HIGHWAY RESEARCH RECORD

Number 152

Geometric Design, Barrier Rails and Sign Supports

5 Reports

Subject Classification

22 Highway Design
51 Highway Safety
53 Traffic Control and Operations

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Washington, D.C., 1967 Publication 1428

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Foreword

Engineers who design highways find that they must be alert to developments in a number of related fields. The five papers in this RECORD present information in several of these fields including geometric design, access violations, accidents, barrier rails and sign supports. While improved traffic flow and structural security are covered, the principal message of the papers in the record is traffic safety.

The paper by Tipton and Pinnell reports on the effect of various arrangements of ramps connecting freeways with continuous frontage roads in Texas. The authors find that standard interchange designs cannot always fulfill the various desired movements at different interchanges. In general, the sequence of an off-ramp located upstream from an on-ramp is preferable to the reverse arrangement.

To find a solution to the high accident rate on a six-lane elevated expressway seven miles in length in Montreal, the Highway Department of the Province of Quebec conducted impact tests on eight barrier systems that modify or replace the existing barriers. Hénault and Perron report that as a result of these tests a 12-in. high reinforced-concrete wall was placed on the existing curb in front of the existing double tube and post rail which remained in place. The selection was based mainly on the very small rebound of the impacting vehicle. This improvement in combination with improved lane marking and a speed limit reduction from 55 to 45 mph have produced a small reduction in overall accidents and a great reduction in fatalities in the face of steadily increasing traffic volumes.

For a freeway to fulfill its potential for safe, comfortable and convenient travel, unauthorized access must be prevented. Tipton, Drew and Spencer report on a study of freeway access violations in Texas. The three most prevalent violations were separation strip crossing exits, median crossings and separation strip crossing entrances. The most common purpose of a violation was to get to or from home. Curbs, chain-link fences and posts with barrier cables were highly effective but prohibitive signs were relatively ineffective.

Huelke and Gikas present a report on the 177 automobile occupants who were killed in 139 accidents in Washtenaw County, Michigan, in four years prior to November 1, 1965. Eighty of these were single car accidents, 58 involving impact of a roadside element. The authors present illustrations of accident-producing conditions and distribution curves for distance of obstacles from the edge of the roadway. In this study, the farthest from the pavement edge of the obstacles involved in fatal impacts was 32 feet.

In November 1957, a long-term load test was undertaken in Ohio on concrete sign support foundations subjected to an overturning moment. A general description of the installation and instrumentation and operations to July 1958 were published in Highway Research Board Bulletin 247. Mr. Behn now completes the record on this project by presenting the observations to July 1965. A short foundation was installed in very poor organic soil for information only and has deflected almost three degrees. In all other cases, with moments up to 195,000 ft-lb, the angular movement observed has been less than one-half degree.

Contents

An Investigation of Factors Affecting the Design Location of Freeway Ramps

WILLIAM EARL TIPTON, Formerly Research Assistant,
CHARLES PINNELL, Head, Design and Traffic Department, Texas Transportation Institute, Texas A and M University

•MANY freeways within our major cities are entering a critical phase of utilization. These facilities are becoming congested during peak periods and are not providing the "level of service" for which they were designed. All possible courses of action should be undertaken to improve the efficiency of freeway operation so that a desired level of service can be maintained.

Past studies aimed at improving the efficiency of operations have primarily dealt with the design and operation of an on-ramp, the design and operation of an off-ramp, or the weaving on the freeway resulting from an on-ramp closely preceding an off-ramp (1-10). Existing freeway interchanges have been designed using the current "best" design for each of the ramps, but the location and configuration of the ramps have for the most part been accomplished in a standardized manner.

STATEMENT OF THE PROBLEM

Ramp location, as used herein, was defined as the location of a ramp or ramps upstream or downstream of an arterial street crossing the freeway. Ramp configuration was defined as the order in which closely spaced pairs of ramps appear. A pair of ramps includes an on-ramp and an off-ramp; therefore, a ramp configuration would be an off-ramp closely followed by an on-ramp or vice versa. Stacked ramps, a modification of the off-ramp followed by an on-ramp configuration, exist in the form of grade-separated ramps (Fig. 1).

Names of interchange designs have resulted from the standardization of ramp configuration. The most prominent of these are the X interchange and the diamond interchange. The X interchange includes an on-ramp upstream of the arterial street and off-ramp downstream of the arterial street for both the inbound and the outbound directions of travel. As illustrated in Figure 2, these 4 ramps form an X from which this type of interchange derived its name. In the diamond interchange, the ramps are the reverse of those in the X interchange, and the 4 ramps form a diamond. This type of interchange is also shown in Figure 2.

To design interchanges properly, the ramps must be located in such a manner as to fulfill the estimated future needs of traffic and provide a minimum of interference to the freeway traffic. This research investigated the operation of several existing layouts and the suitability of different layouts being used at these locations. The stacked ramp configuration was investigated as a possible solution when both an on-ramp and an off-ramp were required at the same location.

This research was a portion of a larger project, "The Effects of Off-Ramps on Freeway Operation," which was conducted by the Texas Transportation Institute in cooperation with the Texas Highway Department and the U. S. Bureau of Public Roads.

Study Objectives

The objectives of this phase of the project were to investigate:

1. The desired movement of entering and exiting traffic at diamond or X-type interchanges;

Paper sponsored by Committee on Geometric Highway Design and presented at the 45th Annual Meeting.

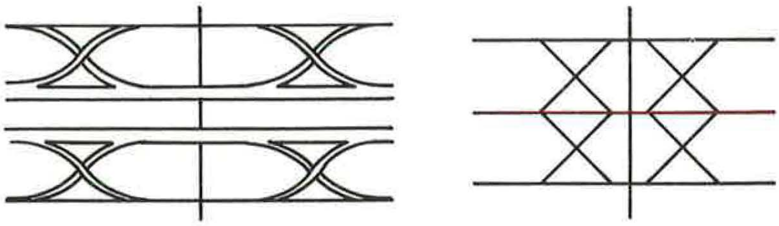


Figure 1. Stacked ramps.

