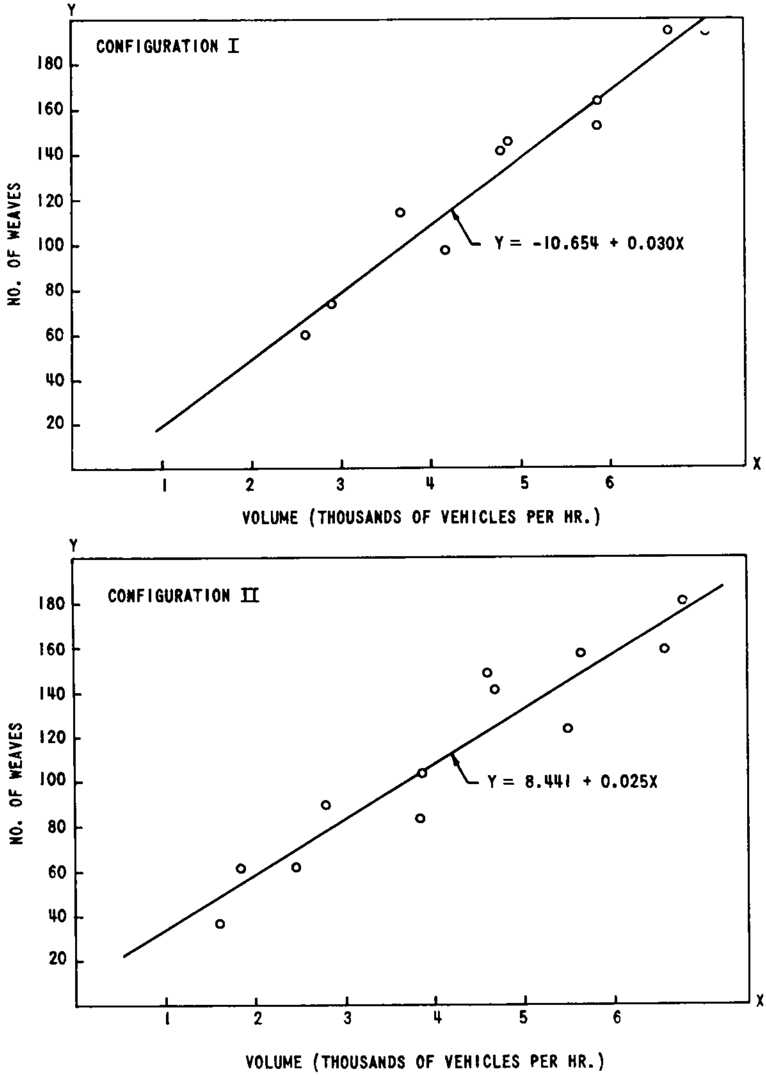


Figure 20. Number of weaves in 5 min on a 3,400-ft section of freeway as a function of total through volume.

configurations is shown in Figure 20. Table 11 gives the relationship between Y (the number of weaves) and X (the hourly volume of traffic). These results show that even at volumes as high as 6,000 vehicles per hour, sufficient gaps exist to permit weaving.

TABLE 11

RELATION BETWEEN THE NUMBER OF WEAVES (Y) AND THE VOLUME OF TRAFFIC (X)

Freeway Configuration	Lane Weaving Movement From	To	Linear Relation ($Y = a + bX$)	Correlation Coefficient (r)	Test of Significance At 0.05 Level
I	1	2	$Y = -10.654 + 0.030X$	0.975	Significant
	2	1	$Y = -29.807 + 0.032X$	0.936	Significant
	2	3	$Y = 27.903 + 0.024X$	0.824	Significant
	3	2	$Y = 13.743 + 0.029X$	0.799	Significant
II	1	2	$Y = 8.441 + 0.025X$	0.946	Significant
	2	1	$Y = -1.337 + 0.029X$	0.965	Significant
	2	3	$Y = -29.350 + 0.037X$	0.950	Significant
	3	2	$Y = -35.052 + 0.033X$	0.949	Significant

CONCLUSIONS AND RECOMMENDATIONS

This study has shown that digital simulation can be used to faithfully duplicate actual traffic flow in an on- and off-ramp area of a freeway. The output of the simulation furthermore, gives measures of effectiveness which can be used to evaluate alternate highway designs.

Experiments were performed for the two following on- and off-ramp configurations:

1. A distance of 1,530 ft between the on- and off-ramps (Configuration I); and
2. A distance of 3,060 ft between the on- and off-ramps (Configuration II).

The resulting traffic flow characteristics for both systems were similar over the volume range 1,000 to 6,000 vehicles per hour.

The computer outputs, however, do indicate that the area immediately following an interchange is an area of transient behavior. The distribution to lane versus volume relationship for vehicles leaving the interchange area has less variation, as indicated by the correlation coefficient, in Configuration II than in I. Output measurements in Configuration II were performed at a greater distance from the on-ramp than in Configuration I. Similar results were found to be true for the actual data collected by the Bureau of Public Roads in Detroit (Table 3). These results indicate the need to measure traffic flow variables at a distance from the on-ramp. It is, therefore, important for effective comparison of traffic data that the location of the sensing equipment should be standardized. Such standardization may well be prescribed through simulation studies.

A further result of the study is useful for planning the location of exit signs. For example, a simulation with 750 on-ramp vehicles per hour and with 400 or more exiting vehicles at an adjoining off-ramp was performed. Results indicated that fewer than 10 percent of the exiting vehicles could be in Lanes 2 and 3 if all exiting vehicles were to successfully perform their exiting movement. For the above volume conditions, exit signs should be located more than 3,000 ft in advance of the off-ramp if friction induced by exiting cars is to be minimized.

The simulation experiments performed indicate the need for research and experimentation in a wide variety of areas to answer questions such as the following:

1. What is the effect of various vehicle distributions to lanes on the traffic flow?
2. What is the effect on traffic flow of various distances between adjoining on-ramps?

3. What is the effect of various desired velocity distributions on traffic flow?
4. At what volume of traffic do weaving movements between lanes become hazardous?
5. What is the effect on traffic flow of various volumes of commercial vehicles?

The simulation model developed in this study can serve as an efficient tool to answer these and other problems in the continuous quest for means of moving traffic safely and efficiently.

REFERENCES

1. "Application of Digital Simulation Techniques to Freeway on Ramp Traffic Operations." Final Report, MRI Project No. 2234-P (Sept. 1, 1958-June 30, 1959).
2. Webb, G. M. and Moskowitz, K., "California Freeway Capacity Study—1956." Proc., 36th Ann. Meeting HRB, NAS-NRC Pub. 542.

A Quantitative Evaluation of Traffic in a Complex Freeway Network

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The proposition that travel time is a fundamental dependent variable in the analysis of transportation systems is developed along with several corollary concepts. The proposition and some of the derived concepts are then put to test in a detailed investigation of traffic in a part of the freeway network in downtown Los Angeles. The network of interest includes two "input" freeways (the Santa Ana and San Bernardino Freeways), three "output" freeways (the Harbor, Hollywood, and Pasadena Freeways), and three intervening on-ramps and one off-ramp which come into the network between the input and output freeways.

License plate methods, including dictation into portable tape and wire recorders and high-speed photography, are used to obtain both travel time and the relative percentages of traffic flow in all the combinations of input-output freeways as well as freeway-ramp combinations. Speeds and headways are measured by lane at the output boundary of the network of interest which is on a freeway proper. Other covariates are classified volume counts which also are by lane on the output boundary. A novel mailing questionnaire is used to establish the surface street paths drivers pursue to get to the on-ramps of the network. In light of a response of the order of 60 percent of the original mailing of 400 questionnaires, the particular techniques used, appear to be quite promising for application to more extensive networks.

Representative of the hypotheses tested are: the effect of on- and off-ramp traffic on network travel time; the effect of multi-axle vehicles on network travel time; (deduced) effects that opening planned additional links of the freeway system will have on the existing network; (deduced) effects that closing a ramp would have on the network as well as the adjacent surface streets.

Besides providing a quantitative description of traffic in the selected network, the study demonstrates that mathematical models based on travel time can be applied to real situations. Although there can be no formal proof of the importance of these models, the fact that they yield useful information in this case suggests that more general applications might be considered.

●A NUMBER OF SITUATIONS have recently been reported in various parts of the country in which decisions have had to be made to change some operational aspect or physical feature of an existing high-performance transportation facility such as a freeway network. Many more decisions of a similar nature will, no doubt, have to be made in the future, partly because the state of the art of land-use planning has not progressed to where evolving land use can be reliably predicted, and partly because the transportation art has not progressed to where the interactions between freeway networks and changing land uses in a burgeoning metropolitan region can be understood.

An emergent need, therefore, will be for increasingly rigorous measurements of freeway systems. These measurements will represent mandatory inputs into any decision-making process to alter the facilities in any way.

The immediate purpose of this paper is to present methods for quantitatively evaluating the performance of a freeway network or a section thereof pursuant to decision-making on changes to improve network performance. The methods are then demonstrated in a study performed in a relatively complex section of a heavily traveled freeway network in downtown Los Angeles. The underlying operational decision in this study concerns the closing of a particular on-ramp, but the ultimate disposition of this problem is not germane to this paper; it is only the methodology that is being emphasized here.

Included in the methodology are certain field techniques that were developed specifically for measuring freeway performance under high-speed, high-volume conditions with minimum disturbance, if any, of traffic. The measurements are inputs into a mathematical model of traffic flow in a network with travel time as the fundamental dependent variable. This concept is not entirely new, for a number of studies have been published which in essence treat traffic in this, or some related manner (1, 2, 3, 4, 5, 6, 7). The difference lies in the manner in which the concept is applied to the movement of ensembles or collections of vehicles through complex networks of flow paths rather than to the movement of a single vehicle.

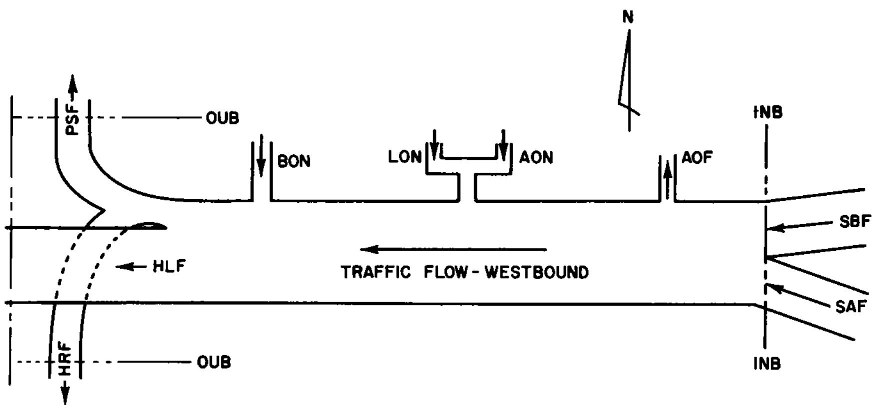
Finally, a more or less peripheral purpose of this paper is to demonstrate a nomenclature that was directly suggested by FORTRAN language used in writing programs for the IBM 709 computer. This computer was used in this study for computing speed, headway, and several other properties of the observed traffic.

NOMENCLATURE

A section of the freeway network in Los Angeles is designated as the "system-of-interest" or simply "system" as shown in Figure 1, and approximately to scale in Figure 2. Figure 3 shows the freeway links which either are under construction or are programmed in the Los Angeles area, and the completion of which might affect the performance of the system of interest.

The primary traffic flow in the system is from east to west so that an imaginary line to the east is defined as the input boundary, INB, of the system, whereas an imaginary line to the west is defined as the output boundary, OUB, of the system.

Input traffic at INB is from either of two freeways, the Santa Ana Freeway, SAF, or the San Bernardino Freeway, SBF. Output traffic is to either of three freeways: the



OUB - Output Boundary
PSF - Pasadena Freeway
HLF - Hollywood Freeway
HRF - Harbor Freeway

BON - Broadway Avenue On-Ramp
LON - Los Angeles Street On-Ramp
AON - Alameda Avenue On-Ramp
AOF - Alameda Avenue Off-Ramp

INB - Input Boundary
SBF - San Bernardino Freeway
SAF - Santa Ana Freeway

Figure 1. Schematic diagram of system of interest.

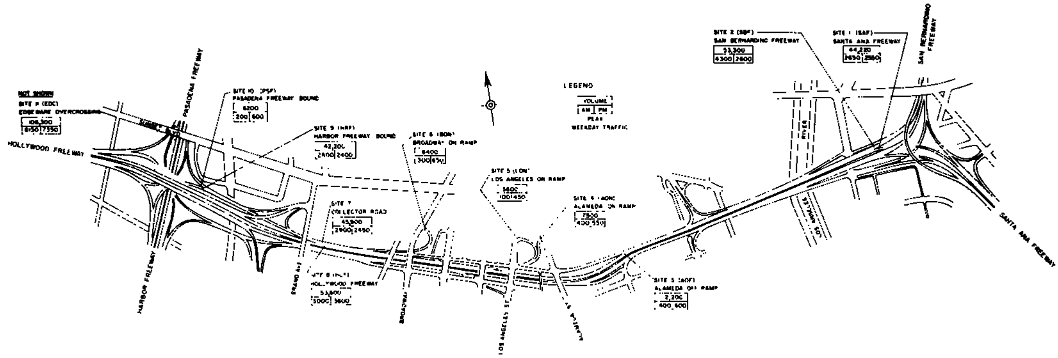


Figure 2. The system of interest as a part of the freeway network.

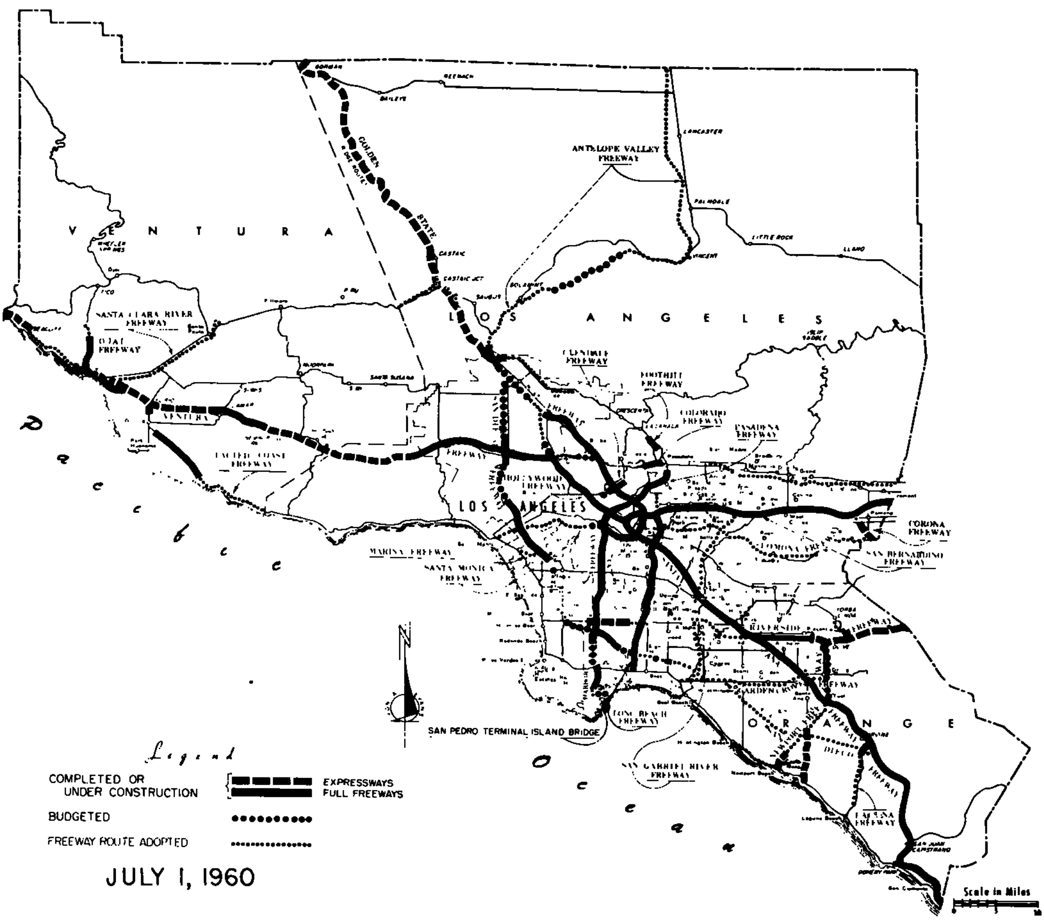


Figure 3. District VII freeways, State of California.

Hollywood Freeway, HLF; the Harbor Freeway, HBF; the Pasadena Freeway, PSF. The HLF traffic continues westbound, PSF traffic is northbound, HBF traffic is southbound. The additional input traffic into the system is via three on-ramps: Broadway Avenue, BON, Alameda Street, AON, and Los Angeles Street, LON. Additional output traffic leaves the system at the Alameda off-ramp, AOF. All three on-ramps and the one off-ramp are between INB and OUB.

Traffic of interest is identified in two ways: first, according to input and output sites; and second, by performance expressed in travel time.

General

System boundaries

INB	input boundary
OUB	output boundary

On-ramps of interest

BON	Broadway on-ramp
LON	Los Angeles on-ramp
AON	Alameda on-ramp

Off-ramp of interest

AOF	Alameda off-ramp

Freeways

SBF	San Bernardino Fwy
SAF	Santa Ana Fwy
HRF	Harbon Fwy
HLF	Hollywood Fwy
PSF	Pasadena Fwy

Ramp status

O	ramp open
C	ramp closed (hypothesized)

Examples:

BONO	Broadway on-ramp open
BONC	Broadway on-ramp closed

Compound attributes (examples)

(SAF) (HRF)	traffic going from SAF to HRF (Santa Ana Fwy to Harbor Fwy), or traffic at SAF going to HRF.
(HRF) (SAF)	traffic at HRF coming from SAF, or traffic going from HRF to SAF.

Operators

N ( )	"number of" operator; specifically, number of vehicles (volume) having attribute shown in parentheses.
P [( ) ( )]	"percentage of" operator; specifically, percentage of vehicles having attribute shown in first parentheses also having attribute in second parentheses.

Examples:

N (SAF) (HRF)	number of vehicles going from SAF to HRF.
P (SAF) (HRF)	percentage of vehicles at SAF that go to HRF.

Vehicle Attributes

I	the Ith vehicle.
CLT	clock time, or time of day.
TRT	travel time.
RSP	reciprocal speed, where if travel time is measured over a distance C:

$$RSP = \frac{(TRT)}{C} = \frac{[CLT\ (OUB)] - [CLT\ (INB)]}{C}$$

CLT (I. INB) = clock time at which the I^{th} vehicle crosses the input boundary of the system; that is, the time of day the I^{th} vehicle enters the system.

CLT (I. OUB) = clock time at which the I^{th} vehicle crosses the output boundary of the system; that is, the time of day the I^{th} vehicle leaves the system.