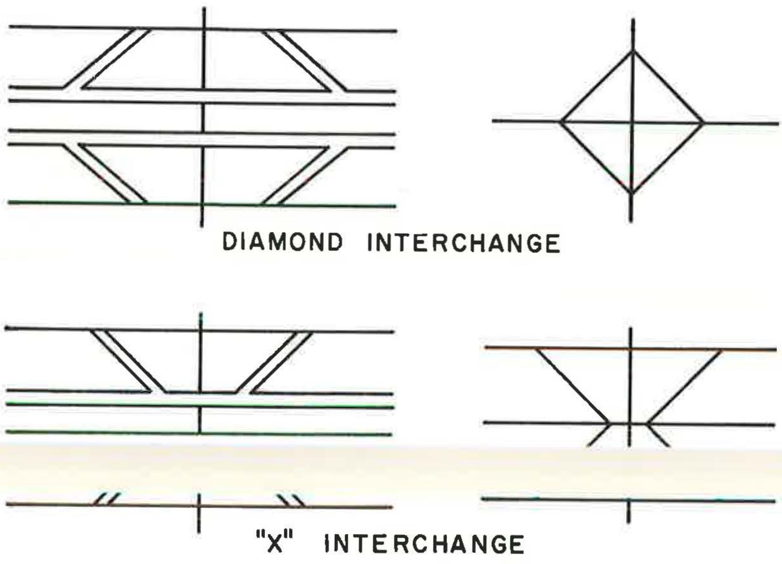


Figure 2. Interchange types.

2. The effect of freeway ramp configuration on the amount of acceptable gap time available to vehicles desiring to enter the freeway at a specific ramp, in order to determine the more desirable ramp configuration;
3. The effect on the amount of acceptable gap time as the distance downstream of an off-ramp increased, in an attempt to develop criteria for ramp spacing; and
4. The suitability of various interchange layouts in fulfilling drivers' desires, providing access to the freeway and abutting property, and reducing the interference to freeway and arterial street traffic.

Study Site

All of the studies for this research took place on the Gulf Freeway in Houston, Texas. This freeway is a 6-lane facility divided by a 4-ft barrier type median. The grade of the Gulf Freeway is near ground level with the exception of the interchanges and railroad crossings. At these locations the freeway rises to pass over an arterial street or railroad. This up and down movement creates a "roller coaster" effect which is shown in the aerial photograph in Figure 3. For the most part, continuous frontage roads parallel this facility. The study sites were located between Dowling Street, which is 2 miles from the central business district (CBD), and the Reveille Interchange, which is 6 miles from the CBD. Figure 4 shows the study area and the freeway layout.

Figure 3. Gulf Freeway, Houston, Texas.

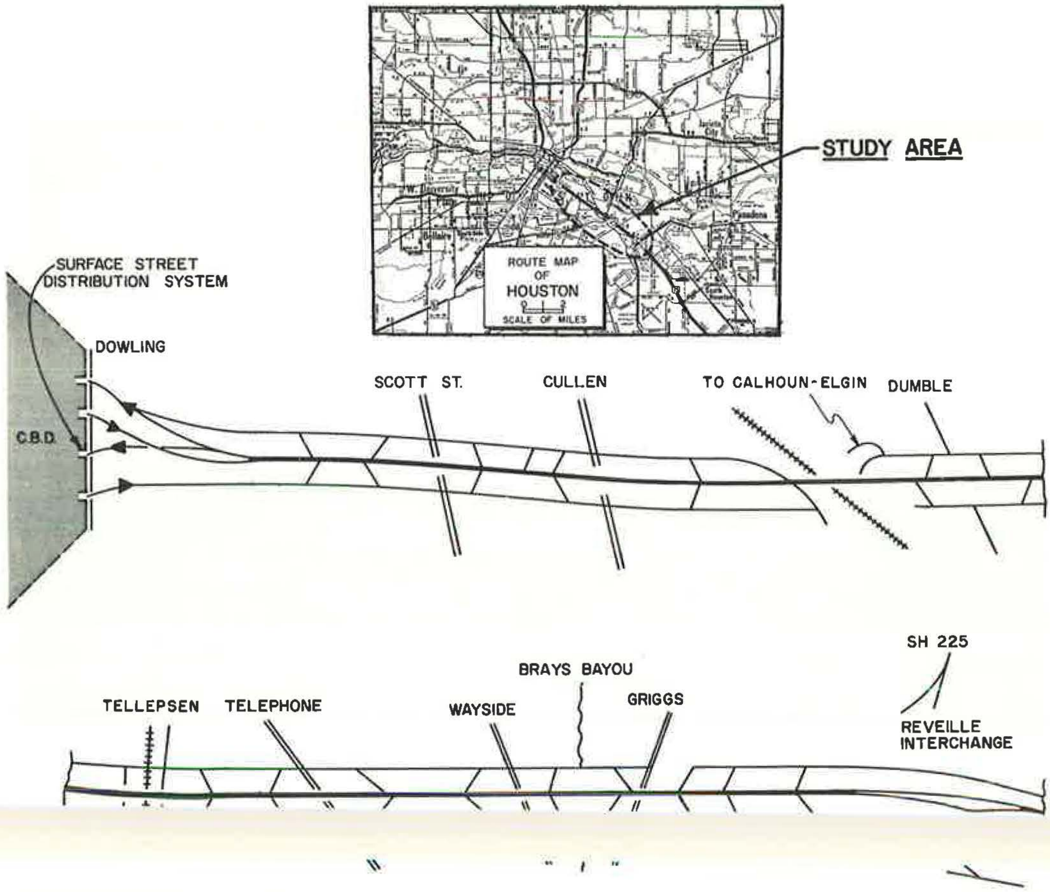


Figure 4. Study area—Gulf Freeway, Houston, Texas.

DRIVERS' DESIRES AT INTERCHANGES

The investigations of the desired movement of entering and exiting traffic at various interchanges were conducted to determine if drivers' desires were the same at most interchanges. If they were, the indication would be that a standard type of interchange (with standard ramp locations) could fulfill drivers' desires, and the procedure of using a standard type of interchange along a section of freeway would be justified. If drivers' desires were not the same at all interchanges, the indication would be that each interchange layout should be based on the anticipated traffic desires for that interchange, and the ramps placed according to these desires.

Method of Study

Drivers' desires at each of the interchanges studied were determined by a license plate survey. The survey was divided into 4 studies to investigate each possible desire. These studies were: (a) Study 1—The Desire To Exit Downstream of the Arterial Street, (b) Study 2—The Desire To Exit Upstream of the Arterial Street, (c) Study 3—The Desire To Enter Upstream of the Arterial Street, and (d) Study 4—The Desire To Enter Downstream of the Arterial Street.

Data for each of these studies were collected at the following interchanges: (a) Cullen Interchange outbound, (b) Telephone Interchange outbound, (c) Wayside Interchange outbound, (d) Woodridge Interchange outbound, and (e) Cullen Interchange inbound. The data collection periods were from 4:00 to 5:30 p.m. at the first 4 interchanges and from 6:30 to 8:00 a.m. at the fifth interchange.

With one exception (Wayside Interchange), the data for the 4 studies were collected at diamond interchanges. As noted previously, a diamond interchange has an off-ramp upstream of the arterial street and an on-ramp downstream of the arterial street. Studies 1 and 3 were conducted at diamond interchanges even though the ramps fulfilling the desires in question did not exist. These desires were determined by recording the license plate number of vehicles that could have used ramps, had they existed, and matching these license plate numbers to those recorded at the ramps actually used. The procedure used for each study is shown in Figure 5 and is explained in detail below.

Study 1: Desire To Exit Downstream of Arterial Street. —License plate numbers were recorded at Points A and B (Fig. 5). Point A was on the existing off-ramp, and Point B was located on the frontage road 500 ft downstream of the bridge abutment. Point B was chosen as the nearest location to the arterial street which could be served by an off-ramp located downstream of the arterial street. The amount of license plate numbers matched between Points A and B was the extent of the desire to exit downstream of the arterial street.

Study 2: Desire To Exit Upstream of Arterial Street. —License plate numbers of vehicles using the off-ramp, Point A, were recorded entering private property and access streets, Point E, and turning left, Point D, or right, Point C, onto the arterial street (Fig. 5). The amount of license plate numbers matched between Point A and Points C, D, and E was the extent of the desire to exit upstream of the arterial street.

Study 3: Desire To Enter Upstream of Arterial Street. —License plate numbers were recorded at Points F and G. Point F was located on the frontage road 700 ft upstream of the bridge abutment. This point was used as the nearest location to the arterial street for which an on-ramp upstream of the arterial street could provide access. Point G was located on the existing on-ramp downstream of the arterial street. The extent of the desire to enter upstream of the arterial street was determined by the amount of license plate numbers matched between Points F and G.

Study 4: Desire To Enter Downstream of Arterial Street. —License plate numbers were recorded of vehicles entering the frontage road from private property and access

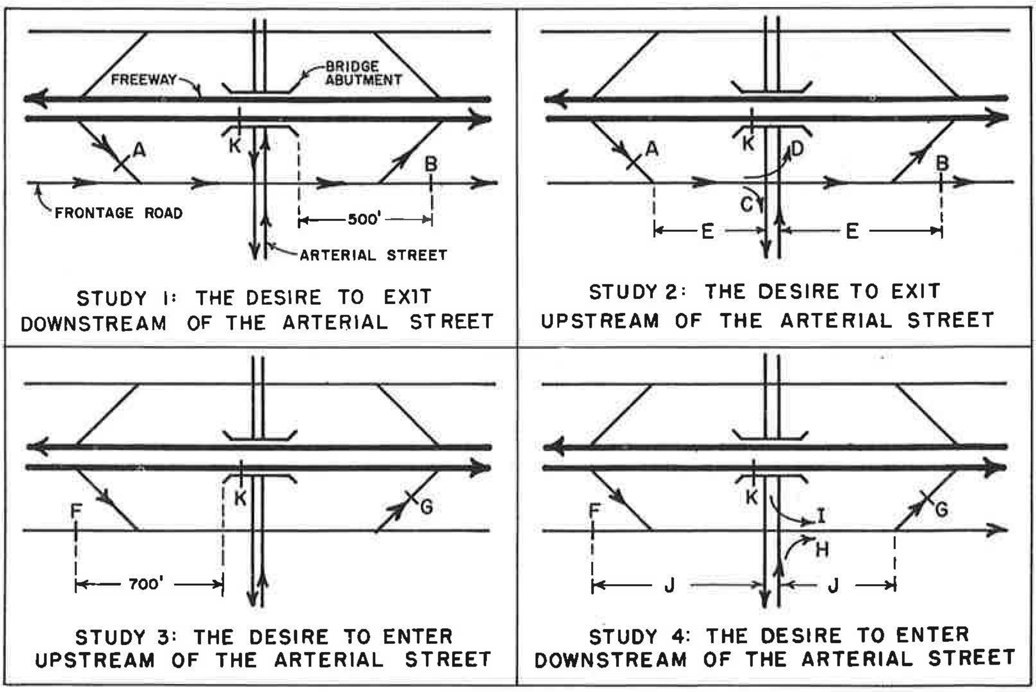


Figure 5. License plate recording points.

streets, Point J, turning left, Point I, and right, Point H, from the arterial street onto the frontage road, and entering the on-ramp, Point G. The amount of license plate numbers matched between Point G and Points H, I, and J was the extent of the desire to enter downstream of the arterial street.

In addition to the license plate survey, the freeway volume crossing the overpass in the direction of travel under study, Point K, was counted in 5-min periods to furnish an indication of freeway operation during the study. Data were collected for all 4 studies simultaneously at each interchange to avoid unnecessary duplication of recording points.

Some method of determining if traffic desired a specific ramp was required. It was decided that if the extent of the drivers' desires for a ramp was greater than 100 during the peak hour, the ramp would be deemed to be desired. This value is not necessarily practical or to be construed as a warrant for the construction of a ramp. In all cases the actual desires are indicated so that the individual reader may evaluate the situation according to his own judgement.