RSP (I. OUB) = reciprocal speed of I^{th} vehicle measured at output boundary; the distance C over which the travel time is measured is sufficiently small to justify treating RSP as applying to point OUB.

TRT (I. INB-OUB) = travel time of I^{th} vehicle in moving from input to output boundary of the system:

TRT (I) (INB=OUB) = CLT (I. OUB) - CLT (I. INB)

MEASURE OF EFFECTIVENESS

The measure of effectiveness in this study is the summation of individual travel times for all vehicles arriving at the system or the adjacent surface street network during some arbitrarily chosen interval of clock time to clear the output boundary of the system or surface street network (whichever applies). Symbolically:

CLT (1-2) CLT (1) to CLT (2) (1)

N (INB. CLT 1-2) number of vehicles arriving at INB during CLT 1-2 (2)

N (BON. CLT 1-2) number of vehicles arriving at BON during CLT 1-2 (3)

N (SSN. CLT 1-2) number of vehicles arriving at SSN (surface street network) during CLT 1-2 (4)

TRT (I. INB) travel time of the I^{th} vehicle arriving at INB of the system to clear OUB of the system

CLT (I. OUB) - CLT (I. INB) (5)

$$\text{TRT (INB. CLT 1-2)} \quad \sum_{I=1}^{I=N\,(\text{INB. CLT 1-2})} \text{TRT (I. INB)} \qquad (6)$$

MOE (CLT 1-2) the measure of effectiveness for CLT 1-2

$$\sum_{\text{all sets}} \text{TRT (set. CLT 1-2)} \qquad (7)$$

where the sets include:

N(INB. CLT 1-2), N(LON. CLT 1-2), N(AON. CLT 1-2)
N(BON. CLT 1-2), N(SSN. CLT 1-2)

OVER-ALL STRUCTURE OF THE STUDY

The measure of effectiveness as defined in Eq. 7 is to be quantified by direct field measurements and certain questionnaire techniques for what would normally be regarded as "before" performance of the system. The "before" here means prior to some operational decision; for example, to close an on-ramp. However, there are no complementary "after" experimental measurements because all operational decisions here are only hypothetical. Accordingly, "after" performance can be arrived at solely by analysis. The general study will, therefore, compare a measured MOE with one arrived at by analysis.

The general study is comprised of several sub-studies, each of which provides one or more of the required inputs to the general problem of the MOE comparisons. However, each sub-study essentially is a complete study within itself, and could be profitably pursued in less general contexts. The sub-studies are listed here briefly to show how they fit into the general plan, and are detailed more fully in the next section.

Sub-study 1—ascertains the spectrum of trip lengths originating at on-ramps of interest.

Sub-study 2—ascertains the pattern of surface street routes traffic follows in getting to the various on-ramps of interest.

Sub-study 3—seeks to estimate properties of the traffic stream at the output boundary of the system.

Sub-study 4—seeks to establish travel time needed to negotiate the "system" over various paths (that is, input-output combinations) and any attendant TRT diseconomies.

Sub-study 5—seeks to estimate the relative traffic volumes using the different possible input-output combinations of the freeway network.

Field measurements were conducted on two days, a Sunday and a weekday. On each of these days the sampling was limited to four 15-min periods in the off-peak hours and a single 1-hr period during the afternoon peak. Because the primary purpose of this paper is to present the technique and underlying logical framework, the data analysis is limited to one 15-min off-peak period and one 1-hr peak period. Clearly, pursuant to any actual operational decision-making (which is not the purpose here), the sampling would have to be broadened.

CONCEPTUAL FRAMEWORK

The "Collection"

The measure of effectiveness as stated in Eq. 7 pertains to a specific group of vehicles; namely all vehicles arriving at the system or at the adjacent surface street network during a specified time interval. This group of vehicles will be referred to as "the collection". The collection is specific to the specified time interval which may be arbitrarily varied to reflect any given operational problem or situation. For example, there could be "the afternoon peak collection" which would include the vehicles arriving during the 2-hr interval corresponding to the afternoon peak.

It is assumed here that the total number of vehicles in the collection remains constant once the defining clock time interval is established. There might be some redistribution of the vehicles in the collection among the different route possibilities after some operational change. For example, some freeway users might become surface street users, or users of a given ramp might use a different ramp, etc. But the total number of vehicles, although redistributed, would be the same for that clock interval. A higher order analysis would, of course, permit the size of the collection to change as a consequence of the operational change, but this study is limited to the more simple analysis.

Inasmuch as the analysis is centered on the properties of the collection rather than of the single vehicle, it is referred to here as "macroscopic"; the single-vehicle type of analysis, in contrast, could be referred to as "microscopic". There already has been extensive theoretical work on microscopic flow as, for example, the movement of the individual vehicle in the stream as reported in various publications on car following theory (8, 9, 10). However, very little work has been done on macroscopic theory as defined here.

(It is interesting to note that macroscopic analyses of physical systems in the classical thermodynamic mode have been yielding highly useful engineering solutions to practical problems for many years, well before the microscopic treatments of statistical mechanics were conceived. There are, of course, many problems today which can be attacked only with statistical mechanics. However, the results are accepted as valid when they do not conflict with whatever classical measurements can be made of macroscopic properties such as temperature or pressure.)

(In transportation theory, on the other hand, microscopic treatments have preceded macroscopic. Accordingly it might be speculated that the absence of macroscopic work is one explanation why theorists have not produced many solutions to immediate problems confronting planners, designers, and ultimately the operators of vital transportation systems.)

Because the collection is defined by a specified time interval, its size (number of vehicles) is a random variable which could have been taken as the measure of effectiveness instead of travel time. In fact, such "volume" models generally characterize much of the work in transportation theory and practice; for example, the volume of traffic clearing a system within a specified time period. There are, however, several reasons why volume is not selected as the measure of effectiveness in this analysis, although it still enters as a covariate to the selected measure, travel time.

Ambiguous Meaning of Low Volume. —A low flow rate expressed as vehicles per unit time can mean that either of two situations prevail: first, there is not much demand no matter how congested or uncongested the system might be; second, there is a high degree of congestion. To identify the situation, additional information is needed, usually involving some form of time measurement.

Non-Definitive Meaning of High Volume. —In the analysis of traffic flow in a network, a high volume or flow rate at the output boundary of the network gives no indication of network conditions, any number of which could be compatible with the high output rate. Although all would be equivalent in this regard, they might be vastly different from the standpoint of over-all network performance. For example, suppose that the flow at some output boundary of a freeway were at the maximum possible rate of 2,200 vehicles per lane per hour, but at the same time the queue of traffic waiting to enter the system continued to lengthen as a result of input rates upstream being greater than the maximum possible output rate at the output boundary. From the volume standpoint, the network would be operating at maximum capacity. However, it could well be that short trip users of the system could be causing an unreasonable increase in the travel time of the long trip users. Thus, from the travel time viewpoint the network would not be operating optimally, even though the output flow was at a maximum rate.

Insufficiency of Volume as a Network Variable. —The critical transportation problem today in metropolitan regions involves the daily home-to-work travel cycle of drivers on surface street and freeway networks. The volume demands on the network on any given day are essentially constant; sooner or later, that collection of vehicles is serviced (that is, all drivers get to their destinations). Therefore the size of the collection—that is, the volume of traffic—cannot possibly be sufficient for describing network performance. The variable aspect is the travel time that each vehicle devotes to its homework travel cycle, or the aggregate travel time for the collection as a whole.

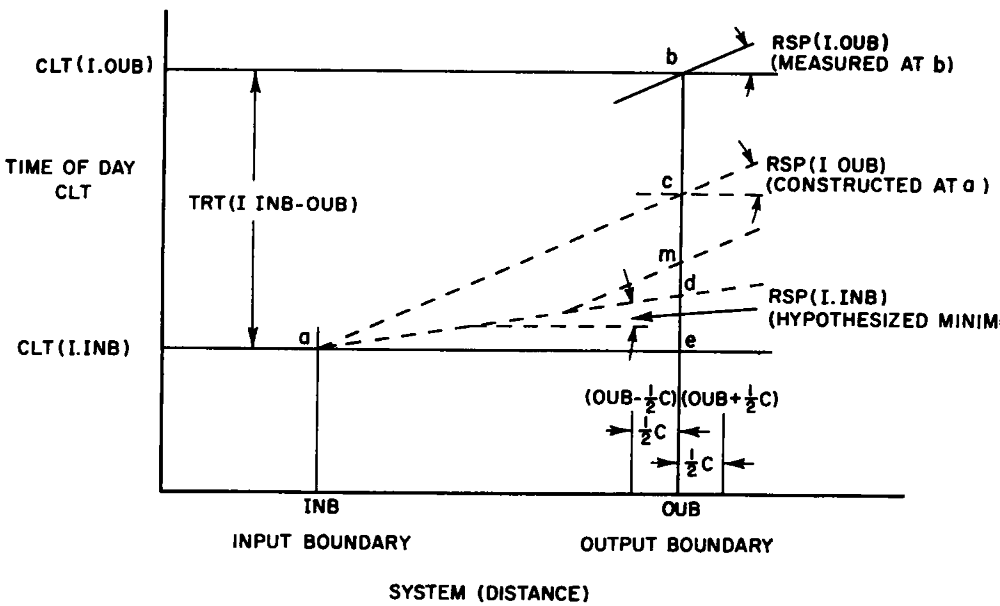


Figure 4. Travel time of the I^{th} vehicle in the system.

Average speeds that can be maintained are often used to describe this variable aspect of network performance. However, the travel distance in the aggregate for the collection is also a constant because the large masses of people do not change their home or place of work from one day to the next. Therefore, including the distance-traveled aspect into the argument, as is implicit in determining average speeds, adds nothing to the analysis.

"TRT" Diseconomies

Consider that the I^{th} vehicle, in entering some system of interest, crosses the defined input boundary INB of the system at some clock time CLT (I. INB). This is represented by point a in the "time-system" space shown in Figure 4. The vehicle later leaves the system, crossing the output boundary OUB at CLT (I. OUB) as represented by point b. By definition, its TRT (the elapsed time while it was negotiating the system) is given by:

$$\text{TRT (I. INB(OUB)} = \text{CLT (I. OUB)} - \text{CLT (I. INB)} \tag{8}$$

The sub-system now is defined as being centered on OUB, with its input boundary at (OUB - ½ C) and its output boundary at (OUB + ½ C). If C is sufficiently small relative to the distance between INB and OUB, then the travel time of the I^{th} vehicle over the sub-system may be considered as the travel time at OUB. This will be assumed to be the case here, so that the reciprocal speed RSP (travel time per unit distance) measured over C can be treated as the RSP at OUB.

$$\text{RSP (I.} \left[(\text{OUB} - \tfrac{1}{2}\text{C}) - (\text{OUB} + \tfrac{1}{2}\text{C})\right]) = \text{RSP (I. OUB)} \tag{9}$$

if

$$\text{C} << (\text{INB} - \text{OUB})$$

In Figure 4, line ac constructed at point a with slope RSP (I. OUB), intersects the OUB ordinate at point c. Line ad is also constructed at point a, but with slope RSP (I. INB). If RSP (I. INB) is a minimum, which would identically imply that speed was a maximum, then point d at the intersection of ad with the OUB ordinate would identify the minimum time it would take a vehicle to travel from INB to OUB. Because the vehicle actually arrives at some later time signified by point b, the length db represents a travel time diseconomy that somehow was incurred by the I^{th} vehicle. The order of magnitude of this diseconomy is an immediate barometer of the performance of the system. Furthermore, defined as it is for an individual vehicle, this diseconomy can readily be summed for all vehicles in the collection to yield a warrant (for more detailed probing into the operation of the system) that is specific to the clock time interval over which the collection was defined. This essentially characterizes the work of Rothrock and Keefer (2), and as they report, represents a very direct approach to the problem.

To analyze the performance of the system more deeply, it is necessary to factor the total diseconomy into parts which can be identified with specific operational aspects of the system. A prior requirement is to identify how much of the total diseconomy is due to internal aspects of the system, and how much is due to external aspects. For example, and an obviously limiting case, if a barrier were placed across the output boundary of the system such that no vehicles could cross it, the ensuing TRT diseconomy could hardly be ascribed to the operation of the system; it would have to be due to the downstream (and hence external) barrier to the system.

In the case shown in Figure 4, the actual arrivel time at the output boundary, point b, does not coincide with what it should have been, point c, had the I^{th} vehicle been able to maintain RSP (I. OUB) throughout the system as well as at OUB. But if the vehicle had been able to maintain RSP (I. OUB), it still would have incurred the diseconomy dc. Thus RSP (I. OUB) is a constraint on performance of the vehicle in the system, and the diseconomy dc may be properly considered as being due, in significant measure, to factors external (that is, downstream) to the system. The diseconomy cb, on the other hand, must be due primarily to occurrences or situations within the system. In this context, therefore, point c splits the total diseconomy into external and internal components.

With point d, by definition, coinciding with maximum possible performance, both dc and db must always be positive. The case dc > db (not shown) implies that the diseconomy cb (which would be negative in this case) is due to factors external to the system. This case is of interest when the analysis deals with conditions downstream of the system. On the other hand, the case dc < db (Fig. 4) implies that the diseconomy cb (which is positive) is due to factors within the system. Inasmuch as the objectives of this study involve analysis of the performance of the system, and not of downstream conditions, the latter case of positive cb is of primary interest.

Effects Producing "Within" TRT Diseconomies

Two factors contributing to the system "within" diseconomy TRTD are considered: (1) the number of cars in the system at the instant any given car in the collection reaches INB, and (2) the number of cars which enter the system after the given car passes INB, but before it crosses OUB. (The "within" diseconomy here, aside from a few minor differences, is the same as the "travel time delay" as defined by Berry and Van Til (5): ".... This type of delay for an individual vehicle is the difference between its actual time required to traverse some fixed distance at the approach to an intersection, and the travel time which would have been required had the vehicle been able to continue at the average approaching speed of traffic....") An analytical construct of "the leading ensemble" or simply "ensemble" used to treat these factors is defined in the following manner.

At the instant CLT (I. INB) that the I^{th} car crosses the input boundary of the system, there exists in the system some set of vehicles. As this vehicle proceeds through the system, the set of vehicles ahead of it changes; some of the vehicles in the original set leave the system at the output boundary or at intervening off-ramps, while other vehicles join the set from intervening on-ramps. The hypothetical summation of the set of vehicles ahead of the I^{th} car at all instants between CLT (I. INB) and CLT (I. OUB) is defined here as its "ensemble", and is illustrated in Figure 5, although this figure pertains to the special case of all vehicles moving at the same speed which will be discussed in detail presently.

The ensemble of the I^{th} car is comprised of three classes of vehicles: (1) some number of vehicles N (INB. OUB) that crossed INB ahead of the I^{th} car (the assumption of uniform speed precludes the possibility of a car entering the system after the I^{th} car, and later passing it to become a part of the ensemble of the I^{th} car), (2) some number of vehicles N (LON. OUB) that come into the mainstream from LON some time after CLT (I. INB), and (3) some number of vehicles N (BON. OUB) that come into the mainstream from BON some time after CLT (I. INB). The travel time of the I^{th} vehicle is considered to have some functional relationship with these three numbers:

$$\begin{aligned} \text{TRT (I)} &= f\ [\text{ENS (I)}] \\ &= f\ [\text{N (INB. OUB), N(LON. OUB), N (BON. OUB)}] \end{aligned} \tag{10}$$

It can be seen from Figure 2 that to the (SAF. HLF) traffic, the (BON. HLF) traffic represents merging movements, while to the (SAF. HRF) traffic, the same (BON.-HLF) traffic represents weaving movements. Similarly, the (SBF. HLF) traffic is weaving across the (SAF. AOF) traffic. It thereby becomes possible to quantify the effects of weaving and merging movements to and from ramps, the separate effects being measured in travel time decrements. Thus, the effects associated with the function (Eq. 10) begin to have general meaning although they are measured in a specific situation.

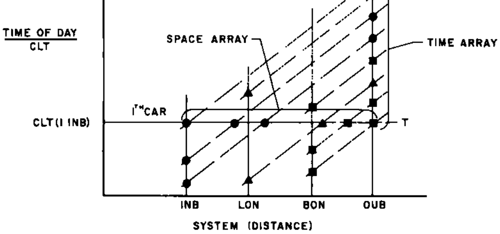


Figure 5. The laminar ensemble.

Another way of classifying the vehicles in the ensemble is by wheelbase, so that the N (INB. OUB) in Eq. 10 could be considered to be comprised of four groups; namely, the four classes of vehicles given under "Nomenclature—Vehicle Attributes". The function in Eq. 10 can then be expanded to

$$\text{TRT (I)} = \text{f } [\text{N (INB.WBC4.OUB), N (INB.WBC2.OUB)}, \ldots \ldots \ldots \ldots \text{N (BON.WBC4.OUB)}] \quad (11)$$

which would then permit evaluating the "truck" effect. For example, if the I$^{\text{th}}$ vehicle in the collection is heading for HRF, and if OUB in the last term in Eq. 11 is HLF, then

$$\text{TRT (I)} = \text{f } \left[\text{N (BON.WBC4, HLF)}\right] \quad (12)$$

describes the effect of weaving trucks. Or, if the I$^{\text{th}}$ car was heading for HLF, Eq. 12 would describe the effect of merging trucks, etc.

Returning to the special case of all vehicles moving at the same uniform and unchanging speed, the locus of each vehicle would be a straight line in the system-time space shown in Figure 5, and would be parallel to the loci of all other vehicles. In this case, which will be called "laminar", the array of vehicles in time at the point OUB can be direcly mapped into the array of vehicles in space at a given instance in time. The time domain of interest at OUB is shown on SS and extends from CLT (I. INB) to CLT (I. OUB); the space domain of interest at CLT (I. INB) is shown on TT and extends from INB to OUB (to wit, the "system").

A large number of laminar situations are possible in any given system, there being a different one for every hypothesized uniform speed. Also, for every laminar case, a large number of different ensembles are possible because for any given number of vehicles in the ensemble, there would be a different ensemble for every different possible spatial (or time) arrangement of input and output vehicles. The particular laminar case of interest here has an average respeed of traffic at the OUB as its hypothesized uniform speed, the average RSP being taken over some clock time domain at OUB, and being of the form

$$\text{Mean (RSP)} = \frac{\sum_{I=1}^{N} \text{RSP (I)}}{N} \quad (13)$$

This study is not concerned with speed as such, but rather with its reciprocal RSP. Aside from this difference, Eq. 13 coincides with what Walker (4) expresses as "Time mean speed":

$$\text{Time mean speed} = \frac{\sum \frac{\text{distance}}{t}}{n}$$

in which t is the travel time for each individual vehicle and n is the number of vehicles.

The array in time realized at OUB rather than the array in space at CLT (I. INB) is treated as defining the ensemble, although the space array seemingly has greater physical meaning as an influence on the travel time for the I$^{\text{th}}$ vehicle to negotiate the system. However, this array is relatively difficult to measure in the field as contrasted with that for time, and because with the laminar assumption it becomes possible to map into the space array from the time array, it becomes feasible to use the more easily measured time array.