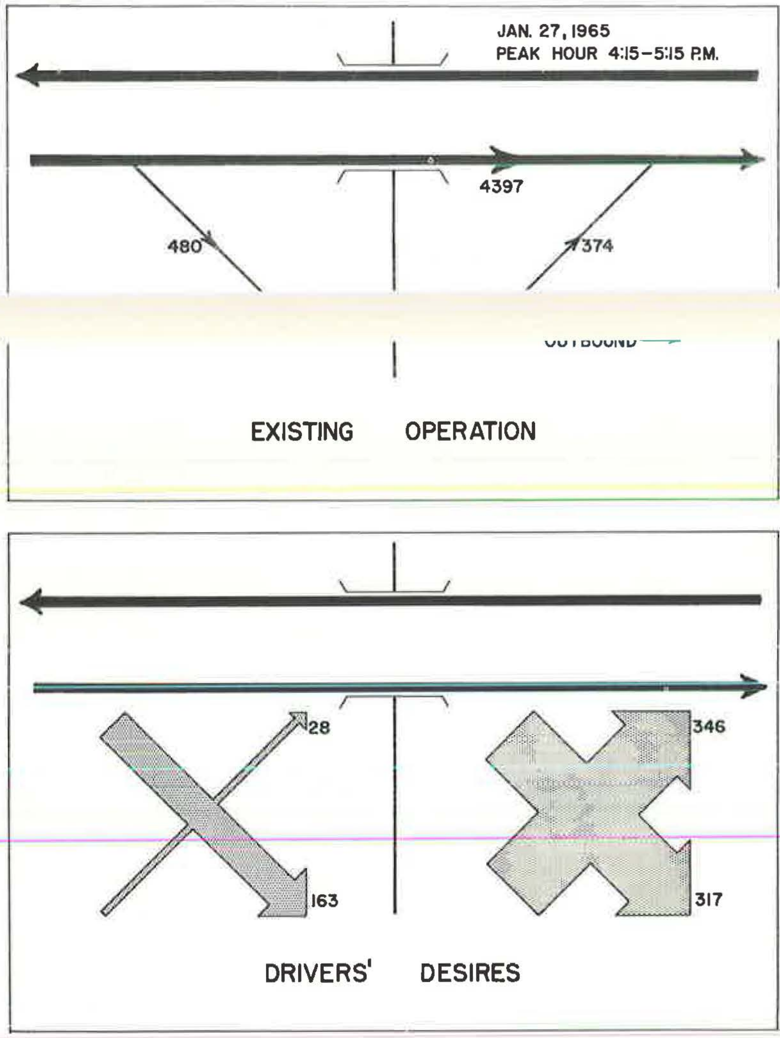


Figure 6. Cullen Interchange outbound.

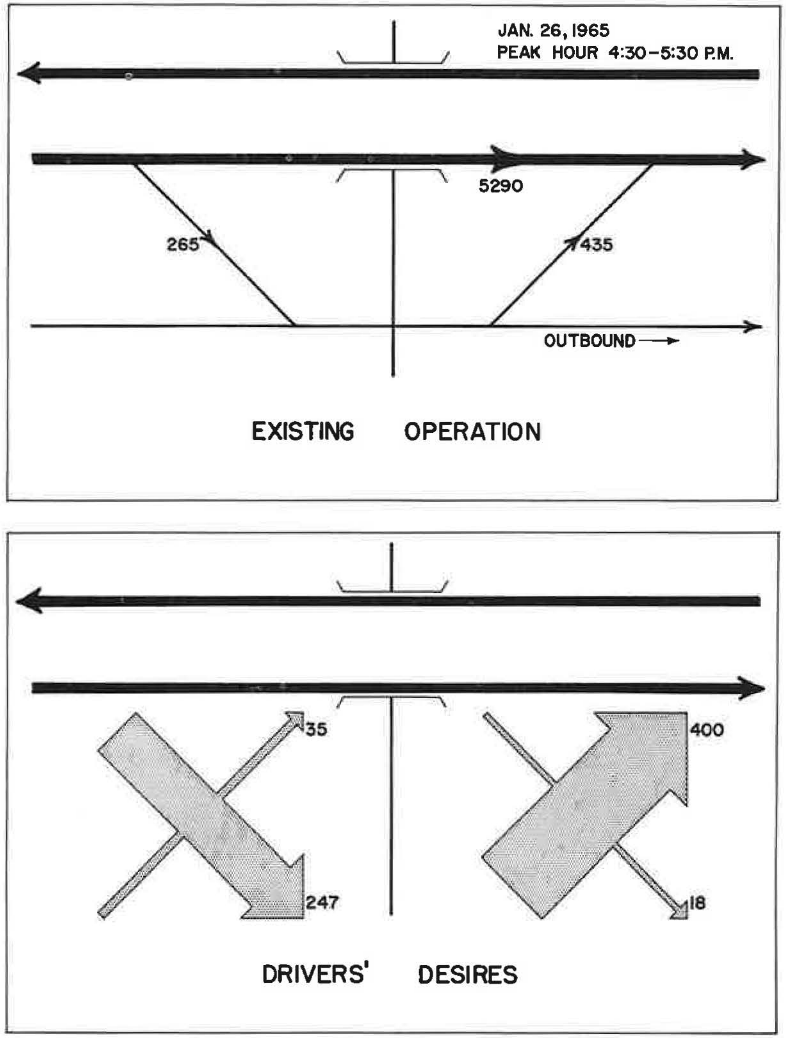


Figure 7. Telephone Interchange outbound.

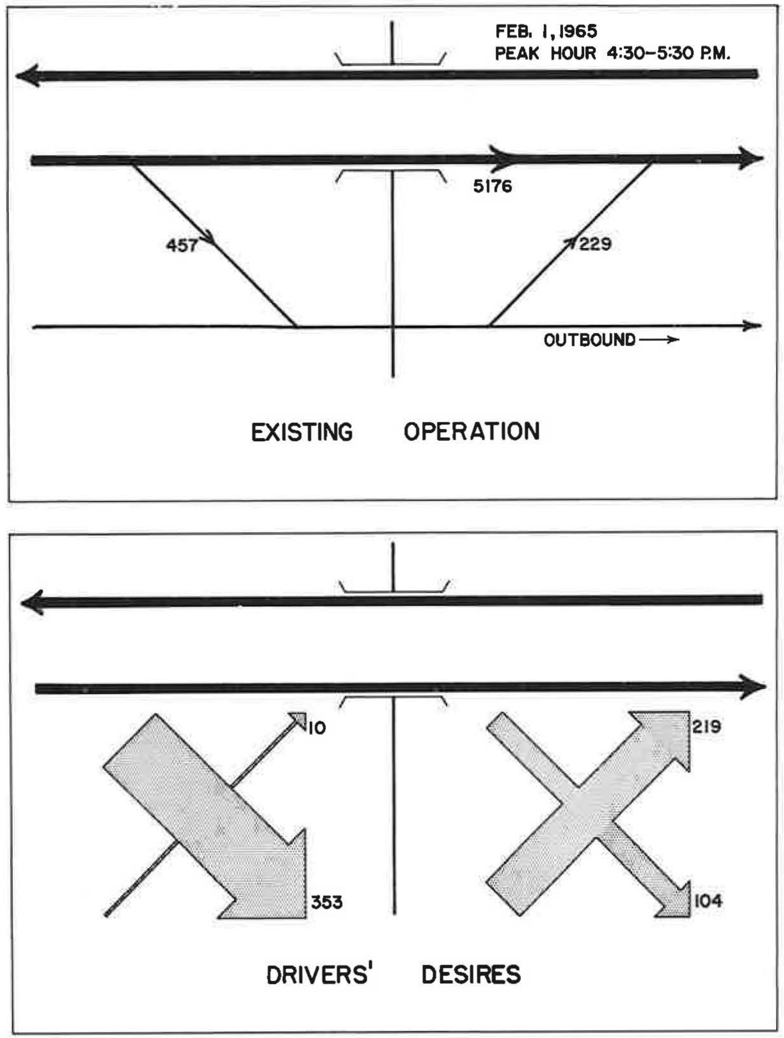


Figure 8. Woodridge Interchange outbound.