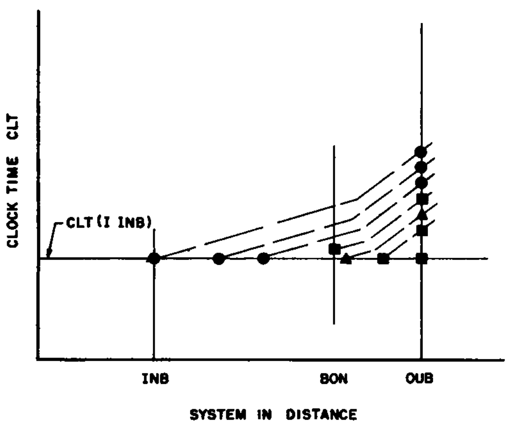


Figure 6. Modified laminar flow yielding MIN TRT (I).

A modified form of laminar demand on OUB is treated here as yielding the minimum "within" travel time for a given size of ensemble. A uniform respeed is assumed at OUB. A fruther assumption is that for this RSP there always is the same time gap GAP (RSP) between successive vehicles leaving the system at OUB. A final assumption, which is not necessary if N is sufficiently large, is that the first of the N vehicles in the ensemble is at OUB at CLT (I. INB). Under these three assumptions, minimum TRT (I) will be

$$\text{MIN TRT (I)} = \text{N} \times \text{GAP (RSP)} \tag{14}$$

This construction is demonstrated in Figure 6, and deviates from the defined laminar ensemble in that it requires each vehicle to move at a lesser respeed than the hypothesized uniform RSP at OUB until it is at the GAP (RSP) behind the vehicle ahead of it and both are moving at RSP (OUB).

There is no formal proof here that Eq. 14 may be correctly identified as the minimum and, in fact, it might be better stated that the correct minimum is at least as small as Eq. 14. The argument is plausible by analogy with laminar flow of some fluid in a pipe, in which case the entropy of the fluid would be a minimum. Departures from laminar flow would result in entropy production, and hence a reduced capability of the stream to do useful work. Departures from the defined laminar ensemble would produce local turbulences, or turbulences some other place in the system (that is, not necessarily at the point where the departure from laminar flow takes place) such that the aggregate travel time would be increased over what it would have been for the laminar case, notwithstanding any local decrease in travel time. This concept, although suggested directly by the Second Principle of Thermodynamics, is highly speculative.

(Whether or not this concept of aggregate travel time increasing with departures from laminar ensembles always holds is a matter of speculation. Seemingly, it would hold for high densities of traffic, but it might hold for low densities. The implications here are quite broad if this concept is ever elevated to the status of a "law" or "principle". For example, a passing or lane-changing maneuver is one form of departure from laminar flow. The concept would then assert that even though the particular vehicle changing lanes in passing achieves a reduction in its travel time, somewhere in the system there is at least an equal increase in travel time. Clearly, if there are no vehicles in the lane into which the passing vehicle moves, there would be no travel time decrement anywhere, and the concept would be violated. On the other hand, if the density were sufficiently high, the likelihood would be low that there would be no cars in the adjacent lane, and the concept could hold. Similar "Second Principle" type of arguments could be offered for ramp locations, multi-axle vehicles, and other factors causing local disturbances in the ensemble.)

In Figure 4, point m is established by Eq. 14 for a given N (ENS), and in turn establishes the minimum travel time of the Ith car, em, for this size ensemble. The minimum travel time is comprised of ed, which is the limiting value for the hypothesized min RSP, and dm, which is due to the size of the ENS (I). Depending on the distance between INB and OUB, there could be some non-zero N (ENS) for which m would coincide with d. However, this case is not of interest here.

It can be seen that dm is the minimum diseconomy for a jointly specified N (ENS) and RSP (OUB). It can be varied by changing N (ENS) as, for example, by diverting some traffic from the system of interest, or by changing RSP (OUB) as, for example, by adding more freeway lanes downstream of OUB. Such changes involve operational decisions which essentially are external to the system of interest. Thus because dm is amenable to change only by external (to the system) decisions, it will be referred to as the "external diseconomy". The remainder of the total diseconomy db is mb and can be due only to factors "within" the system, and accordingly will be referred to as the "within diseconomy". This differs from the earlier statement in this paper wherein point c in Figure 4 was asserted as defining the "within" diseconomy. Because point c can never fall below point m, then cb can never be greater than mb. Or, for the most part cb will not represent the entire "within" diseconomy. It nevertheless represents a gross, first approximation type of warrant as to the need for operational decisions that directly concern the system of interest.

FIELD METHODS

The field data inputs required for the various sub-studies all stem from a single set of field measurements comprised of: (1) license plate identifications at various input and output boundaries of the system, and (2) performance of the traffic stream at a given output boundary. A parallel requirement was that a reference framework of clock time be vigorously coordinated among all of the sites where data were to be collected. Finally, there was a somewhat unusual need for safety precautions as a consequence of the location of sites at which measurements were to be made.

License Identification

License identifications were required for three separate purposes: (1) to obtain travel time for vehicles to clear the network via different combinations of input and output boundaries, (2) to estimate the relative volumes of traffic moving over the different flow paths, and (3) to identify the "ensembles". These multi-purpose uses justified using somewhat more detailed field methods than would normally be needed to obtain licenses. In fact, in certain aspects, redundant identifications were deliberately obtained at sites where identifications had to be very precise.

Two methods were used to obtain licenses: (1) direct visual observation followed by voice dictation into tape recorders, and (2) photography. The tape recorder method was used for on-ramp and off-ramp sites; the photographic method was used for primary identification of licenses on the freeway proper. However, dictation methods were used redundantly on the photographic sites to correlate time-of-day with the time at the other sites in the network. They were also used (and again redundantly) to obtain a check on respeed at the output boundary. Detailed descriptions of the two methods follow.

Dictation Method.—In this method, the observer dictated the license numbers of passing vehicles directly into the microphone of a tape (or wire) recorder. Alphabetic characters were dictated with the phonetic alphabet; for example, "able" for a letter "A", etc; out-of-state cars were identified as such by the observer dictating "out-of-state" into the record; and all other pertinent information bearing on the study, location of the observer, his name, etc., were also dictated into the record. In this manner a relatively large amount of information could be readily and compactly stored in the field for later transcription and analysis in the office.

Portable, transitorized tape recorders were used at the relatively inaccessible ramp sites, highlighting another advantage in the dictation method; namely, the absence of an AC power requirement. This afforded the investigation an unusual degree of flexibility in placing observers at strategic points throughout the network.

Possibly the most important advantage of the dictation method was that time-of-day at which vehicles arrived at the recording sites could be established with a high degree of accuracy. It was found in a pilot study that high-quality tape recorders maintained essentially constant tape speed, even when battery-powered. (The special pilot study on tape recorder methods was conducted to determine the order of magnitude of the discrepancies in measurements of time-of-day that could be expected in using this technique. A total of 107 randomly arriving licenses were dictated into one of the machines in a 10-min period calibrated with five stop watches. In eleven later playbacks of 10-min recording, the maximum discrepancy for the known 10-min interval was 20 sec; the average was 5.5 sec. The median discrepancy in the time of arrival for a given license was found to be 3 sec. Notwithstanding the fact that the results pertained only to the machine tested, the order of magnitude of the discrepancies appeared to be well within the requirements of the study, particularly because time reference checks were to be dictated at known 5-min intervals). Accordingly, the recorders were run continuously once a study was under way. At 5-min intervals, the observer, referring to his watch, dictated the time-of-day into the microphone. In the later playback of the tapes in the laboratory, it was possible to establish the real time of arrival of a vehicle by relating the audible message of its license to the audible message of the real time reference points. To do this the data were transcribed in several runs. In the first run, the transcriber concentrated solely on the license numbers. In the second, he started a stop watch the instant he heard the first timing reference point, and allowed the watch to run for the remainder of the study with the tape recorder running simultaneously. He then established the time of arrival of a vehicle by relating the auditory license message to the visual stop watch reading, and entering the watch reading next to the already entered license number. For heavy traffic conditions, a third transcribing run was used to check the accuracy of both the license and its time of arrival.

Finally, an adjusted time of arrival was established by linear interpolation of the transcribed time of arrival between the dictated time reference points. As an example, if the auditory messages indicated that exactly 5 min had elapsed between two timing reference points, while the stop watch showed 5 min and 10 sec (indicating that the tape speed was slower in the transcription process than it had been in the dictation process in the field) the 10-sec discrepancy would be distributed throughout the 5-min period: every time-of-arrival in the first minute would be reduced by 2 sec, in the second minute by 4 sec, etc.

It is difficult to estimate precisely the accuracy with which the time-of-day was established with these techniques. That there was a high degree of accuracy is suggested by the fact that independent transcribers produced time-of-arrivals which rarely differed more than 3 sec from values for the same arrival (that is, vehicle) produced by other independent transcriptions of the same dictated record. However, there is no way of knowing whether or not field observers dictated licenses of vehicles at precisely the same point, notwithstanding the fact that they were instructed to do so. The best estimate of accuracy, taking all factors into account, but which nonetheless must be considered as speculative, is of the order of 10 sec.

Photographic Method.—This method involved photographing licenses of passing vehicles with pulse-type cameras from overhead structures. Aside from the advantages inherent in photography (permanent visual record of license, visual display of clock time, automatic actuation, and others), the method presented difficult problems, and was used only after pilot studies indicated that problems associated with using dictation methods at multi-lane freeway sites would be more difficult to overcome. (Pilot studies indicated that dictation methods would not be too satisfactory for multi-lane freeway traffic. If an observer were located on an overhead structure, it was quite difficult for him to read and then dictate the licenses of a large sample of the vehicles passing below him because of distance and his angle of view, particularly when vehicles were moving at high speeds. It was found that the number of vehicles licenses a trained observer could pick out of a fast-moving stream of freeway traffic was not sufficiently large for flow pattern work (although more than adequate for travel time measurements alone). Placing observers on median strips or on the shoulders to reduce the sight distance and provide a better angle of view was ruled out because of danger to the observers, and because of the deleterious effect that their presence would have on the traffic flow. This effect was observed in some work reported earlier (11), so that as a general policy, observers were to be kept out of view of the passing motorists as much as possible.)

The difficult problems in photographing licenses of fast-moving vehicles so that the separate license characters could later be read on magnifying film readers related to an entire complex of factors: film speed and grain, camera angle relative to angle of inclination of license plate which in turn depended in large measure on grade of the road and whether the front or rear plate was being photographed, changing ambient light, glare reflected off the plate, or shadow falling across the plate, and others. These will not be described in detail here; the net effect however was to limit the coverage of single cameras to single lanes at the all-important output boundary.

The primary camera system in the study used Kodak Cine Special 16-mm movie cameras that were set for single-frame operation. (When a movie camera is used as a pulse camera, as in this study, its shutter never reaches full speed due to starting inertia. Consequently, the smallest shutter opening is not sufficiently small to give an exposure fast enough to resolve the license detail of a fast-moving car. A special adaptor had to be designed that gave the camera a shutter speed of approximately 1/400 sec.) In 16-mm film, there are 40 frames per foot, so that the Cine with its 100-ft magazine has a capacity of recording licenses of approximately 4,000 cars without reloading. This was the principal reason for selecting this camera for freeway work. However, there was a major disadvantage; namely, with this size of film, there are significant problems of image size which necessitated using a separate camera for each lane.

The cameras were triggered manually; that is, by an observer watching for the instant a car came into the camera field. (In the new photographic system under development at ITTE, cameras will be triggered electrically by car wheels rolling over pressure-actuated detectors.) Prior to the beginning of the study, and with traffic temporarily by-passed around the given lane, two strips of adhesive tape were placed on the road at the beginning and end of the field of view of the camera with the lens properly focused. The observer thus was able to determine when a vehicle was in the field of view (after the license crossed the first lane and before it crossed the second). This eliminated having him continuously looking through the camera viewfinder which is extremely fatiguing.

A total of seven cameras were simultaneously used in this study. In two cases, a small watch movement was placed in the field of view, and brought into focus by a system of lenses external to the camera. (This system was designed and operated by Stephen Craig, photographic consultant to the project.) The time base for the other cameras was provided by superimposing (redundantly) independent dictation of licenses into tape recorders at the same site. These time-identified licenses were later matched with the photographed licenses to establish time reference points for the remaining (unmatched) photographed licenses. Times-of-day for the unmatched licenses were then established by linear interpolation between the nearest reference points. Although only approximately 25 percent of the licenses could be obtained via dictation (as contrasted with virtually 100 percent with the photography), there were sufficient time-identified licenses (and hence reference points) to establish time to within an estimated 5 sec.

ITTE is now developing a 35-mm camera system specifically for photographing licenses of fast-moving vehicles on a freeway. The system is being designed around a Robot Recorder - E 35-mm camera having a film magazine capacity of 200 ft, or approximately 2,400 frames. In contrast with 16-mm film, the larger size image will permit a single exposure to cover two, or possibly more lanes of traffic. A special collimating prism adaptor having its own light source, has been designed that superimposes an instrument panel on the image of the vehicle. A watch and other instrument displays can be included in the panel. The system was tested in this study, and although it performed satisfactorily, it was not sufficiently reliable to be included as a part of the basic instrumentation. Once the system is fully developed, it should significantly simplify the task of recording licenses and time of fast-moving vehicles.

Properties of Traffic Stream at Output Boundary

An important study objective was the identification of traffic factors "within" the system that were influencing travel time from those occurring "downstream"; that is, beyond the output boundary. This was accomplished by a combination of a set of speed-headway measurements and a classified count of traffic, both at the output boundary, and both carefully controlled so as to be on the same clock time base. Ideally, these control measurements should have been made over the entire output boundary. In this study, they were limited to the output boundary at site 8 (the Hollywood Freeway) because of the amount of equipment required.

Cross-Coupling of Recorders. —Two pairs of Esterline-Angus 20-Pen Recorders were located at site 8 (the Grand Avenue Overcrossing overlooking the Hollywood Freeway). The first pair was used for respeed-headway measurements; the second for classified volume counting. In a given pair, one recorder would be in operation while the other was on stand-by. The set of signal inputs to a given pair was via an external control unit. By throwing a single selector switch on the external control unit, the complete set of signal inputs could be transferred from the operating recorder to the stand-by recorder, with the operating recorder being simultaneously changed to stand-by while the stand-by unit went into operation. The reason for this provision was that for accurate respeed measurement, it was necessary to have the paper at maximum travel speed or 3 in. per second; a roll of paper would be used in approximately six min. The switching arrangement allowed an instantaneous shift from one recorder to the other without losing any data. While the second unit was in operation, a fresh roll of paper would be inserted in the first recorder which would then be ready to be switched on when paper ran out on the second unit. The arrangement also permitted switching recorders when any common recorder malfunctions occurred—paper jamming, pens running dry, etc.

Speed and Cross-Reference Truck Classification. —Two pressure-type, vehicle-actuated detectors, similar in some respects to those reported by Mathewson, Brenner, and Reiss (12), were installed directly on the roadway, parallel to each other and 4 ft apart. As the wheel of a vehicle rolled over a detector, mechanically separated contacts would be closed and a pen on the recorder energized. By calibrating the paper speed of the recorder, it was possible to measure the time gap between successive pips and accordingly establish the vehicle speed (actually respeed), headway, wheel base, and several interrelated functions. With three lanes of traffic at site 8, it was mandatory that the electrical signal coming with the closure of the contacts in any detector in any lane be completely independent of the signal coming with actuation of any other detector. A multi-lane detector was accordingly designed and built that met this requirement of independent lane-by-lane detection of traffic. There were, therefore, six pens assigned to traffic detection.

A parallel requirement to speed-headway measurements by lane was to identify these measurements by type of vehicle. This identification, which was needed to measure the "truck" effect as well as the "compact car" effect reported by Burch (13), could be accomplished to a certain extent by studying the pip pattern. For example, a three-axle truck would produce a definitive array of three pips on one channel (corresponding to the first of the parallel detectors) translated from three pips on a parallel channel (the second detector). However, under high density conditions, ambiguous patterns occur quite frequently. As an example, a pattern for a two-axle, long wheel-base vehicle (specifically, busses), ahead of a five-axle truck trailer combination, is easily interpreted as a three-axle truck followed by two compact cars, etc.

Accordingly, an independent, direct input was designed whereby an observer actuated two sets of buttons on a control panel, with each button energizing a separate pen on the recorder. There were three buttons in the first set, one for each lane. In the second set, there were five buttons, one each for: a truck-full-trailer combination, a truck-semi-trailer combination, a three-axle truck, a two-axle truck, and a bus. The observer depressed the proper lane button and the proper truck classification button as he saw the first wheel of the particular vehicle cross the first detector in the given lane.

The complete pen use is shown in Figure 7 which is a facsimile of an actual record obtained in this speed measurement cross referenced to vehicle classification. A ninth button on the control panel (actuating pen No. 20) was used for time calibration purposes, being depressed every minute (on the minute) by the observer who for this purpose was provided with a continuously running sweep second watch.

The procedures followed in reducing these data, the subsequent computer analyses, and specimen results are described later in this paper.

Classified Volume Counting. —The second pair of Esterline-Angus recorders (also at site 8) were used for classified counting of multi-axle vehicles, and two-axle trucks and busses. As with the speed measurement use of the recorders, observers depressed buttons on a control panel for lane indication. The type of vehicle, however, was indicated by multi-counts on an indicator button on the control panel; for example, one count signified a bus or two-axle truck, two counts indicated a truck-semi-trailer combination, three counts indicated a truck-full-trailer combination.