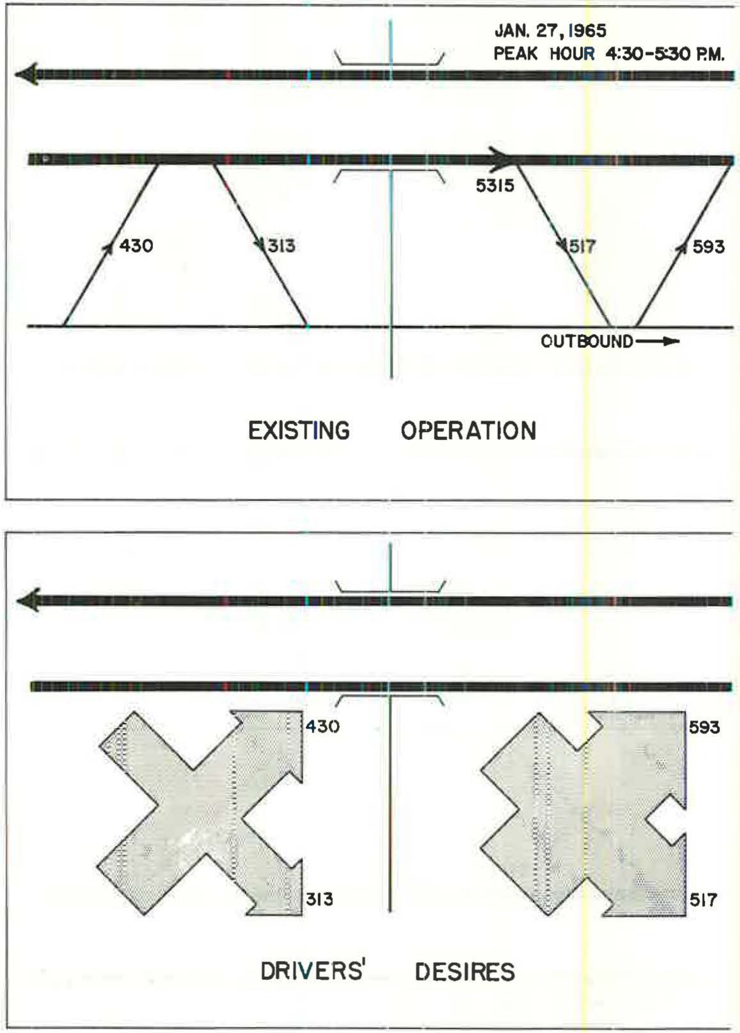


Figure 9. Wayside Interchange outbound.

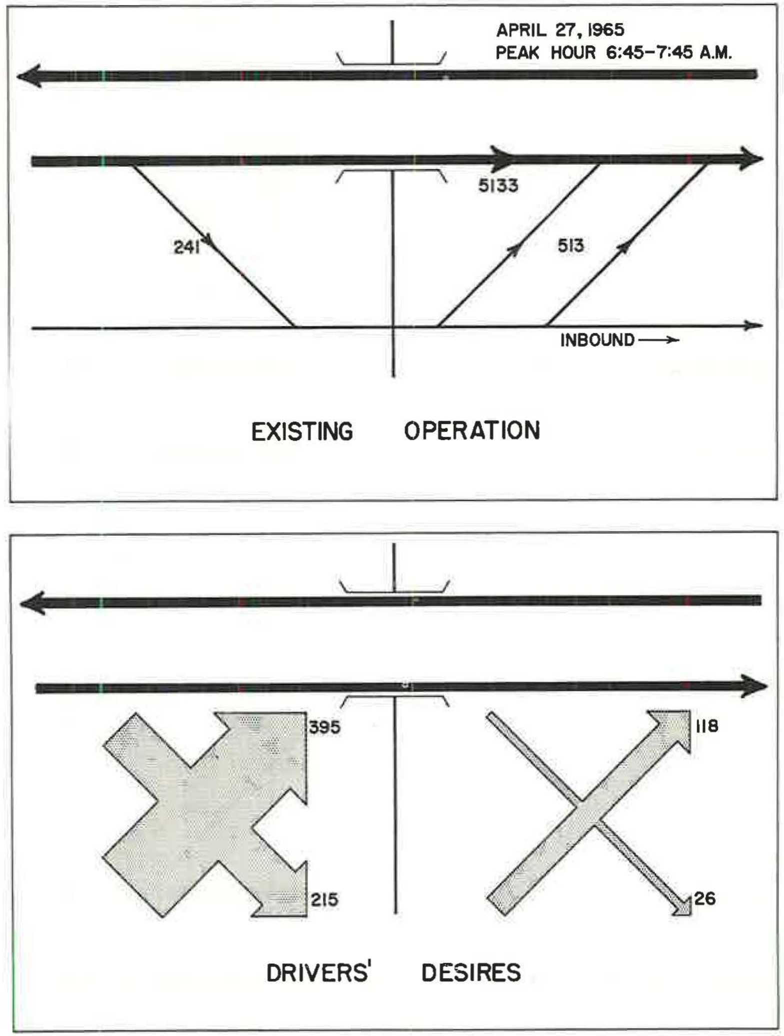


Figure 10. Cullen Interchange inbound.

Discussion of Results

Cullen Interchange Outbound. —The results of the investigation of drivers' desires at the Cullen Interchange outbound are shown in Figure 6. These desires indicated that an off-ramp located downstream of the arterial street was desired in addition to the existing ramps. Thus, at this interchange, traffic desired an off-ramp upstream of the arterial street and an on-ramp and an off-ramp downstream of the arterial street.

Telephone Interchange Outbound. —The traffic desires at the Telephone Interchange outbound are shown in Figure 7. These desires indicated that only the existing ramps were desired. At this interchange, an off-ramp located upstream of the arterial street and an on-ramp located downstream of the arterial street were desired.

Woodridge Interchange Outbound. —At the Woodridge Interchange outbound, drivers' desires indicated that an off-ramp downstream of the arterial street was desired in addition to the existing ramps. The traffic desires are shown in Figure 8. Therefore, an off-ramp upstream of the arterial street, and an on-ramp and an off-ramp downstream of the arterial street were desired at this interchange.

Wayside Interchange Outbound. —Drivers' desires at the Wayside Interchange outbound are shown in Figure 9. These desires indicated that each of the ramps in the existing interchange was desired. (The existing interchange was assumed to have included the on-ramp downstream of Telephone Road.) Thus, an on-ramp and an off-ramp were desired upstream and downstream of the arterial street.

Cullen Interchange Inbound. —The results of the investigation of drivers' desires at the Cullen Interchange inbound are shown in Figure 10. The desired movements indicated that an on-ramp was desired upstream of the arterial street in addition to the existing off-ramp, and that one of the existing on-ramps located downstream of the arterial street was desired. Therefore, at this interchange, an on-ramp and an off-ramp were desired upstream of the arterial street, and one on-ramp was desired downstream of the arterial street.

Conclusions

The results of the investigation of drivers' desires at interchanges illustrated that the desires differed at the 5 interchanges studied, and that various combinations of ramps were required to fulfill these desires. The desired ramp locations are given in Table 1. It was concluded that:

1. Standard interchange designs could not always fulfill the desired movement of traffic.
2. The desired movements of traffic could be fulfilled by individual consideration of the desires at each interchange and the placement of the ramps according to these desires.

TABLE 1

DESIRED RAMP LOCATIONS

Desired Ramp	Cullen Interchange Outbound	Telephone Interchange Outbound	Woodridge Interchange Outbound	Wayside Interchange Outbound	Cullen Interchange Inbound
An off-ramp located downstream of the arterial street	Yes[a]	No	Yes[a]	Yes	No
An off-ramp located upstream of the arterial street	Yes	Yes	Yes	Yes	Yes
An on-ramp located upstream of the arterial street	No	No	No	Yes	Yes[a]
An on-ramp located downstream of the arterial street	Yes	Yes	Yes	Yes	Yes

[a]This ramp was desired, but it does not exist.

FREEWAY RAMP CONFIGURATION

The effect of freeway ramp configuration on the amount of acceptable gap time available to vehicles desiring to enter a freeway at a specific on-ramp was investigated in order to determine the more desirable ramp configuration. In the past it has been assumed that the greatest amount of acceptable gap time available to vehicles desiring to enter the freeway would be provided by removing off-ramp traffic before allowing on-ramp traffic to enter. This research tested that assumption to determine if it was valid and to evaluate the advantage to freeway operation that might result.

In this research an acceptable gap was defined as a gap an average driver would accept when entering a freeway. The selection of an acceptable gap time for an average driver was not critical as used in this research because the same basis of comparison was used for each configuration. An average value of 3 sec was chosen.

Theoretical Gap Distributions

To determine the effect of freeway ramp configuration on the amount of acceptable gap time available, theoretical gap distributions were fitted to the observed data. The exponential distribution can be fitted to the observed distribution of gaps for free-flowing volumes, but it is unsatisfactory for high volumes because of 2 conditions: (a) vehicles have length and must follow each other at some minimum headway, and (b) vehicles cannot pass at will even on a freeway. Gerlough (11) proposed that the first condition be overcome by shifting the exponential curve to the right an amount equal to a certain minimum headway, T. The probability of a gap greater than t then becomes

$$P\,(g > t) = e^{-(t - T)/(\bar{t} - T)}$$

To overcome the second condition, it was proposed by Schuhl (12) that the traffic stream be considered as composed of a combination of free-flowing and constrained vehicles. Haight (13) suggested that gaps less than the minimum headway, T, be considered improbable, whereas the shifted exponential considered them impossible. [illegible] their maximum probability at the origin and then decline as t approaches infinity. Therefore, these distributions imply that the smaller the gap, the more likely it is to occur. This implication is in error, and it was recently proven to be in error by May (14). Thus, the exponential distribution was not used in this research.

The Pearson Type III and the Erlang distributions were used in this research since they overcome the aforementioned conditions. These distributions are 2-parameter generalizations of the exponential distribution. The Pearson Type III and the Erlang distribution frequency functions are determined by multiplying the exponential distribution frequency function by some appropriate power of t (15) which gives

$$f\,(t) = \frac{t^{a-1}}{(a-1)\,!}\,(qa)^{a}\,e^{-aqt}$$

The difference between the Pearson Type III and the Erlang distributions was that for the Erlang distribution; the value of a was rounded to the nearest integer before it was used in the frequency equation. The 2 parameters used in this research were the mean and the variance. The mean was used because it influenced the location of the curve, and the variance was used because it influenced the shape of the curve.

Some difficulty was encountered in fitting the theoretical distributions to the observed data. It was found in some instances that neither theoretical distribution (Pearson Type III or Erlang) could be fitted to the data observed in one-sec intervals, and that the distributions sometimes could be fitted to the same data observed in 2-sec intervals. This was also noted by Gerlough (16) who stated, "Some traffic phenomena may be random when observed for an interval of one length but non-random when observed with an interval of a different length."

The chi-square test at the 5 percent level of significance was used to test the hypotheses that the theoretical distributions fitted the observed data.

Method of Study

The study procedure used in the investigation of freeway ramp configuration was a test of the hypothesis that the greatest amount of acceptable gap time available to vehicles desiring to enter the freeway was furnished by removing off-ramp traffic before allowing on-ramp traffic to enter. These studies investigated 2 ramp configurations. They were Case 1—an off-ramp located upstream of an on-ramp, and Case 2—an on-ramp located upstream of an off-ramp. These configurations are shown in Figure 11.