There were two reasons for using the Esterline-Angus equipment for this volume counting. The first was to reference the presence of multi-axle vehicles, two-axle trucks and busses on the clock time scale. This was redundant with the classification being accomplished in the speed measurement system at the same time. The second reason was to have on hand a stand-by pair of Esterline-Angus recorders ready to be connected into the speed measuring system in the event of failure of either one of the

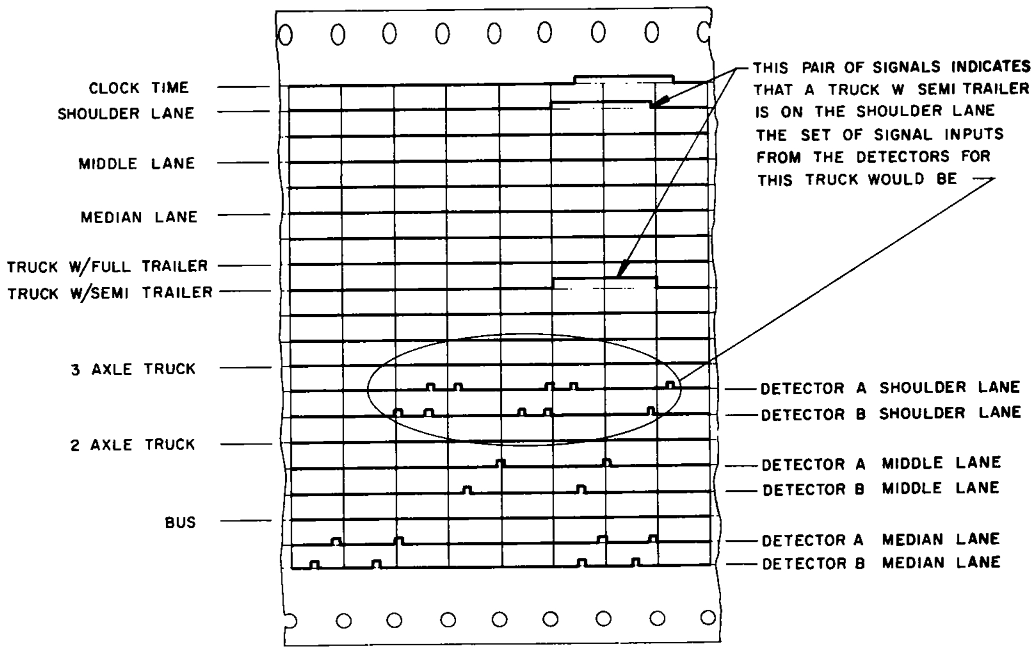


Figure 7. Facsimile of Esterline-Angus record for speed and cross-referenced truck classification.

pair originally set up for speed measurement. To meet this latter eventuality, both pairs of recorders with their control panels, interconnecting circuitry connectors, etc., were wired identically, and hence were completely interchangeable. However, the pair used for the classified volume counting was set to operate at a much slower paper speed, 1 ft per minute as compared to 3 in. per second. The low speed, which could be instantaneously switched to high speed, was used for the classified volume counting because there was no need to time the gap between successive pips.

Redundant "RSP" Measurement.—In view of the importance of the RSP measurement at the output boundary, provisions were made in the experimental plan for its being measured redundantly. The first (and primary) method used the previously described equipment system (multi-lane detectors, Esterline-Angus Recorders, etc.). The second method required that licenses be identified (on a sampling basis) at some secondary point downstream of the output boundary. Matching licenses identified at the secondary point with those identified at the output boundary would establish travel time between the output boundary and the secondary point. With the intervening distance known, an RSP pertaining to the section of freeway immediately downstream of the section of interest could be computed. This RSP would not necessarily have the same value as the RSP measured over the 4-ft distance at the output boundary, and although it probably would not be as closely correlated with system performance as would be the spot RSP, it nevertheless would provide an order of magnitude indication of downstream effects.

For site 7, the downstream secondary control points were sites 9 and 10 which happened to be required for separating HRF and PSF traffic. It later developed that enough detectors could not be provided in time for the study to equip both sites 7 and 8 for RSP measurement. The decision was made to limit the RSP measurement to site 8. Thus the secondary RSP measurement (via the travel time between site 7 and sites 9, 10) was the only RSP measurement at site 7.

For site 8, the downstream secondary control point was at site 11 (Edgeware Overcrossing) which is approximately ½ mi downstream (west) of site 8. (There are no intervening overcrossings.) But, aside from providing for the redundant RSP measurement, the license identification at site 11 was used to test the power of the license matching technique for establishing flow patterns with low sampling ratios.

General Network Control

An overriding problem was to synchronize the clock time base for all measurements (dictation of licenses, photographing licenses, RSP measurements, classified volume counting) throughout the network. This was done in part by assembling all observers at the start of the day in a briefing period, and having each synchronize his watch with an electric clock mounted permanently in the assembly area. Thereafter during the course of the day, repeated time checks were made over a special radio network provided through the courtesy of the Los Angeles Police Department. Five police walky-talky ratios were loaned to the project, and were set on an infrequently used police channel. The radios were then distributed to personnel in charge of the most widely dispersed sites (sites 1, 2, 8, 9, and 11).

Radio communication of this nature was considered to be mandatory, not only because of the time check requirement, but also (and more important) because of safety considerations. Several of the sites were essentially inaccessible except at high risk due to the high-speed traffic. Police protection was required to get field personnel into and out of these sites. Personnel were relatively safe once they were at their sites, but there always was the possibility of an accident. The radio network was made available to communicate any untoward happenings to the field director. Several motorcycle officers and other police officers in squad cars were on a stand-by basis through the course of the day.

SPECTRUM OF TRIP LENGTHS ACCORDING TO ON-RAMP OF ORIGINATION

Increasing consideration has been given in recent years to determining the origin and destination of traffic on modern freeways and data are available on the vehicle-miles being driven on freeways. However, there has been relatively little work done on apportioning this mileage to specific on-ramps or off-ramps, although such information would serve to demonstrate in part whether a ramp was being used principally for long-trip or for short-trip purposes. Limited-access facilities are not intended for short-trip purposes, so that if the short-trip use of a particular ramp is sufficiently high, a decision to close the ramp might readily be made without further analysis as to any detrimental effects the short-trip use would have on the long-trip freeway user. There, of course, would have to be prior analyses as to suitable alternate ramps and surface street paths to them. Therefore, an immediate screening type of study would be to ascertain the spectrum of trip lengths on the freeway proper originating at the different on-ramps of interest.

The conventional roadside-interview type of survey undoubtedly would be adequate for determining the length of freeway trips originating at a given on-ramp (or terminating at a given off-ramp) provided the volume of ramp traffic was not too high. In this particular study, however, the three ramps of interest carry considerable traffic during peak hours; the traffic back-up that would have resulted from roadside interviews would have created intolerable inconveniences to the drivers, and would also have resulted in incorrect travel times. Consequently, a somewhat novel questionnaire technique was developed. Although the technique was used in on-ramp situations in this study, it can be applied to situations on the freeways proper.

Licenses of vehicles coming on to the ramp were recorded by voice dictation. It was not necessary to stop the vehicle or impede its travel in any way. In fact, observers were hidden from the motorists' direct view as much as possible. Later, the licenses were taken to the Department of Motor Vehicles which in turn supplied the addresses of the registered owners to whom a questionnaire was to be mailed.

The mailing to the registered owners consisted of a letter explaining the study objectives, a special map type of questionnaire, and a self-addressed, stamped, business reply envelope. Specimen cover letters and questionnaires are shown in Figures 8 and 9, respectively. The information sought in the questionnaire related to: (1) freeway trip length of the ramp user, (2) the specific freeway to which the ramp user was

Institute of Transportation and Traffic Engineering
UNIVERSITY OF CALIFORNIA
LOS ANGELES 24, CALIFORNIA

Registered Owner of
Vehicle License No. ____________

Dear Sir:

We are a research group in the Institute of Transportation and Traffic Engineering of the University of California, and need and earnestly seek your help in a special study we are conducting on freeway operation in the Los Angeles area.

Our problem is to find out how the freeways are being used, where cars get on and leave the freeways, the city streets they use to get to the freeway. Our method is to record licenses of vehicles we observe on different freeway ramps throughout the city. We then write directly to the registered owners at addresses we obtain from the license records of the Department of Motor Vehicles of the State. This is how we obtained your address. Specifically, a vehicle registered in your name was observed:

DATE:

TIME:

PLACE:

We ask you to fill in the enclosed questionnaire and mail it back to us immediately in the addressed envelope. If you were not, but know who uas driving the vehicle we observed, would you still complete the form, please?

You will note that we do not ask you to sign the questionnaire or otherwise identify yourself. We do this on purpose to insure that your privacy will be respected.

May we thank you in advance for your needed cooperation in this work that is so important to all of us; the information you supply will no doubt result in long-range benefits to all freeway users.

Yours truly,

Robert Brenner

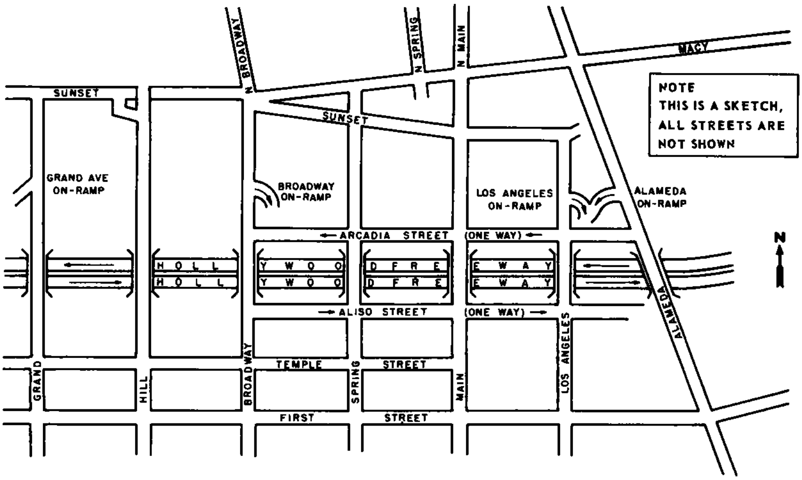
Robert Brenner, Project Engineer
The Institute of Transportation and
Traffic Engineering

Encl.

Figure 8. Cover letter used in mailing questionnaire study.

INSTITUTE OF TRANSPORTATION AND TRAFFIC ENGINEERING
UNIVERSITY OF CALIFORNIA LOS ANGELES 24 CALIFORNIA

FREEWAY RESEARCH PROJECT



1 YOUR CAR WAS OBSERVED HERE ON THE ________________ ON RAMP ON MAY ____

2 DRAW A LINE DIRECTLY ON THIS MAP SHOWING HOW YOU GOT TO THIS RAMP

3 ON THAT DAY, WAS YOUR VEHICLE HEADING FOR (MARK ONE)

HARBOR FREEWAY? ☐ HOLLYWOOD FREEWAY? ☐ PASADENA FREEWAY? ☐

4 AT WHAT RAMP DID YOUR VEHICLE LEAVE THE FREEWAY? ________________

AS SOON AS YOU HAVE COMPLETED THIS FORM PLEASE RETURN IT TO US IN THE ENVELOPE WE HAVE SUPPLIED IF YOU HAVE ANY QUESTIONS, YOU MAY PHONE US AT BRADSHAW 2-6161 EXTENSION 551 THANK YOU FOR YOUR COOPERATION

Figure 9. The mailing questionnaire.

headed, and (3) the surface street path followed in getting to the ramp. The discussion here is limited to trip length; the latter two classes of results are treated in the next sub-study.

At the outset, it was recognized that the questionnaires would have to be in the mails as quickly as possible if there was to be a satisfactory return and if the results were to be reliable. Consequently, extraordinary measures were taken to assure that the questionnaires would be posted within 24 hr after the vehicles were observed. In fact, the study was deliberately scheduled (for a Tuesday) so that there would be no weekend intervening between the date the vehicle was observed and the date the registered owner received the questionnaire. The transcription of licenses from dictated records, cross-checking, and alphabetic-numeric sorting (required for locating addresses) began within 3 hr after the field work was completed and continued throughout the night of the study. The addresses of the registered owners were on hand by noon of the following day. Addressing of letters and envelopes was completed by 5 p.m., and the questionnaire in the mails by 6 p.m.

The initial listing was limited to licenses observed between 4 and 4:30 p.m., and was comprised of 200 licenses at the Broadway on-ramp, and 100 each at the Los Angeles and Alameda on-ramps. The sampling ratios (licenses used for mailing vs total volume) were 99 percent at BON, 45 percent at LON, and 52 percent at AON. The sampling was heavier at BON because this was the on-ramp of primary operational interest.

Addresses could not be located for 36 of the original listing of 400 licenses, so that the mailing went out to 364 persons. Of these, there were in all 213 usable replies, and 17 non-usable (person had moved; car was leased; car sold new, registered owner unknown; etc.). The useful reply percentage (of the entire mailing) was approximately 59 percent.

The gratifyingly high percentage return can be ascribed to any of a number of factors: speed in getting the questionnaire into the mail, wording of the cover letter, assurance of anonymity, respective prestige values of the names of the University and the Division of Highways, the fact that the communication was personal, novelty, etc. There were, however, numerous additional commentaries in the replies: detailed descriptions of routes, reasons why particular routes are selected, offers to be of further assistance, even a few discussions of "poor" ramp design. These unsolicited commentaries coupled with the high percentage of returns indicate a high degree of public interest in research projects on freeway operations. This, of course, could also explain the high return.

The results shown in Figure 10 are the spectra of trip lengths, and Figure 11, the cumulative frequency distribution of trip lengths. The median trip lengths are BON—5.1 mi, LON—7.0 mi, AON—8.4 mi. The 75th percentiles are BON—2.6 mi, LON—4.4 mi, AON—4.4 mi.

Any judgment as to what constitutes a "short" trip on a freeway would be completely arbitrary. An equally arbitrary judgment would bear on how high the percentage would have to be before being considered to be excessive, how low to be acceptable, etc. There are no norms for either of these judgments. To the authors, it appears that use of all three ramps during the period sampled is compatible with the intent in freeway construction. There is limited indication of excessive short-trip use, certainly not to any degree that would preclude more detailed analysis of the total network problems prior to any decision to close any one or all of the ramps.

Another way of interpreting the results is that, were any of the three ramps to be closed, not many of the motorists presently using these ramps would be dissuaded from using the freeways, albeit by entering via another on-ramp. Closing the ramps would thus raise problems of how the diversion of traffic from one on-ramp to another would affect the intervening surface street network. This is the subject of the next sub-study.

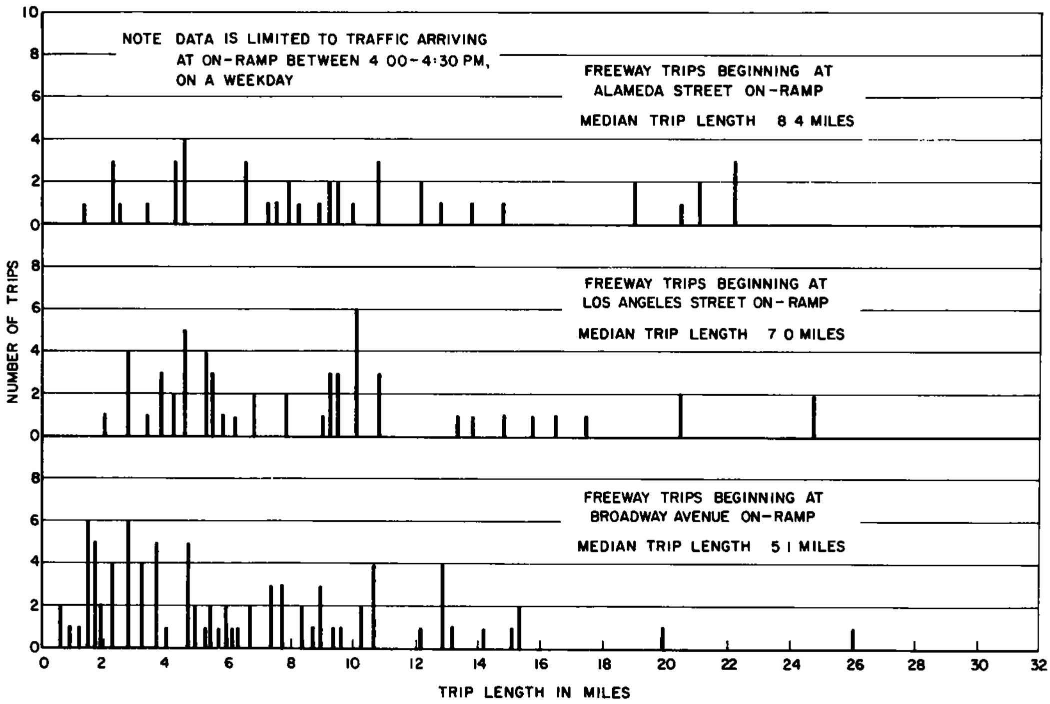


Figure 10. Distribution of trip lengths as obtained from mailing questionnaire.

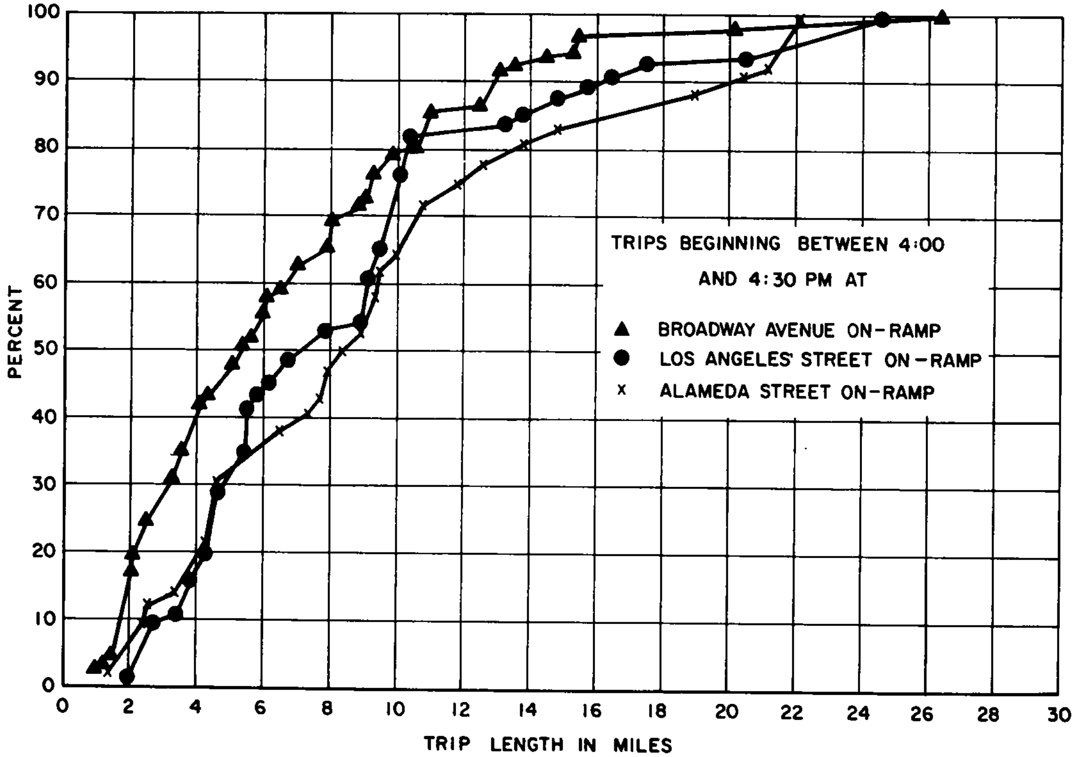


Figure 11. Cumulative distribution of trip lengths over all freeways.

SURFACE STREET TRAVEL PATTERNS OF RAMP USERS

The questionnaire described in the previous sub-study was devised in the form of a schematic map of the surface streets specifically to yield at once both the surface street paths and a general idea of the point of origin of ramp users. Because the questionnaire was sent only to ramp users, the information was automatically keyed to specific on-ramps. Also, keyed to the on-ramp use would be the freeway to which the respondent was heading, this being one of the questionnaire items. It was considered mandatory to have all of this information for subsequent analysis of alternate ramps and surface street routes that would be used in the event of ramp closings, or opening of new freeway links.

It was recognized at the outset that there would be problems in evaluating the reliability of the data, because no completely suitable statistical method could be identified. Consequently, a somewhat oblique split-half reliability procedure was followed. The replies were arranged in nine groups, one for each combination of on-ramp and freeway (destination). Through use of random number tables, each group was divided into two samples. The samples were then tabulated separately and the totals compared with each other. Although the data actually identify the point of origin of the traffic and the complete surface street pattern followed in getting to the on-ramps, the statistical analysis was limited to treatment of the cardinal directions from which traffic had come upon reaching the on-ramps. The data are given in Table 1, and are shown schematically in Figure 12. To simplify the presentation here, these data are limited to cardinal directions. The actual data (questionnaire replies) show the complete surface street route traffic followed in getting to the different ramps.

The sole case where there appears to be any inconsistency between the "A" and "B" samples is for the BON to HLF case. The null hypothesis was tested for this case using a chi square technique, and was not rejected. (The experimental chi square value

TABLE 1

CARDINAL DIRECTIONS FROM WHICH RAMP USERS APPROACH THE RAMP

			Direction of Approach Via Surface Streets, From the:		
On-Ramp	To Freeway	Sample	North	East	South
Broadway (BON)	Hollywood (HLF)	A	3	12	1) 31
		B	3	6	6)
	Harbor (HRF)	A	1	16	7) 47
		B	2	16	5)
	Pasadena (PSF)	A	1	4	1) 12
		B	3	2	1)
Los Angeles (LON)	Hollywood (HLF)	A	1	-	21) 42
		B	1	-	19)
	Harbor (HRF)	A	4	-	-) 9
		B	3	-	2)
	Pasadena (PSF)	A	1	-	-) 2
		B	1	-	-)
Alameda (AON)	Hollywood (HLF)	A	9	-	4) 24
		B	9	-	2)
	Harbor (HRF)	A	5	-	4) 16
		B	3	-	4)
	Pasadena (PSF)	A	1	-	-) 3
		B	2	-	-)

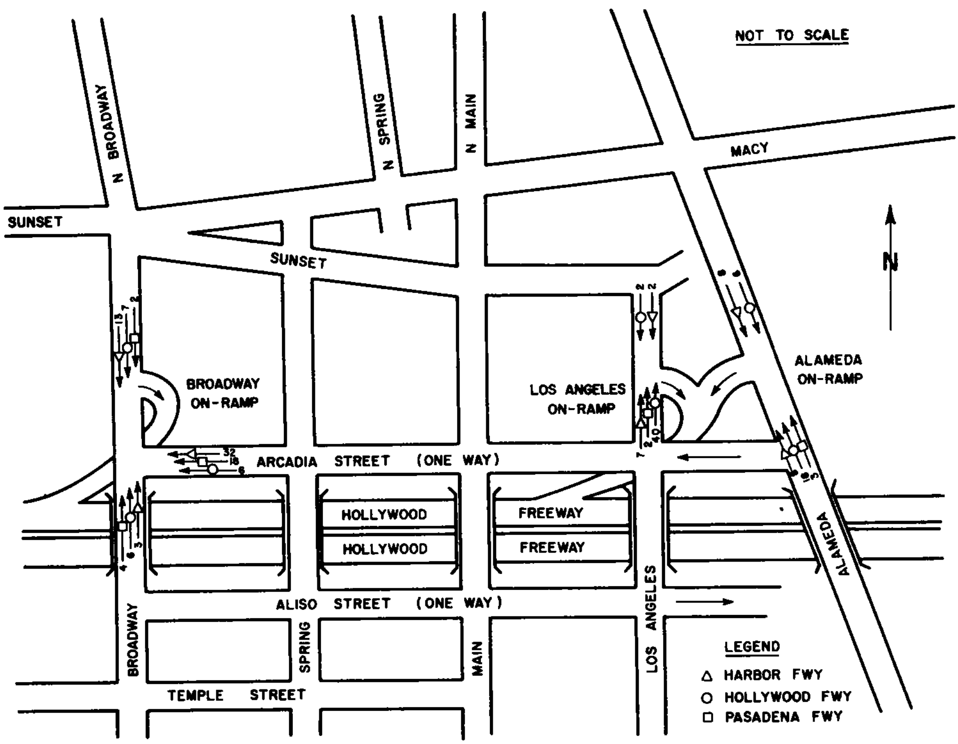


Figure 12. Cardinal directions from which traffic came in getting to on-ramps of interest.

uncorrected for continuity is 5.533. The chi square, 2 DF values at the 10 and 5 percent levels of significance are 4.605 and 5.991, respectively.) This may be interpreted as meaning that although the A and B patterns differ, the discrepancies are not sufficiently large to be ascribed to anything other than chance. A somewhat heuristic interpretation of the discrepancy would be that the variability of the BON - HLF pattern is high, which in turn would imply that more data would have to be taken to lessen the uncertainty as to this particular flow pattern.

The results, in the main, demonstrate a high degree of self consistency, and show that the principal flow to the on-ramp of interest is from the south. In the case of the LON - HLF pattern, 18 of the 21, and 15 of the 19 northbound vehicles (coming from the south), originated south of First Street. This result (which comes from detailed tabulations of travel patterns which are not presented in this paper) demonstrates the potential power of the questionnaire technique for determining more precise surface street patterns.

The principal reason for determining the surface street paths (which essentially was the principal reason for the questionnaire itself) was to establish a basis for estimating the effect that closing a particular on-ramp would have on its present users. There are questions of: (1) will the present ramp user continue to use the freeway, albeit by taking a different on-ramp; (2) under these circumstances, which ramp will be chosen; (3) what new route will be taken on the surface streets to get to his new on-ramp. Inferential judgments such as these are inherently difficult, and can never be made with certainty. But they can be made with a greater degree of confidence if the present points of origin within the surface street network are known for the ramp users along with the actual routes they take.

To estimate the surface street patterns that would prevail after projected ramp changes took place, a secondary questionnaire was constructed specifically directed toward traffic experts and others who were knowledgeable of traffic conditions in the downtown Los Angeles area. The experts were from three independent organizations: (1) Traffic Department, City of Los Angeles; (2) Police Department, City of Los Angeles; and (3) Traffic Department of District VII, California Division of Highways. Selection of the experts who were to receive the questionnaire was left to the heads of the separate organizations.

The "expert" questionnaire was constructed directly from the replies received from the general "mailing" questionnaire. This is to say that the routes the motorists indicated they were taking to get to the specific on-ramps were identified in the "expert" questionnaire. Each route was identified by a sequence of intersection numbers which were assigned arbitrarily to every intersection in the adjacent downtown area. The routes (sequence of intersection numbers) were further cross referenced to on-ramps, there being in all 60 route-ramp combinations. Illustrative of the instructions to the experts:

> "...Judge as best as you can how a vehicle would change from the given route if the ramp for which it was originally heading were closed. List the new ramp and the new route in the space provided next to the given route, and in the same manner, that is by the intersection numbers...."

To minimize any bias the "mailing" questionnaire might have on the "expert" questionnaire, the number of vehicles following each route was not disclosed to the experts. It was recognized that this might result in some aspect of the diversion pattern being inconsistent with traffic conditions. However, the investigators judged that if such an inconsistency did arise, they could reconcile it.

As was anticipated, the replies were consistent with each with a few exceptions, and from the results of this and the mailing questionnaire, the ramp diversions were deduced as given in Table 2. The diversion pattern appears to be reasonable with the exception of the ramp traffic headed for the Harbor Freeway. The present load and resulting back-up of traffic on the collector road to the Harbor Freeway during the evening break is so high, that the 210 vehicles shown as going to the Harbor Freeway (having been diverted from the Broadway on-ramp) would probably get to the Harbor Freeway via a ramp (3rd Street) which is not a part of the system of interest in this study.

TABLE 2

ADDITIONAL TRAFFIC VOLUME ON ADJACENT ON-RAMPS WITH THE CLOSING OF THE BROADWAY ON-RAMP
(Figures Pertain to 1-hr Volume 4-5 p.m.)

Ramp	Volume with Broadway On-Ramp Open	Vehicles Diverted From the (Closed) Broadway On-Ramp	Total Volume When Broadway On-Ramp is Closed
Broadway Avenue On-Ramp	620	0	0
Grand Avenue On-Ramp to:			
Hollywood Fwy	745	150	895
Harbor Fwy	300	210	510
Pasadena Fwy	240	15	255
Castellar Street On-Ramp	1,670	70	1,740
Alameda Street On-Ramp	540	95	635
Los Angeles Street On-Ramp	385	80	465
		620	

PROPERTIES OF TRAFFIC STREAM AT OUTPUT BOUNDARY OF SYSTEM

If the input and output boundaries of the freeway system of interest were placed at locations where entering and exit speeds, respectively, were at the maximum legal limit (specifically, 65 mph in California), performance measured at the boundaries could be said to be due to factors "within" the boundaries. In this study, however, the boundaries are defined so as to include only the section of freeway that is of operational interest, and at these locations, speeds are anything but maximum during the period of the study. Performance measured at these boundaries must accordingly include effects due to the freeway network downstream (that is, beyond) the OUB, effects due to factors upstream (that is, ahead) of the INB, and finally effects which could be said to be due primarily to factors within the boundaries. It is these latter effects with which the project is concerned, and a general objective is to isolate them from the downstream and upstream effects. Upstream factors are removed from the analysis by having the MOE measured for the "collection" defined on the basis of clock time at the INB. Downstream factors are taken care of in some measure by relatively precise measurement of traffic at the OUB. This measurement is the subject of this particular sub-study.

The field techniques used in the speed-headway measurements at the output boundary have already been described as involving multi-lane detectors in combination with the Esterline-Angus recorders, and the resulting Esterline-Angus record is shown in Figure 7. Figure 13 shows a facsimile of the Esterline-Angus record in sufficient additional detail to facilitate descriptions of the data reduction procedures and subsequent computer analysis.

Data Reduction Procedures

There were three primary "times" to be identified for each vehicle. These are, referring to Figure 13:

A_{2I-1} = time the first wheel of the vehicle crossed the first detector;

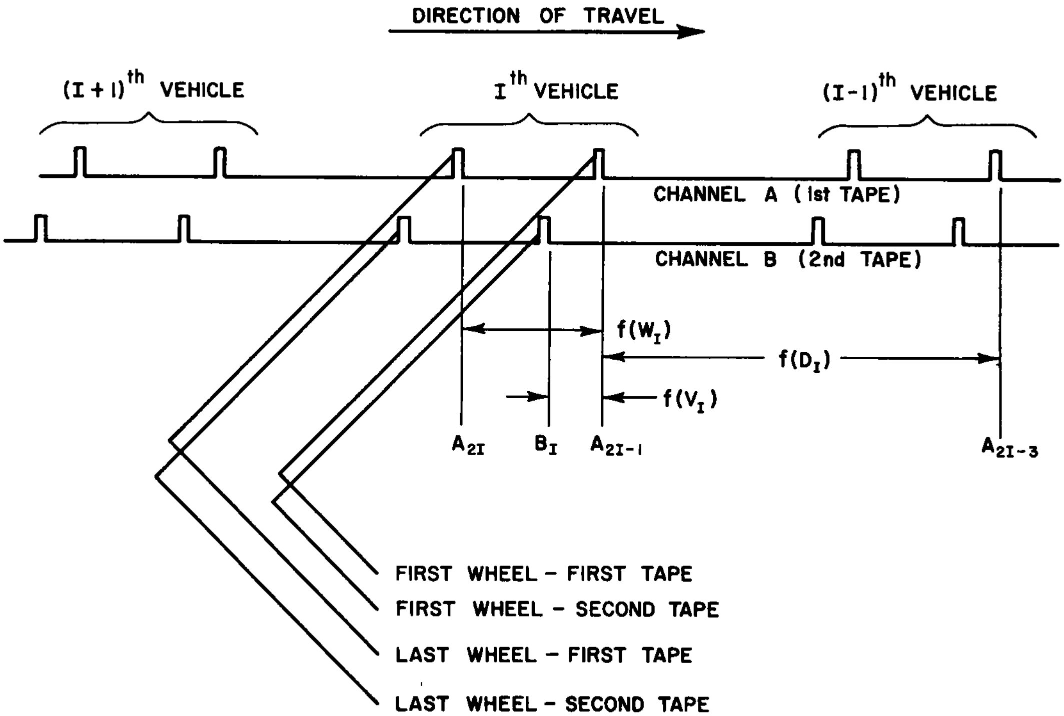


Figure 13. Nomenclature used in speed and headway study.

A_{2I} = time the last wheel of the vehicle (or truck-trailer combination) crossed the first detector; and

B_I = time the first wheel of the vehicle crossed the second detector.

A fourth "time", B'_I, was used only to demonstrate that this technique could yield an acceleration measurement.

Inasmuch as one channel (number 20) of the chart record was used as an independent clock time indication, it was possible to convert each of the "time" measurements (A_{2I-1}, A_{2I}, B_I) to "clock time" measurements by relating them to the nearest time indication. The time indications were made every minute on the minute. The analysis accordingly was set up in 1-min blocks; that is, from one time indication to the next. Paper speed was 3 in. per second, or 15 ft of paper travel in the 1-min block.

To facilitate the "reading" of the time pips over the 1-min interval, a special 16-ft long wooden channel was designed with paper take-up reels at each end. The channel was marked off in 3-in. units, each unit corresponding to 1-sec of real time. The units were subdivided into 100-millisecond (Distances) increments. A vernier scale was constructed to be used in conjunction with the channel board which further increased the accuracy of the reading to 10 milliseconds by direct reading of the vernier scale, and to 1 millisecond by estimation of the vernier scale.

The procedure was to line up the first time pip with the zero time point on the channel and scotch tape the record in place. The vernier, which was made of transparent lucite, was then placed on top of the record, and indexed on successive pips simply by sliding it along the channel.

Additional Nomenclature

Two sets of additional nomenclature are required to describe the properties of the traffic stream as measured at the output boundary. The first set identifies attributes of individual vehicles in their respective positions in the moving stream crossing the output boundary of the system. The criterion for establishing the attribute of following-too-closely (FTC) is the widely quoted ¾-sec brake reaction time. The criterion for the attribute of closing-too-rapidly (CIG) is 8 ft/sec² which stems in part from the judgment of Gazis, Herman, and Maradudin (14) who use ⅓ g (approximately 10.7 ft/sec²) as a maximum "comfortable" deceleration. Thus, the 8 ft/sec² value used in this study is more conservative than that of Gazis and the others. Different criterion constants can be used at will, and which ones are appropriate are strictly matters of opinion.

FTC (I)	following too closely, (I^{th}: car following the $(I-1)^{th}$ car too closely), where: $FTC\ (I) = \left[D\ (I) \leq \left\{V\ (I)\ ft/sec\right\} \left\{3/4\ sec\right\}\right]$
NFTC (I)	not following too closely, where: $NFTC\ (I) = \left[D\ (I) > \left\{V\ (I)\ ft/sec\right\} \left\{3/4\ sec\right\}\right]$
DFV (I)	difference between speeds of I^{th} and $(I-1)^{th}$ cars, where: $DFV\ (I) = V\ (I) - V\ (I-1)$
CLG (I)	closing too rapidly (I^{th} car is closing too rapidly on the $(I-1)^{th}$ car), where: $CLG\ (I) = \left[\left\{(DFV)^2\ (I)\ /\ 2\ D\ (I)\right\} \geq 8.0\ ft/sec^2\right]$
NCLG (I)	not closing too rapidly, where: $NCLG\ (I) = \left[\left\{(DFV)^2\ (I)\ /\ 2\ D\ (I)\right\} < 8.0\ ft/sec^2\right]$
FTC. CLG	following too closely and closing too rapidly (compound attribute), where: (FTC. CLG) (I) = both FTC (I) and CLG (I) hold for the I^{th} car
$\left\{N\ (FTC.CLG)\right\} \left\{(I)\right\}$	the property FTC. CLG does not hold for the I^{th} car, or: $\left\{N\ (FTC.CLG)\right\} \left\{(I)\right\}$ = either NFTC (I), or NCLG (I), or both hold for the I^{th} car.

The following additional nomenclature is used to describe properties of a group of vehicles (as contrasted with the preceding nomenclature, which pertains to individual vehicles).

N(J)	total number of cars recorded in the J^{th} time group
N(WBC-)(J)	number of cars of wheel base classification WBC1, WBC2, etc., recorded in the J^{th} time group
N(FTC)(J)	number of cars following too closely in the J^{th} time group
N (NFTC)(J)	number of cars not following too closely in the J^{th} time group
N(WBC-. FTC)(J)	number of cars of wheel base classification WBC1, WBC2, etc., that were following too closely in the J^{th} time group
$P \left\{N(J)\right\} \left\{(WBC-)\right\}$	percentage of the cars in the J^{th} time group that were of wheel base classification WBC1, WBC2, etc. $$P \left\{N(J)\right\} \left\{(WBC-)\right\} = \frac{N(WBC-)(J)}{N(J)}$$
$P \left\{(WBC-)\right\} \left\{(FTC)\right\}$	percentage of the cars of wheel base classification WBC1, WBC2, etc., that were following too closely $$P \left\{(WBC-)(J)\right\} \left\{(FTC)\right\} = \frac{N(WBC-.FTC)(J)}{N(WBC-)(J)}$$
MEAN V(J)	mean speed of all cars in the J^{th} time group; alternately, $\overline{V}_J$

MEAN D(J)	mean following distance of all cars in the J^{th} time group; alternately, $\overline{D}_J$
VAR V(J)	variance of speeds of all cars in the J^{th} time group; alternately, $S^2_{V, J}$
VAR D(J)	variance of following distance of all cars in the J^{th} time group; alternately, $S^2_{D, J}$
MEAN V(WBC-)(J)	mean speed of all cars of wheel base classification WBC1, WBC2, etc., in the J^{th} time group; alternately, $\overline{V}_{(WBC-)(J)}$
VAR V(WBC-)(J)	variance of speeds of all cars of wheel base WBC1, WBC2, etc., in the J^{th} time group; alternately, $S^2_{(WBC-)(J)}$
MEAN D(WBC-)(J)	mean following distance of all cars of wheel base WBC1, WBC2, etc., in the J^{th} time group; alternately, $\overline{D}_{(WBC-)(J)}$
VAR D(WBC-)(J)	variance of following distance of all cars of wheel base WBC1, WBC2, etc., in the J^{th} time group; alternately, $S^2_{D(WBC-)(J)}$
MEAN V(WBC-. FTC)(J)	mean speed of all cars of wheel base WBC1, WBC2, etc., and following too closely in the J^{th} time group; alternately, $\overline{V}_{(WBC-. FTC)(J)}$

The grouping of vehicles was according to predetermined (clock) time intervals at the output boundary. The interval in this study is 1 min and is identified by the letter J in relation to a reference time. Thus J = 9 means the 9^{th} minute after the reference time. In this study the beginning of each separate study was considered as the reference time for that study.