A comparison of the total amount of acceptable gap time available at a Case 1 and a Case 2 ramp configuration was desired. For such a comparison to be valid, the study conditions at each location must have been approximately the same. Thus the lane 1 (right lane) freeway volume, Point A in Figure 11, and the off-ramp volume, Point B, at a Case 1 configuration must have been approximately equal to the respective volumes at a Case 2 configuration. Up to a 10 percent difference in the respective volumes was allowed since it was felt that this amount would not significantly alter the results. Using this procedure, the effects of ramps upstream of the study area were minimized.

Data Collection

Data were collected twice at each study location. Case 1 studies were conducted at the following locations: the Griggs off-ramp and the Wayside on-ramp—outbound, and the Calhoun-Elgin off-ramp and the Dumble on-ramp—inbound. Case 2 studies were conducted at the following locations: the Scott on-ramp and the Cullen off-ramp—outbound, and the Tellepsen on-ramp and Telephone off-ramp—outbound. For both cases, the gaps in lane 1 (right lane) of the freeway were measured just upstream of the nose of the entrance ramp. In this manner the total amount of gap time available on the freeway for entering vehicles was determined. The points of data collection for each case are illustrated in Figure 11. A 176-ft speed trap was established between

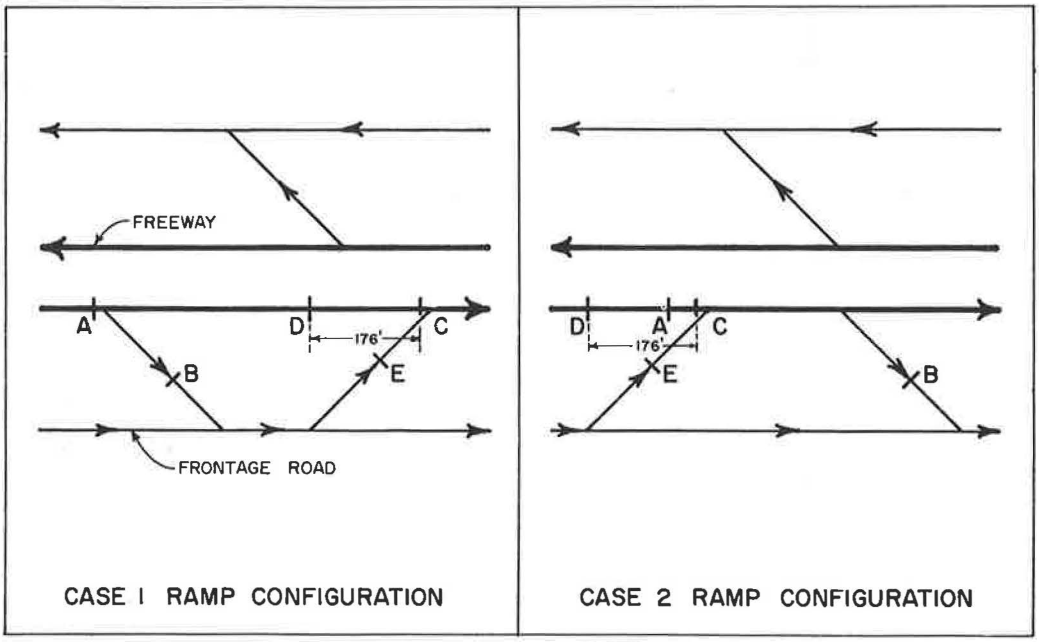


Figure 11. Data collection points.

Points D and C to determine the lane 1 speeds during the study. The freeway volume and the lane 1 volume in the direction of travel under study were counted at Point A, upstream of the first ramp for both cases. The off-ramp volume, Point B, and the on-ramp volume, Point E, were counted during the study.

An Esterline-Angus 20-pen recorder (Fig. 12) was used to record the volume counts, the gap times, and the travel times through the speed trap. The pens were used as follows:

1. Pen No. 1 was used at the beginning of the speed trap at Point D, 176 feet upstream from the nose of the on-ramp, to record when the front bumper of each vehicle in lane 1 passed the beginning of the speed trap.

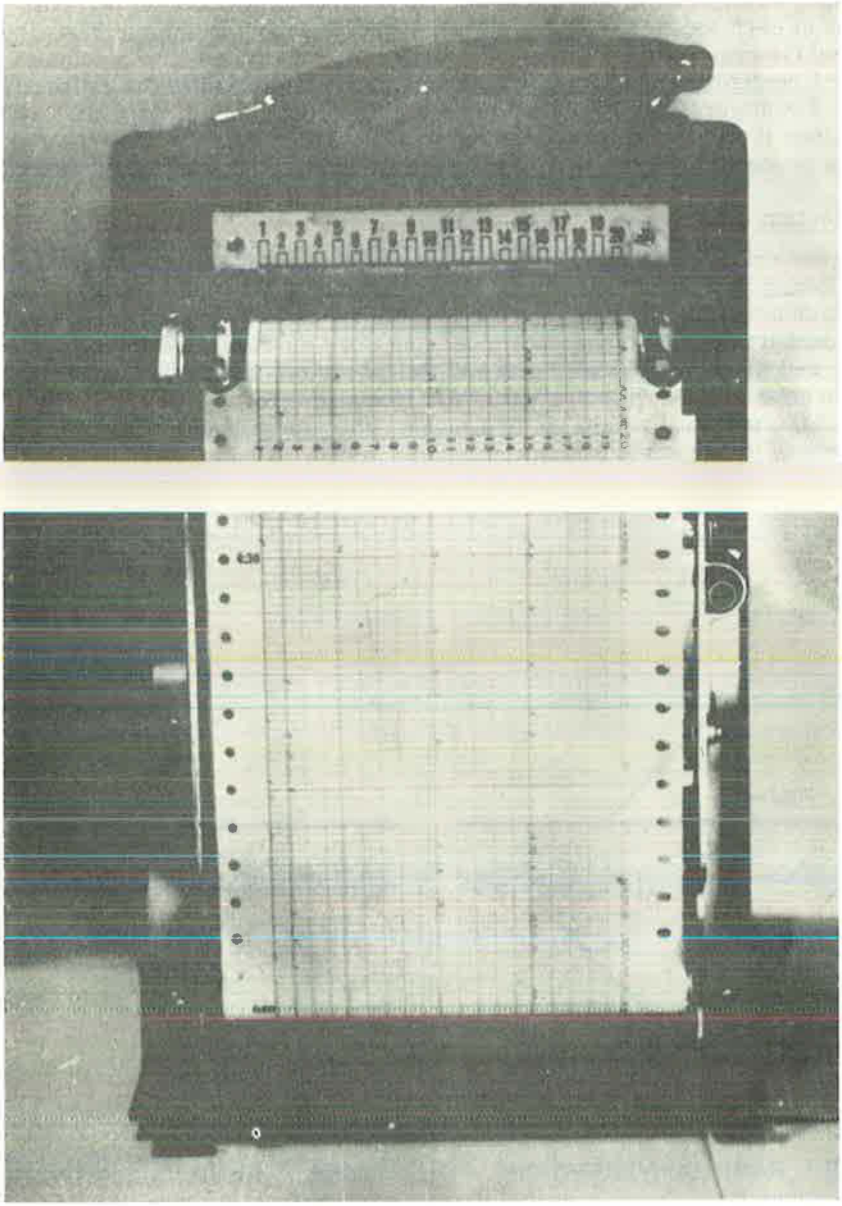


Figure 12. Esterline-Angus 20-pen recorder.