Computer Analysis

A program was written for the computer to perform the necessary mathematical operations on the set of time data, and to output the results in a directly readable form. This latter objective is one of the principal reasons why the FORTRAN-like language was used to describe the variables in the study. The machine used in the study is a part of the facilities of the Western Data Processing Center located on the Los Angeles Campus of the University of California. Specimen outputs of the computer program are shown in Figures 14 and 15.

Results

The results, which essentially are enumerative in nature, are read directly from the computer output. The output of primary interest to the general study is the distribution of MEAN (RSP) according to clock time as given in Eq. 13. With the MEAN (RSP) known, the macroscopic "within" TRT diseconomy can accordingly be identified. The detailed computations of the diseconomies are given in the analysis section.

Several interesting additional observations may be made which suggest the possibility of much more general uses of this coordinated field and computer technique. In Figure 14, the first three vehicles are moving at fairly high speeds (48, 46, 52 fps); the 4th vehicle, which measures 617 in. from its first to last axle, is a truck moving at a speed of 20.5 fps. All vehicles behind this truck are moving at the greatly reduced speed. The speeds proceed to build up until the arrival of what appears to be an Isetta type of vehicle (the 7th vehicle in the 15th min in Figure 15), whereupon speeds again are reduced.

Referring again to the 14th minute (Fig. 14), the following distances of the high-speed vehicles are 184, 150, 171 ft, whereas that for the truck (4th vehicle in line) is 583 ft. Thereafter, the following distances are 82, 62, 88 ft, etc. The picture conveyed by the data, therefore, is a large space gap ahead of the truck followed by a relatively tightly bunched platoon of slow-moving vehicles.

Figures 14 and 15 pertain only to the shoulder lane. However, the general study

```
THE FOLLOWING IS FOR DATA COLLECTED AT SITE  8 LANE  3

DATA IS FOR MINUTE NUMBER.14
```

V(I)	W(I)	D(I)	WBC(I)	B(I)	FTC	CLG	FTC.CLG
48.193	102.940	184.000	2	7.035	NO	NO	NO
45.977	113.655	149.880	3	10.149	NO	NO	NO
51.948	109.714	171.172	2	13.962	NO	YES	NO
20.513	617.108	583.376	4	25.210	NO	NO	NO
18.957	115.564	82.380	3	29.242	NO	NO	NO
18.433	507.429	61.725	4	32.504	NO	NO	NO
14.440	126.845	87.834	3	37.329	NO	NO	NO
18.265	116.822	36.022	3	39.904	NO	YES	NO
15.267	120.000	40.201	3	42.148	NO	NO	NO
13.072	115.765	27.405	3	43.987	NO	NO	NO
13.652	115.167	33.255	3	46.518	NO	YES	NO
24.390	112.976	76.997	3	52.029	NO	YES	NO
28.777	126.389	58.463	3	54.401	NO	YES	NO
25.157	112.000	58.763	3	56.463	NO	NO	NO
22.989	126.621	89.912	3	60.052	NO	NO	NO

```
                 N= 15.              N(FTC)=  0.             N(CLG)=  5.          N(FTC.CLG)=  0.

          N(WBC1)=  0.             N(WBC2)=  2.            N(WBC3)= 11.             N(WBC4)=  2.
      N(WBC1.FTC)=  0.         N(WBC2.FTC)=  0.        N(WBC3.FTC)=  0.         N(WBC4.FTC)=  0.
      N(WBC1.CLG)=  0.         N(WBC2.CLG)=  1.        N(WBC3.CLG)=  4.         N(WBC4.CLG)=  0.
  N(WBC1.FTC.CLG)=  0.     N(WBC2.FTC.CLG)=  0.    N(WBC3.FTC.CLG)=  0.     N(WBC4.FTC.CLG)=  0.
      P(N)(WBC1)=  0.         P(N)(WBC2)= 13.33       P(N)(WBC3)= 73.33        P(N)(WBC4)= 13.33
    P(WBC1)(FTC)=  0.       P(WBC2)(FTC)=  0.       P(WBC3)(FTC)=  0.        P(WBC4)(FTC)=  0.
    P(WBC1)(CLG)=  0.       P(WBC2)(CLG)= 50.00     P(WBC3)(CLG)= 36.36      P(WBC4)(CLG)=  0.
P(WBC1)(FTC.CLG)=  0.   P(WBC2)(FTC.CLG)=  0.   P(WBC3)(FTC.CLG)=  0.    P(WBC4)(FTC.CLG)=  0.

           TNVNC=  0.                N= 15.              ALLV= 15.     FVNC=TNVNC/ALLV=  0.
           MEANV= 25.335         MEAND=110.226           VARV=167.399
           SAVEV= 35.063         SSUMN=107.

    MEANV(WBC1)=  0.        MEANV(WBC2)= 50.070     MEANV(WBC3)= 21.904     MEANV(WBC4)= 19.473
    MEAND(WBC1)=  0.        MEAND(WBC2)=177.586     MEAND(WBC3)= 67.555     MEAND(WBC4)=3[illegible].551
     VARV(WBC1)=  0.         VARV(WBC2)=  7.051      VARV(WBC3)= 91.247      VARV(WBC4)= [illegible].162
```

Figure 14. Specimen output of the computer program used for computing properties of the traffic stream at the output boundary in the 14th minute.

```
THE FOLLOWING IS FOR DATA COLLECTED AT SITE  8 LANE  3

DATA IS FOR MINUTE NUMBER,15
```

V(I)	W(I)	D(I)	WBC(I)	B(I)	FTC	CLG	FTC.CLG
32.787	130.229	90.917	3	3.536	NO	YES	NO
32.000	122.112	80.852	3	6.005	NO	NO	NO
32.000	123.643	91.008	3	8.349	NO	NO	NO
36.036	129.277	69.000	3	11.010	NO	YES	NO
35.714	121.296	80.108	3	13.224	NO	NO	NO
35.398	127.434	103.679	3	16.128	NO	NO	NO
12.945	45.049	79.540	1	18.581	NO	NO	NO
27.397	120.658	25.139	3	20.360	NO	YES	NO
40.816	117.551	21.699	3	21.104	YES	YES	YES
37.037	122.667	120.735	3	24.072	NO	NO	NO
35.398	109.593	116.000	2	27.209	NO	NO	NO
34.783	112.276	49.097	3	28.598	NO	NO	NO
37.037	111.111	73.635	3	30.708	NO	YES	NO
40.816	115.102	57.778	3	32.252	NO	YES	NO
37.037	137.778	84.041	4	34.227	NO	NO	NO
41.667	137.000	105.000	4	37.150	NO	YES	NO
42.105	133.333	69.042	3	39.286	NO	YES	NO
41.237	126.165	115.074	3	42.021	NO	NO	NO
43.011	133.645	66.345	3	43.638	NO	YES	NO
42.553	116.936	64.633	3	45.143	NO	NO	NO
37.383	120.672	56.255	3	45.478	NO	NO	NO
42.553	137.361	73.421	4	46.429	NO	YES	NO
37.383	109.009	43.659	2	49.408	NO	NO	NO
44.944	138.000	40.720	4	50.700	NO	YES	NO
47.059	145.694	64.000	4	52.565	NO	YES	NO
47.619	125.714	73.500	3	54.120	NO	YES	NO
52.632	107.369	127.714	2	56.300	NO	YES	NO
41.667	127.000	81.158	3	58.302	NO	NO	NO
43.011	125.741	63.167	4	59.375	NO	YES	NO

```
            N= 29             N(FTC)=  1.           N(CLG)= 15.        N(FTC.CLG)=  1.

        N(WBC1)=  1.          N(WBC2)=  3.          N(WBC3)= 19.         N(WBC4)=  6.
    N(WBC1.FTC)=  0.      N(WBC2.FTC)=  0.      N(WBC3.FTC)=  1.     N(WBC4.FTC)=  0.
    N(WBC1.CLG)=  0.      N(WBC2.CLG)=  1.      N(WBC3.CLG)=  9.     N(WBC4.CLG)=  5.
N(WBC1.FTC.CLG)=  0   N(WBC2.FTC.CLG)=  0   N(WBC3.FTC.CLG)=  1. N(WBC4.FTC.CLG)=  0.
      P(ND(WBC1)=  3.45     P(N(WBC2)= 10.34     P(N(WBC3)= 65.52    P(ND(WBC4)= 20.69
   P(WBC1)(FTC)=  0.     P(WBC2)(FTC)=  0     P(WBC3)(FTC)=  5.26  P(WBC4)(FTC)=  0
   P(WBC1)(CLG)=  0     P(WBC2)(CLG)= 33.33   P(WBC3)(CLG)= 47.37  P(WBC4)(CLG)= 83.33
P(WBC1)(FTC.CLG)=  0. P(WBC2)(FTC.CLG)=  0. P(WBC3)(FTC.CLG)=  5.26 P(WBC4)(FTC.CLG)=  0.

         THVNC=  0.               N= 29.            ALLV= 29.      FVNC=THVNC/ALLV=  0.
         MEANV= 37.346        MEAND= 77.037         VARV= 52.537
         SAVEV= 35.702        SSUMN=130.

 MEANV(WBC1)= 12.945     MEANV(WBC2)= 41.804     MEANV(WBC3)= 37.753     MEANV(WBC4)= 42.7[illegible]
 MEAND(WBC1)= 79.540     MEAND(WBC2)= 95.791     MEAND(WBC3)= 74.152     MEAND(WBC4)= [illegible]
  VARV(WBC1)=  0.         VARV(WBC2)= [illegible].907   VARV(WBC3)= 24.137    VARV(WBC4)= 11.[illegible]
```

Figure 15. Specimen output of the computer program used for computing properties of the traffic stream at the output boundary in the 15th minute.

includes analyses for the middle and median lanes as well, with all three analyses being on the identical time base. This permits correlating properties of the traffic stream in one lane with those in the adjacent lanes. For example, an analysis (not shown here) of traffic in the middle lane shows that this traffic was moving quite rapidly at the instant the truck in question in Figure 14 presumably was causing the shoulder lane traffic to move much more slowly. The computer output thus easily discloses the widely recognized situation: a truck moving slowly up a grade followed by a long line of cars which are prevented from passing the truck by high-speed traffic in the adjacent lane.

The major significance of the work is that, through the use of the relatively simple multi-lane detector field techniques coupled with the high-speed computer analysis, it becomes possible and economically feasible to quantify in a highly precise manner a wide variety of properties of interacting traffic streams. Readily recognized are the implications this has for theoretical and applied traffic research.

TRAVEL TIME REQUIRED TO NEGOTIATE SYSTEM OF INTEREST, AND ANY INFERRED DISECONOMIES

License matching was selected as the method for determining travel time in this study for three reasons. First, it was mandatory that travel time be measured as accurately as possible, and, in the words of Highway Research Board Committee on Operating Speeds in Urban Areas (4):

> "..... The license matching method has been accepted
> by the Committee as being a reliable standard upon
> which to base the accuracy of other methods..........."

Second, it was known from the inception of the study that a large number of travel time measurements would be required at fairly close (clock) time intervals. Repeated trials in pilot studies indicated that something of the order of three to four runs could be made by a single floating car during the afternoon peak hour. Thus, it did not appear that any of the variations of floating-car techniques would be practical. As it developed with the license matching method, the equivalent of approximately 5,000 floating-car runs were obtained in a 1-hr period.

Third, this study required travel time measurements over 38 input-output combinations, with each of the lanes at the primary output boundary being treated as separated sites for purposes of lane-to-lane comparisons. Furthermore, the measurements had to be made essentially at the same (clock) time. It was out of the question to attempt this set of measurements with anything but license matching methods.

Notwithstanding these strong arguments favoring the license method, there were the difficulties associated with the method, as pointed out by Walker (4), of obtaining matchings, subtracting time of passage, and eliminating spurious matchings. These difficulties were minimized in large measure by using computer method that was reported earlier by Brenner, Mathewson, and Gerlough (11). A technical memorandum outlining the program will shortly be published (15).

The input to the computer included the individual licenses, site at which they were identified, and (clock) time at which they were identified. The license encoding was limited to the first five characters (3 alphabetic, 2 numeric) to reduce card punching costs. However, obviously spurious matchings occurred with the five-character identification. Suspicious matchings were apparent from unusual travel times, and could easily be confirmed as spurious by reference to the original field data. Consequently, in future studies, all six characters will probably be used.

Although the clock time was recorded in the field to the nearest second, it was encoded by minute for computer analysis because the computer program permits a five part linear interpolation of time which could yield interpolated answers to 12-sec accuracy. This was considered adequate for purposes of this study. Also, this procedure of outputting travel time in units of ⅕-min significantly simplifies enumeration of results.

The computer output of interest included the license that was a matching, its input site, its output site, the input clock time, output clock time, and finally the difference in clock time; that is, travel time.

A specimen graph of travel time plotted against clock time at the input boundary is shown in Figure 16. This particular figure pertains to travel from site 1 to site 8, and hence represents the primary freeway traffic; that is, "the slot" traffic. Each point on the curve represents the average of the travel times for all vehicles arriving at the input boundary in the given minute. The minute input volumes vary from 28 to 42 vehicles, and the graph represents in all 960 vehicles. Similar graphs have been prepared for every input-output combination but are not shown here.

It can be seen in Figure 16 that distinct travel time peaks occurred at approximately 4:10 p.m. and 4:40 p.m. for the movement from sites 1 to 8. The same double peak picture occurs in all of the other flow path combinations; for example, sites 2 to 8, 1 to 7, 2 to 7, etc. It is a matter of considerable interest to examine some of the possible explanations for the two peaks occurring, instead of a single uniform p.m. peak, as would be normally expected.

The first possibility is that there was coincident traffic congestion downstream of the section of interest. However, an examination of RSP (OUB) failed to disclose any unusual build up at the same time the two TRT peaks occurred. This may be seen in Figure 16 in which the inferred mean TRT curve is plotted on the same (clock) time scale as the realized mean TRT curve. The inferred curve is deduced directly from, and is a linear function of RSP (OUB). Each point on it, as with the realized curve, represents an average of the RSP (OUB) performance of upwards of 20 vehicles. It can be seen that RSP (OUB) is generally increasing during the course of the hour, but there are no peaks that could be related to the realized TRT peaks. The only distinct peak, at CLT (INB) = 1,651, was due to a minor bumper-to-bumper accident which occurred downstream of OUB at 1,657.

A second possible explanation of the TRT peaks is volume of traffic at the output boundary. In Figures 17 (a) and (b), 1-min lane volumes and 1-min totals of all traffic of the output boundaries are plotted on the same (clock) time scale as the TRT. Also plotted are 5-min totals of truck volumes for all trucks, busses, and 5-min totals for

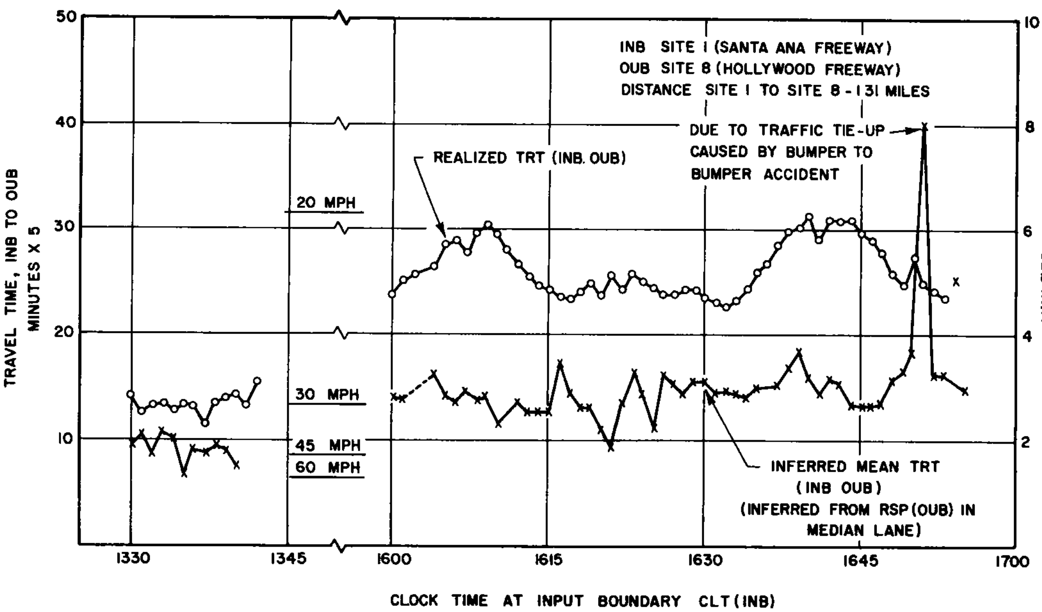


Figure 16. Specimen graph of travel time as a function of clock time at the input boundary.

the "heavy" trucks; that is, trucks with semi or full trailers. It can be seen that the total volume curve essentially remains flat over the hour. Both truck curves, if anything tend to drop from 4 to 5 p.m. Therefore, from these curves, there is little indication that the TRT peaks coincide with the volume factors (total volume, lane volumes, truck volumes).

A particularly interesting result is seen by comparing the curve of shoulder lane volume with the truck curves. The shoulder lane volume is in a decided upward trend from 4 to 5 p.m. while there concomitantly is a definite downward trend for both truck curves. Most of the heavy trucks (73 percent) are in the shoulder lane. Thus the paired trends could be construed as an argument on the reduction of capacity with truck

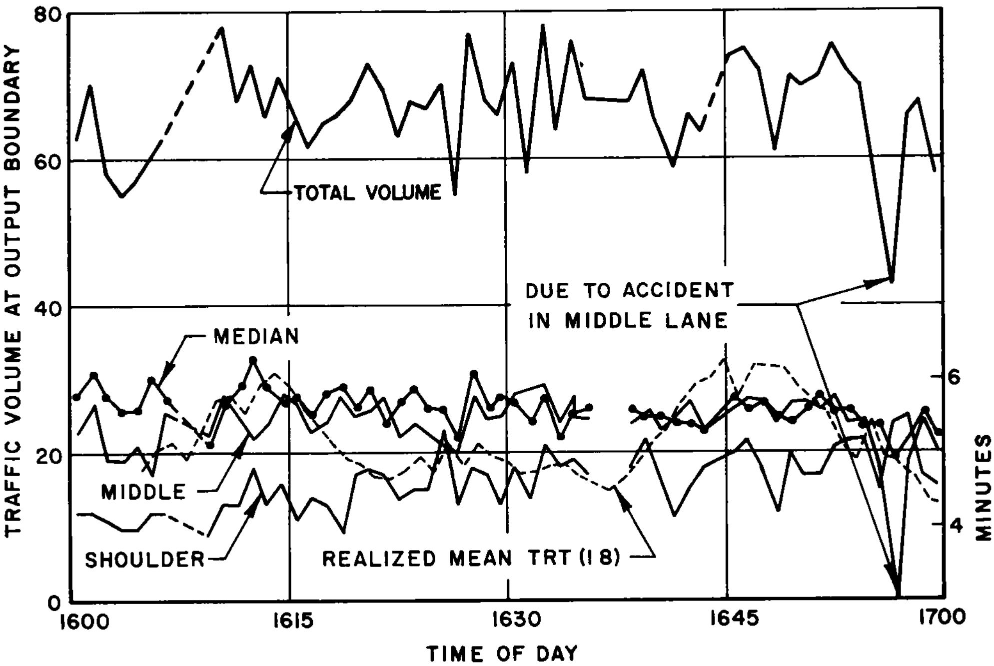


Figure 17(a). Lane-by-lane volumes, total volume, truck volume at the output boundary as a function at clock time.

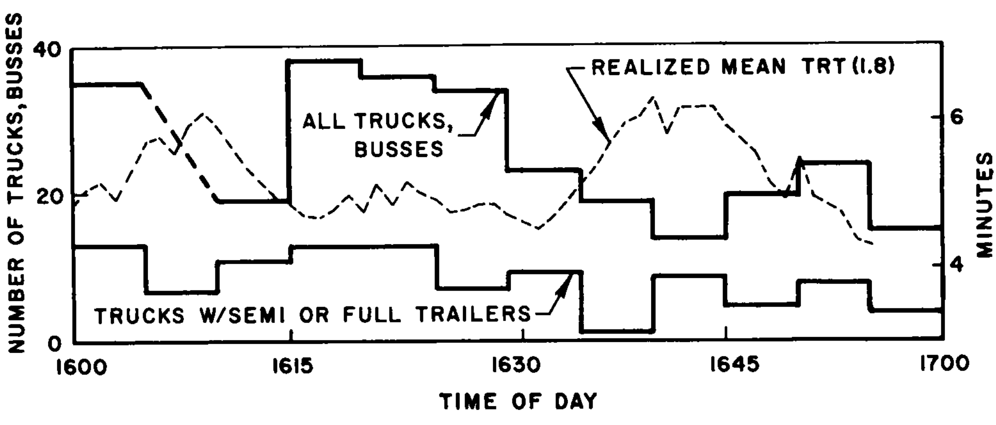


Figure 17(b). Lane-by-lane volumes, total volume, truck volume at the output boundary as a function at clock time.

traffic. On the other hand, the volumes in the middle and median lanes are essentially constant for most of the hour, and hence might represent the practical capacities. The build up in the shoulder lane might therefore be due to a build up in total demand on the system, with the incremental volume appearing in the shoulder lane because the other two lanes were operating close to practical capacity. The concomitant decrease in truck volume might not therefore be the dominant reason for the increase in shoulder lane total volume. (The capacity results obtained at site 8 are in relatively close agreement with those reported by Webb and Moscowitz (17) on observations made near this site in August 1956. They suggest using the volume during the highest 5-min period as a basis for estimating the peak hour volume. The essentially flat nature of the median and middle lane curves in Figure 17 support their view completely. It also appears that a slightly more refined estimate might be obtained by taking separate 5-min estimates for the middle and median lane, and then applying a correction factor for the shoulder lane based on a "least squares" straight line. First, however, further studies would be needed to establish the generality of the linear coefficients.) The important conclusion here in the travel time context, is that the TRT peaks occur contrary to capacity factors—indications that the TRT variable relates to more complex factors than capacity alone.

The only data in this study which appear to be closely related to the travel time peaks are the number of vehicles coming onto the freeway proper from intervening on-ramps. The on-ramp movements are plotted in Figure 18 on the same (clock) time scale as travel time. There are three distinct on-ramp peaks, approximately at 4:12 p.m., 4:22 p.m., and 4:37 p.m. The 4:37 peak of on-ramp traffic could be causing the 4:35 to 4:48 TRT peak; the 4:12 peak of on-ramp traffic could be related to the 4:05 to 4:12 TRT peak; the 4:22 on-ramp peak might be related to the (small) 4:20 to 4:25 TRT peak.

In the absence of more detailed information, these observations as to possible relationships between the on-ramp movements and through travel time can only be treated as conjectures. It is known, however, that there are at least two distinct "waves" of on-ramp traffic during the afternoon peak hour coincident with the two quitting times, 4:00 p.m. and 4:30 p.m., for many governmental workers in the civic center. It accordingly is at least a plausible consideration that the TRT peaks are related to on-ramp movements as shown.

ESTIMATES OF FLOW PERCENTAGES FOR DIFFERENT INPUT-OUTPUT COMBINATIONS OF THE SYSTEM

This sub-study deals with estimating the percentages of traffic at a given input site moving through the system to exit at a given output site. Quantifying the relative volumes of traffic moving between the various input-output combinations is referred to as a "flow pattern analysis", and has a wide variety of uses. The immediate use in this study is for obtaining travel time for the "collection" as defined earlier, with the make-up of the "collection" being directly deduced from the flow patterns. Another important reason for determining flow patterns is to provide one of several bases required for estimating the amount of traffic that will be diverted from the "system of interest" as additional links in the total freeway network are completed.

Some of the present SAF - HRF, and SBF - HRF traffic will use the Santa Monica Freeway (now under construction) and by-pass the "slot" which presently is a necessary link in its freeway path; some of the SAF - HLF, and SBF - HLF traffic will also be diverted from the "slot" when the Golden State Freeway is completed. Conceivably, the amount of diverted traffic might reduce the load on the "system" to a point where the system would operate without undue congestion even with ramps remaining open that are now being considered for closing. Any relief afforded by closing the ramp would thus only be needed for a relatively short period of time, and might not be worth whatever negative publicity could come with closing the ramp. More important than negative publicity, however, would be the expenditure of significant sums of money to relieve a present bottleneck in the face of some likelihood that the bottleneck would automatically be relieved with the completion of additional links of the network.

Knowledge of the flow patterns is invaluable for evaluating this likelihood that the problem will correct itself as the planned-for network links are completed.

The problem of estimating the volume of traffic at a given on-ramp that proceeds to a particular off-ramp of a freeway system has been treated in detail in an earlier work by Brenner, Mathewson and Gerlough (11). In a later work, Mathewson and Brenner apply the same technique to estimating through traffic on a surface street network (16). The technique is particularly well suited to the present study because of the difficulty in obtaining a 100 percent sample at some of the sites coupled with the fact that the technique permits sampling of traffic. The formula used for estimating the flow percentages in this technique (derived in the earlier work (11)) is:

$$P\left\{(A)\ (B)\right\} = \frac{[NM\ \{(A)\quad (B)\}]\quad [TV\ (B)]}{[NI(A)]\quad [NI(B)]} \tag{15}$$

in which

$P\left\{(A)\ (B)\right\}$ = percentage of vehicles at A going to B;

NI (A) = number of vehicles identified at A;

NI (B) = number of vehicles identified at B;

TV (B) = total volume of traffic at B; and

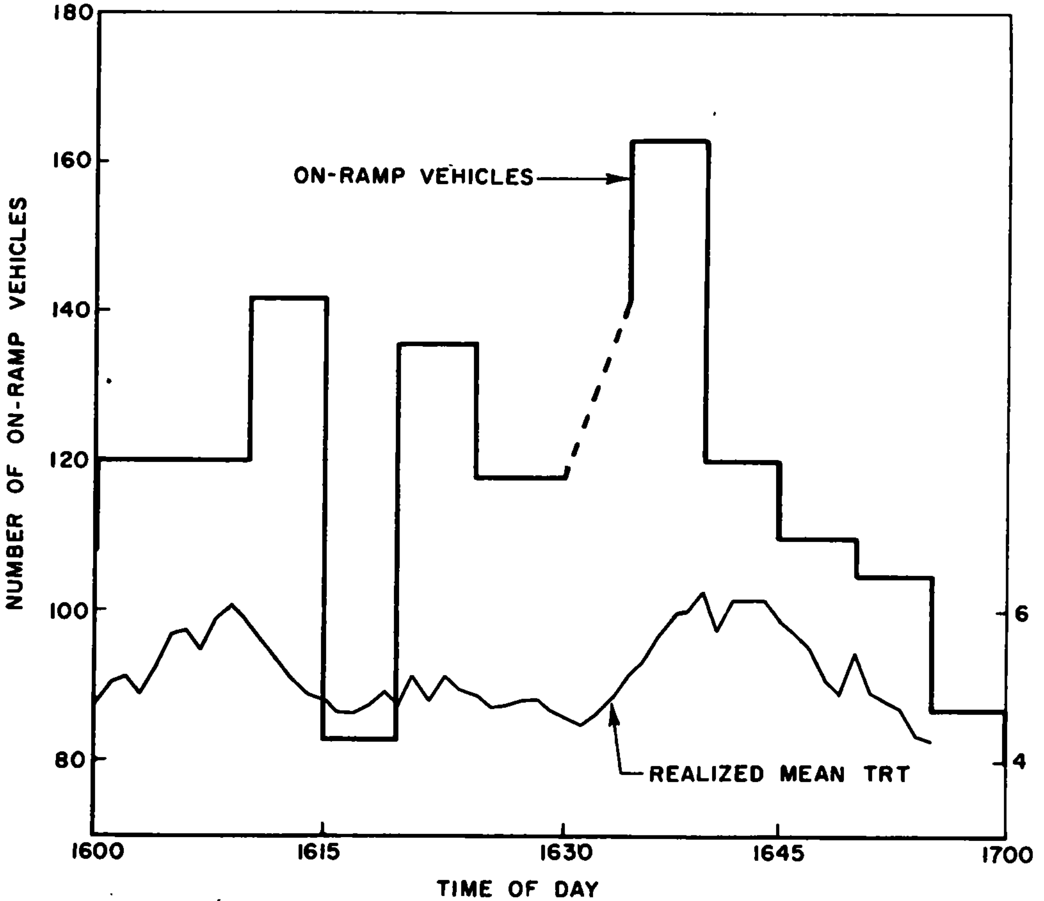


Figure 18. On-ramp movements as a function of clock time at on-ramp.

NM $\left\{(A)(B)\right\}$ = number of license matchings between the NI (A) and NI (B) identifications.

All of the factors in Eq. 15 had to be measured experimentally to determine network travel time as discussed in "Travel Time Required to Negotiate System of Interest, and any Inferred Diseconomies." Thus, the flow percentages could be obtained at no extra experimental cost, and the flow pattern analysis can accordingly be regarded as an immediate by-product of the travel-time analysis, or vice versa. However, not nearly as many license matchings are needed to produce a travel time analysis of fairly high percision as are needed for a precise flow pattern analysis.

The results shown in Figure 19 concern the percentage of traffic at specific input sites that proceed to specific output sites—P (INB. OUB). The results shown in Figure 20 concern the percentage of traffic at specific output sites that come from specific input sites—P (OUB. INB). Both sets of percentages were computed in two ways. In the first, Eq. 15 is the basis of the first method which essentially seeks to correct for the sampling ratios. The second method is based strictly on the distribution of matchings and became necessary after it was found that with the first method, the total percentages were significantly lower than the expected summation for a set of probabilities; namely, unit.

Some of the discrepancies between the percentage totals and unity can be explained by traffic to the unmanned intervening sites. The greater part of the discrepancy,

AT ↓ / TO →		OUTPUT SITES: Off-Ramp 3	Freeways 8	9	10	Primary Total	Auxiliary to Site 11
INPUT SITES: Freeways	1	.01 / .01	.48 / .78	.10 / .16	.04 / .05	.63 / 1.00	.45 /
	2	.03 / .07	.20 / .45	.21 / .47	.05 / .01	.49 / 1.00	.19 /
On-Ramps	4		.45 / .71	.14 / .23	.05 / .06	.64 / 1.00	.35 /
	5		.49 / .81	.08 / .12	.05 / .07	.62 / 1.00	.47 /
	6		.25 / .39	.32 / .52	.07 / .09	.64 / 1.00	.31 /

Flow percentage as estimated with sampling ratio adjustments. / Flow percentage as estimated from matchings, without considering sampling ratios.

Figure 19. Percentages of traffic at input sites that proceed to output sites P(INB.OUB).

AT → / From ↓		OUTPUT SITES: Off-Ramp 3	Freeways 8	Freeways 9	Freeways 10
INPUT SITES: Freeways	1	.10 / .25	.33 / .55	.11 / .24	.18 / .50
	2	.40 / .75	.14 / 21	.24 / .49	.02 / .07
INPUT SITES: On-Ramps	4		.05 / .09	.03 / .07	.04 / .12
	5		.05 / .09	.01 / .03	.04 / .11
	6		.04 / .06	.08 / .17	.07 / .20
Total		.50 / 1.00	.61 / 1.00	.47 / 1.00	.35 / 1.00

Flow percentage as estimated with sampling ratio adjustments. — / — Flow percentage as estimated from matchings, without considering sampling ratios.

Figure 20. Percentages of traffic at output sites that come from input sites P(OUB.INB).

however, can only be interpreted as stemming from licenses being incorrectly identified. These errors prevent licenses from being matched (missed matchings) resulting in flow percentages computed according to Eq. 15 being lower than they should be. Furthermore, the ultimate estimate of the percentage is lowered by erroneous identifications at the input site independently of erroneous identification at the output sites, and by errors at the output site independently of those at the input site. Identifying licenses incorrectly is known to be the major bias in applying Eq. 15 to a real traffic situation, and is discussed in detail by Brenner, et al. (11).

Using the matchings (which would have to be correct identifications) as reference points, it was possible to isolate obvious errors in the (unmatched) identifications, and to estimate that something of the order of 5 percent of the identifications could be suspected as being in error, and this number of erroneous identifications would easily explain the flow extimates being low. But, rather than proceed through the entire set of data in the extremely laborious checking for incorrect identifications, a more gross estimation procedure was used. In this procedure, flow percentages were estimated by the relative percentages of matchings within a total set of matchings. For example, of the 1,766 matchings among the site 8 traffic, 960 were site 1 matchings, and the ratio of 960 to 1,766 or 0.78 was treated as the flow percentage of site 8 traffic from site 1.

This procedure based on matchings, although not precise, is not as gross as might be suspected at first glance. For one thing, it is exact if the sampling ratios at the output sites are the same. The sampling ratios at sites 8 and 9 are 0.84 and 0.83, respectively. Thus the respective number of matchings site 1 to site 8, and site 1 to site 9 are essentially in direct proportion to the respective flow percentages. Because the flows to these output sites from the input sites 1 and 2 represent the major part of the traffic through the system, accurate percentage estimates for these two primary patterns cause the patterns for the system as a whole to be fairly accurate.

Additional support for the belief that the gross procedure has yielded results that are sufficiently accurate for purposes of this study may be seen in the ramp data. From the mailing questionnaire, the flow percentages of BON traffic to HRF, PSF, and HLF are 50.0 percent, 11.8 percent, and 38.2 percent, respectively. The same percentages as obtained from the gross matching procedure for the same traffic are 52.0 percent, 39 percent, and 9 percent, respectively. Similar agreement exists for the other on-ramps.

Notwithstanding the fact that the number of matchings obtained in this study were sufficiently high (and the sampling ratios at the output boundary for the primary traffic happened to coincide) to yield what appear to be sufficiently accurate estimates of the flow percentages for the major flow patterns, it nevertheless is necessary to examine the present and ultimate usefulness of the techniques based on Eq. 15. The identification errors could have occurred anywhere in the total process beginning with the observer dictating the license, continuing through the data reduction process, and to the ultimate encoding of the data onto punched cards. Errors obviously occurred in the dictation in the field; there is evidence of this in a by-product investigation which will be described presently. There also is evidence of errors in the photographic reading due mainly to changing light conditions under which the licenses were originally photographed. Furthermore, the investigators detected encoding errors, notwithstanding the fact that the card punching was also verified.

These three major sources of error can be remedied at least in part. The observers used in the study were not experienced in the dictation methods and with more training and experience would undoubtedly perform better. Since the date of the study, there has become available a new film (18) designed to give more than twice the speed of the film used in the study without a significant increase in graininess. It appears that with the new film a much sharper picture of the license plate will be obtained, with the result that errors in license identification should be reduced. Finally, there are several controls on the data handling procedures that would further reduce errors. It, therefore, appears that Eq. 15 will prove to be more effective for freeway work than shown in the present study. At the present state of the art, however, the technique probably should be applied principally at ramp-to-ramp types of freeway pattern analysis.

The by-product investigation mentioned earlier involved computing two independent sets of flow percentages for traffic heading for the Hollywood Freeway. Site 8 (Grand Avenue overcrossing) is the output boundary in one set; site 11 (Edgeware Avenue overcrossing) is the output boundary in the second set. The input boundaries are identical for the two sets. Both sites are on the Hollywood Freeway, and are approximately ½ mi apart, with traffic of interest reaching site 8 first.

Site 8, being the primary OUB of the study, had relatively complete coverage; namely, a camera on each of its three lanes together with the observer using a tape recorder for time check purposes. The over-all sampling ratio, licenses identified to total volume of traffic, was 84 percent. Site 11, on the other hand, had the much lower sampling ratio of 16 percent for several reasons. This site was included in the experiment plan principally as the downstream control point for site 8; that is, to provide an RSP (OUB) measurement in the event of equipment failure at site 8. Inasmuch as relatively small number of license identifications would suffice for this purpose, identifications were limited to whatever identifications two observers, each with his own tape recorder, could make of the very heavy traffic on the four lanes at this site. Also contributing to the depressed sampling ratio was the fact that the volume of traffic was much higher at site 11 (ADT = 106,300) than site 8 (ADT = 59,100) because all of the traffic bound for the Hollywood Freeway from the Harbor Freeway

(ADT = 36,000) and Pasadena Freeway (ADT = 11,200) passes site 11, but not site 8.

Because all of the traffic at site 8 has to pass site 11 (there being no intervening off-ramps), the percentage of traffic at a given on-ramp going to site 8 is identically the same as the percentage from the same on-ramp going to site 11. By treating sites 8 and 11 as two independent output boundaries in the license matching formulas, two independent estimates are obtained of the same variable. The question is how well the two estimates will agree with each other.

It may be seen in Figure 19, that the two sets of flow percentage estimates are in close correspondence, indicating that two observers, equipped with nothing more than tape recorders, were able to produce a flow pattern that is essentially the same as that produced by the more expensive camera technique. The far-reaching implication is that with the portable tape recorder technique, it should be possible to map complete time gradients for a freeway network and at the same time obtain reasonably accurate estimates of flow patterns—both at relatively low cost. The photographic method, particularly with the new film, would continue as the more desirable in those situations where the flow patterns have to be determined more accurately.

GENERAL ANALYSIS

The general analysis seeks to estimate a priori how closing a particular on-ramp, the Broadway Avenue On-Ramp in this case, will affect the travel time the "collection" requires to clear the system of interest, with the "collection" being comprised basically of three classes of traffic: (1) the freeway traffic, (2) the surface street traffic, and (3) the ramp traffic that will have to go to a different ramp. The analysis brings together the results of the previously described sub-studies, seeking only order-of-magnitude estimates. Furthermore, the analysis is limited to a 1-hr p.m. peak period. The extrapolations of these data to a full day, week, or month period are not performed here because they contribute nothing additional to the presentation of the analytical techniques. Clearly, it would be necessary to extrapolate the data in actual practice.

Effect on Present Ramp Users

The "mailing" questionnaire and the subsequent "expert" questionnaire established that most of the present Broadway Avenue On-Ramp (BON) traffic would head for the Grand Avenue On-Ramp (GON) in the event BON was closed. The most conservative treatment is assumed, specifically, that all of the BON traffic would head for GON, because this condition would cause the greater decrement to surface street patterns as well as to the diverted traffic.

A total of seventeen points of origin of BON users were identified in the surrounding surface street network. For each point of origin, a new route was traced out on the surface street network to the freeways via GON. The new route was then compared with the present routes to BON. From the known differences in distance between the routes to GON and BON, a travel time decrement was estimated considering that the average speed of traffic was 15 mph on surface streets and 30 mph on the freeway. The realized average spot speed on the freeway (median lane) was 26.5 mph, so that using the 30-mph speed is conservative. The travel time decrement for each route was then weighted by the number of vehicles using that route, the (weighting) numbers being determined from the replies to the mailing questionnaire. The weighted decrements were finally added to yield an estimated total decrement for all traffic during the hour. This amounted to 1,070 vehicle-minutes.

The analysis was then repeated considering that freeway traffic was moving at 15 mph, instead of 30 mph. The results show a time saving of 585 vehicle-minutes in the hour if BON traffic took the alternate routes via GON to the freeway as contrasted with the time loss of 1,070 vehicle-minutes in the previous analysis. Thus, if freeway traffic is moving at the slow speed, it pays the BON traffic to stay on the surface streets a bit longer and enter the freeway network at the GON ramp. If the freeway traffic is moving at the faster speed, the BON traffic experiences a time loss by having to delay getting onto the freeway until GON. The point of indifference for the BON traffic is not computed here, but would have freeway traffic moving at some speed between 15 mph and 30 mph. Instead, the 30-mph figure is used and the time loss considered to be 1, 070 vehicle-minutes.

Effect on Present Surface Street Traffic

For each of the seventeen points of origin in the surface street network, a distance decrement had to be computed between the two alternate paths to the freeway network (via BON or GON) to establish the time decrement. As was done with the time decrement, the distance decrement was also weighted by the relative number of ramp users taking the present (BON) path according to the mailing questionnaire. The weighted sum was +10. 7 miles which shows that by using the alternate path via GON, present BON traffic in the p. m. peak hour would save this aggregate distance. In other words, present BON users are driving slightly longer distances on the surface streets to get to the freeway in order to obtain the time benefit of driving on the freeway. This exemplifies the widely recognized pheonmenon of drivers traveling greater distances (in this case both on the surface streets and the freeway) to reduce their over-all travel time. A problem of major interest would be to determine these trade-off curves (distance for time) under a wider variety of conditions.

Taken at face value for the purposes here, the results suggest that diverting the BON traffic to GON will not be detrimental to present surface street traffic because, if anything, the diversion will reduce the mileage being driven on surface streets. This is an oversimplification, of course, for there might be local disturbances that would cause the surface street traffic to suffer travel time decrements in spite of the aggregate vehicle mileage being reduced. In any case, it does not appear that the effects on present surface street traffic will be severe.

Effect on Present Users of the Alternate Ramps (s)

The most critical local disturbance resulting from diverting traffic from one ramp to another is expected to be at the alternate ramp. If the alternate ramp is lightly loaded, the disturbance will be minimal, but if it is already heavily loaded, the additional traffic would undoubtedly cause a severe local disturbance. As stated earlier, in the most conservative analysis (that is, to produce the most severe local disturbance) all of the BON traffic would be considered to be diverted to the one alternate ramp, GON. However, from the "expert" questionnaire it is judged that were the Broadway On-Ramp to be closed, the present BON. PSF traffic would go to the Castellar On-Ramp, not to the Grand Avenue On-Ramp. This amounts to 9 percent of the BON traffic. Also, the back-up on the collector road of traffic heading for the Harbor Freeway during the peak hours usually extends beyond the point where the GON traffic merges with the collector road. It accordingly is judged that BON. HRF traffic would head for GON for the first few days after BON is closed, but after encountering the back-up at GON, would seek out the more logical ramp to get on to HRF at 3rd Street. (This ramp is not shown in any of the figures because it is not a part of the defined "system", but should now be included in the over-all analysis. Required is the effect that present BON. HRF traffic will have on the surface street network which intervenes between points of origin and the 3rd Street On-Ramp. To simplify the presentation here, this iteration is omitted, and the assertion made that the BON. HRF traffic will create far less disturbance using this as the alternate ramp rather than GON.) From the flow pattern analysis, the BON. HRF traffic is 52 percent of the BON traffic.

Thus, the net effect closing BON would have on GON at the outset would be an additional loading comprised of 61 percent of the present BON traffic or 400 vehicles. This would drop to 39 percent of the present BON traffic, or 250 vehicles after the system became stabilized. The Grand Avenue On-Ramp presently carries 1, 400 vehicles during the evening peak hour, so that its ultimate peak hour volume will rise to 1, 800 vehicles immediately after the BON is closed, and drop off to 1, 650 vehicles after several weeks.

It is difficult to estimate precisely the travel time decrement that the increased loading of 250 vehicles per hour (or 4 to 5 vehicles per minute) will produce on a one-lane ramp. However, with several minor adjustments, Wingo's (3) ingression model can be applied to the problem. In his model

$$Y = \frac{N^2}{2C} \tag{15}$$

in which

Y = total ingression loss for the inflow phase of a cycle;
N = number of vehicles constituting the cyclic demand on the ramp; and
C = cyclic capacity of the ramp.

If N_1 and N_2, respectively, represent the cyclic demands on GON before and after BON is closed, the incremental ingression loss per cycle will be

$$\Delta Y = \frac{N_2^2 - N_1^2}{2\ C} \tag{16}$$

To simplify the arithmetical computations here, 1 min is arbitrarily set as the cycle length as a basis for treating the cyclical demand on GON, and the total change in the ingression loss for the 1-hr period is considered to be 60 times the value for the 1-min cycle. For a more precise analysis, the actual cyclic demand (which would be directly related to the cycle length, division, phase relationships of the traffic signals at adjacent intersections) should be considered, as well as variations in demand from cycle to cycle.

Using the simplifying assumptions, and considering that the capacity of the ramp is 1,000 vehicles per hour in Eq. 16, the change in ingression losses during the peak hour at GON due to increased demand is found to be 246 vehicle-minutes.

Effect on Present Users of the Freeway

Based on the pattern analysis described in the section on "Estimates of Flow Percentages for Different Input-Output Combinations of the System," the nature of the 4 to 5 p.m. "collection" can be approximated (Table 5). The problem here is to estimate how eliminating BON traffic will affect the travel time the remainder of the "collection" requires to negotiate the "system". As discussed in the section on "Travel Time Required to Negotiate System of Interest, and Any Inferred Diseconomies," the two realized travel time peaks in Figure 16 appear to be related most directly to ramp movements; capacity-type arguments do not appear to correlate too closely. Whatever caused them, the two peaks demonstrate that significant travel time diseconomies are being generated in the system.

To quantify the diseconomies, some "normal" travel time must be selected as a base from which the diseconomy is to be established. Several possible bases are plotted in Figure 16 including horizontal lines which portray speeds of 20, 30, and 60 mph being uniformly maintained by a vehicle proceeding from INB to OUB, and an inferred travel time curve based on RSP (OUB). Each point on this inferred TRT curve is a point C in Figure 4, and was determined by adding the TRT (INB. OUB) to CLT (INB) to obtain a CLT (OUB). The RSP was then measured at this CLT (OUB) and used to find the ordinate of the inferred TRT curve at the CLT (INB) abscissa. The inferred TRT curve thus is shifted on the clock time axis by the amount of the TRT (INB. OUB) for the collection at CLT(INB).

From the standpoint of the over-all concept, it is a matter of indifference whether one of the constant speed curves or the inferred curve is selected as the base from which the diseconomies are to be established. The method in every case would involve measuring the travel time difference between the realized curve and the base curve, and then weighting the difference by the number of vehicles that experienced that travel time decrement. The inferred TRT curve is used because it is keyed into actual measurements, RSP (OUB), while any constant speed curve would have been more or less arbitrarily selected.

It must be recognized that there also is an arbitrary aspect to the inferred TRT curve. Specifically, the curve is a direct function of the location of the output boundary. For example, if the boundary was at the beginning of an upgrade, one curve would have resulted, but there would have been a different one had the boundary been on a level tangent section, etc. However, the output boundary was chosen to fit the problem of operational interest (that is, performance of the "slot"), and not because of geometry of the freeway at that point. Thus the use of the inferred TRT curve is not completely arbitrary.

With the inferred TRT curve as a base, the macroscopic TRT diseconomy was found to be 11,451 vehicle-minutes during the hour 4 to 5 p.m. Remaining to be answered is the difficult question of how much of this is due to traffic from the on-ramp in question, and how much is due to the other ramps and different factors such as the presence of trucks, the grade (maximum 4.8 percent) of the freeway, lane changing, and others. The question will be answered indirectly in the next section by estimating how much the reduction in travel time would have to be to match the different increases in travel time associated with the hypothesized ramp closing.

Summary Analysis

The total negative travel time saving (that is, increase in travel time) is 1,316 vehicle-minutes in the 1-hr period. This is due to the ingression losses of the waiting stream at GON, and to the (maximum) time loss that present BON traffic will experience with the closing of BON. The total number of vehicles in the collection is approximately 6,000, so that the "break-even" point for the ramp-closing decision is that the average vehicle in the collection realizes a minimum TRT saving of 13 sec with the closing of BON.

At this point, and until the planned-for detailed statistical analysis actually partials the total "within" diseconomy of 11,451 vehicle-minutes into the components due to truck effects, volume effects, and effects of the separate ramps, it is strictly a matter of opinion as to whether or not the break-even point was reached in these data. The authors are of the opinion that the break-even point was substantially exceeded because: (1) most of the total diseconomy appears to be due to the two peaks, (2) ramp movements are the only factors which peak in the same manner, and (3) BON traffic represents approximately one-third of all of the ramp movements.

TABLE 3

THE "COLLECTION" DURING THE HOUR 4 TO 5 P.M.

Item	BONO	BONC	ΔTRT[1] (veh-min)
N (BON)	620	0	-
N (SAF. HLF)	1,990	1,990	5,337
N (SAF. HRF)	408	408	956
N (SAF. PSF)	128	128	300
N (SBF. HLF)	1,170	1,170	1,703
N (SBF. HRF)	1,220	1,220	2,360
N (SBF. PSF)	27	27	57
N (AON. HLF)	390	390	529
N (LON. HLF)	364	364	
N (AON. HRF)	127	127	154
N (LON. HRF)	54	54	
N (AON. PSF)	33	33	55
N (LON. PSF)	32	32	
Total			11.451

[1]ΔTRT is computed using the inferred TRT curve as the base.

Diversion Analysis

There remains one other consideration to complete the analysis; namely, how any present operational decision might be affected by completion of additional links of the programmed freeway network. By estimating the amount of traffic that will be diverted from the system of interest, the traffic volume that the system will have to handle presumably will be known. The issue then becomes one of deciding whether the present operational decisions still would be indicated under the ultimate volume conditions.

Based on the flow pattern analysis in the section on "Estimates of Flow Percentages for Different Input-Output Combinations of the System", it is estimated that with the completion of the freeway network in the Los Angeles region, 25 percent of the SAF traffic and 15 percent of the SBF traffic will be diverted from the "system of interest". Or, 17 percent of the present "system" traffic will be diverted, including 14 percent of the present truck traffic. Diversion patterns such as these have been estimated for many years; it is expected that with the flow pattern analyses providing additional information, the reliability of the estimates can be improved.

As discussed in the section on "Travel Time Required to Negotiate System of Interest, and Any Inferred Diseconomies", the relationships of TRT diseconomies to volume of traffic are not clearly recognized in the data obtained in this study. Consequently, it is difficult to estimate the amount by which the TRT diseconomy will be reduced even though it is possible to estimate that something of the order of 31, 000 vehicles will be diverted from the system.

One other point must be emphasized here. It is clear from the sections on "Spectrum of Trip Lengths According to On-Ramp of Origination", "Surface Street Travel Patterns of Ramp Users", and "Estimates of Flow Percentages for Different Input-Output Combinations of the System", that motorists trade distance for time in the region in question. If travel time through the system of interest is improved either as a consequence of traffic being diverted to other freeway links, or as a consequence of additional capacity being provided, additional traffic will be attracted to the "system" from the adjacent surface streets. This would mean more ramp traffic as well as through traffic. Thus, if the ramp effect observed at the present is real, it can be compounded rather than relieved with the completion of the links of the programmed network.

SUMMARY AND CONCLUSIONS

This paper presents a series of interrelated sub-studies which together comprise one approach to quantifying freeway performance. The fundamental dependent variable is travel time for an individual vehicle to negotiate any given link of a freeway network, the link being referred to as the "system" and being defined by a pair of arbitrarily-set boundaries. The measure of effectiveness is the summation of travel time that each of a group of vehicles requires to clear the system. The group of vehicles, called the "collection", is comprised of all vehicles arriving at all "input" boundaries of the system within some specified interval of clock time.

The quantification requires two basic classes of field measurements: first, the clock time identifications of vehicles at input and output boundaries of the system; second, closely controlled spot-speed and headway measurements at the output boundary. Derived from the license identifications is a questionnaire technique involving a direct mailing to registered owners of vehicles observed at specific ramp locations. The questionnaire serves to establish surface street paths that ramp users take in getting to or from ramps and the freeway use emanating from (or to) specific ramps. With these data, it becomes possible to extend the "system of interest" to include the adjacent surface streets, and to examine the effect of a change in one sub-system (for example, the freeway) on the total system.

Numerical results are obtained in the study for a part of the freeway network in downtown Los Angeles, but cannot be construed as representing the "system" for any period other than the day of the study. Furthermore, they are not necessarily representative of other sections of the freeway network in Los Angeles or elsewhere. Nevertheless, they are of more than passing interest insofar as they demonstrate that travel time models can produce unambiguous answers to real problems, and thereby suggest the festibility of using measures of effectiveness oriented around travel time.

The data in this study demonstrate that capacity-type arguments (in this case, flow at the output boundary) do not always fully explain travel time. Traffic flow theories oriented around capacity will not, therefore, by themselves portray performance of freeway networks, and it is suggested here that they be supplemented by theories based on travel time. There, of course, has been work on what might be referred to as theoretical "travel time" models of traffic flow (3). But the results of this paper suggest that the "travel time" phenomenon is more complex than might have been suspected, and warrants even more detailed attention than theorists have given it to date.

In addition to the theoretical work per se, there also must be "real world" measurements to validate (or negate) proposed theories and even, possibly, to suggest new theoretical avenues. Travel time models of traffic flow accordingly require measurement of travel time; capacity-type models require the much simpler counting of vehicles. This latter fact alone might explain why there has been much theoretical and applied work on capacity-type arguments, but relatively little on travel time. From this it follows that more attention should be given to the problem of measuring and analyzing travel time.

The license matching method offers substantial promise of bringing travel time field work within reasonable economic bounds, particularly with high speed electronic computers accomplishing the actual matching, subtracting clock times, detailed cross tabulations, etc. Another significant economic justification is that while producing the travel time analyses, the matching method also can yield the travel pattern analyses at essentially no additional cost. The most difficult aspect of the technique is, and will continue to be, identifying the license of vehicles moving at high speeds. Significant improvements are possible in the photographic techniques. But the full potential of the method will probably not be realized until the license plates themselves have been designed specifically to facilitate the numbers being transduced automatically into form suitable for computer input.

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REFERENCES

1. Deen, T.B., "The Use of Travel Time as a Factor in Rating Urban Streets." Traff. Engr. 30:4, 13-18, 24 (Jan. 1960).
2. Rothrock, C.A., and Keefer, L.A., "Measurement of Urban Traffic Congestion." HRB Bull. 156, pp. 1-13 (1957).
3. Wingo, L., "Measurement of Congestion in Transportation Systems." HRB Bull. 221, pp. 1-28 (1959).
4. Walker, W.P. "Speed and Travel Time Measurement in Urban Areas." HRB Bull. 156, pp. 27-44 (1957).
5. Berry, D.S., and Van Til, C.J. "A Comparison of Three Methods for Measuring Delay at Intersections." Traff. Engr. 25:3, 93-100 (Dec. 1954).
6. Webster, F.V., "Traffic Signal Settings." Road Research Tech. Paper No. 39, 44 pp., Road Research Lab., Harmondsworth, Middlesex, England (1958).

7. Johnston, W.W., "Travel Time and Planning." Traffic Quart. X:1, 67-79 (Jan. 1956).
8. Pipes, L.A., "An Operational Analysis of Traffic Dynamics." Jour. of Appl. Physics, 24:274-281 (1958).
9. Lighthill, M.J., and Whitham, G. B., "On Kinematic Waves II. A Theory of Traffic Flow on Long Crowded Roads." Proc., Royal Society (London), Series A, 48:227-289 (1952).
10. Haight, F.A., "Mathematical Theories of Road Traffic." Special Report, ITTE, Univ. of California, Los Angeles, 42 pp. (March 1960).
11. Brenner, R., Mathewson, J.H., Gerlough, D.L., "A General Method for Estimating Through Traffic in a Road Network." HRB Abstracts, 27:8, 32-48 (Sept. 1957).
12. Mathewson, J.H., Brenner, R., and Reiss, R.J., "A Segmented Electrical Element for Detecting Vehicular Traffic." HRB Proc., 29:374-383 (1949).
13. Burch, J.S., "Gasoline Taxes and the Small Car." Traff. Quart., 14:2, 221-5 (April 1960).
14. Gazis, D., Herman R., and Maradudin, A., "The Problem of the Amber Signal Light in Traffic Flow." General Motors Corporation, Warren, Mich., 22 pp.
15. Mosher, W.W., Jr., "Computer Program for Determining Traffic Patterns and Travel Time in a Generalized Network." Tech Memo L-15, ITTE, Univ. of California, Los Angeles.
16. Mathewson, J.H., and Brenner, R., "An Investigation of Through Traffic in a Surface Street Network." ITTE, Univ. of California, Los Angeles (March 1960).
17. Webb, G., and Moskowitz, K., "California Freeway Capacity Study—1956." HRB Proc., 36:587-641 (1957).
18. Spangler, F.W., and Beilfus, H.R., "A High-Speed Black and White Negative Film." Jour. of the SMPTE, 69:10, 742-4 (Oct. 1960).

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