2. Pen No. 2 was used at the nose of the on-ramp, Point C, to record when the front bumper of each vehicle in lane 1 passed the nose of the on-ramp, to end the speed trap, and to measure the gaps in units of time between successive vehicles in lane 1.
3. Pen No. 5 was used at Point E to record the on-ramp volume.
4. Pen No. 10 was used at Point B to record the off-volume.
5. Pen No. 15 was used at Point A to record the volume count of lane 1 upstream of the first ramp.
6. Pen No. 20 was also used at Point A to record the 3-lane freeway volume in the direction of travel under study.

The recorded information was reduced and placed on IBM cards for the data analysis. The freeway gaps were measured to the nearest one-tenth of a second. One IBM card was used for each vehicle. This card contained a vehicle number, a gap time, and a travel time through the speed trap for that vehicle. Each card was also coded with information to identify the study site, date, type of study, length of speed trap, and time of start of the study. The frequency of the gaps is given in Appendix A. The freeway volumes recorded were counted and tabulated in 5-min periods for use in the data analysis.

Data Analysis

Periods were selected from the data which could be compared according to the requirements discussed in the Method of Study. Comparisons resulted between data collected at: (a) Scott and Cullen—outbound, and Griggs and Wayside—outbound; and (b) Tellepsen and Telephone—outbound, and Calhoun-Elgin and Dumble—inbound. Table 2 indicates the validity of these comparisons by providing the lane 1 volume recorded at Point A, the off-ramp volume recorded at Point B, and the respective percent differences of these values for each comparison.

Using a data observation interval of one second, the Pearson Type III and the Erlang distributions failed to fit the Tellepsen-Telephone and the Calhoun-Elgin and Dumble data. The data observation interval was increased to 2 sec, and the Pearson Type III distribution was found to fit both sets of observed data for the 50-min periods to be compared. The time periods of the data, the interval of the observed data, the value of chi-square, the degrees of freedom (d.f.) and the significance of the chi-square tests are given in Table 3.

The Pearson Type III and the Erlang distributions, when using a 2-sec data observation interval, failed to fit the Griggs-Wayside data and the Scott-Cullen data for the 55 min of data to be compared. Since these data were collected at a time very close to the afternoon peak period, 5-min periods of data were used so that a change in the traffic characteristics would not occur, making a fit of a distribution to these data impossible. Attempts were made to fit a distribution to 2 different 5-min periods of data from each

TABLE 2

COMPARISON OF LANE 1 AND OFF-RAMP VOLUMES

Location	Case	Date	Time Period (p.m.)	Avg. 5-Min Off-Ramp Vol.	Avg. 5-Min Lane 1 Vol.	Freeway Vol.	Lane 1 Avg. Speed
Calhoun-Elgin and Dumble—Inbound	1	1/12/65	2:30-3:20	29.2	75.6	250	47.5
Tellespen and Telephone—Outbound	2	1/15/65	1:15-2:05	29.7	80.4	266	48.3
				Diff = 2%	Diff = 6%		
Griggs and Wayside—Outbound	1	1/12/65	4:55-5	55.0	112.0	419	47.5
Scott and Cullen—Outbound	2	1/13/65	4:10-4:15	53.0	112.0	381	49.0
				Diff = 3%	Diff = 0%		

TABLE 3

CHI-SQUARE TESTS RESULTS

Study Location	Case	Time Period (p.m.)	Data Interval (sec.)	Chi-Square Tests Results			
				Pearson Type III	d.f.	Erlang	d.f.
Calhoun-Elgin and Dumble—Inbound	1	2:30-3:20	1	60.54	15	65.91	15
			2	15.88[a]	9	19.14	9
Tellepsen and Telephone—Outbound	2	1:15-2:05	1	79.66	11	121.72	11
			2	9.86[a]	5	22.72	6
Griggs and Wayside—Outbound	1	4:30-5:25	2	26.46	7	69.17	7
		5:05-5:10	1	55.48	4	49.20	4
		4:55-5:00	1	12.39	4	12.59	4
		5:05-5:10	2	8.72	1	5.18	1
		4:55-5	2	3.52[a]	2	4.66[a]	2
Scott and Cullen—Outbound	2	4:05-5	2	25.69	4	60.48	4
		4:35-4:40	1	15.43	3	32.68	3
		4:10-4:15	1	14.92	3	12.63	3
		4:35-4:40	2	4.19	1	5.37	1
		4:10-4:15	2	3.48[a]	1	2.70a	1

[a]Significant at the 5 percent level.

location, with a one-second data observation interval. All 4 of these attempts failed to fit a distribution to the data. The attempts were made again using the same data with 2-sec data observation intervals. Two of these time periods, which could be compared, were found to follow the Pearson Type III and the Erlang distributions. The Pearson Type III distribution was used in the analysis of results. The information concerning the time periods of the data and the chi-square test results are given in Table 3.

Discussion of Results

[illegible] served at the Tellepsen-Telephone location (a Case 2 configuration) and the Calhoun-Elgin and Dumble location (a Case 1 configuration) are shown in Figure 13. The total area under each of the curves was equal to one, which is the probability of there being a gap equal to or greater than zero seconds in length. The area under each of the curves to the right of the 3-sec line was the probability of an available, acceptable gap at the on-ramp. The probability of an acceptable gap was 0.46 for the Case 2 configuration and 0.68 for the Case 1 configuration. Since the probability of an acceptable gap was the percent of the gaps which were greater than 3 sec, this probability was an excellent indication of the possible ramp capacities. For this comparison, the ratio was 1.49. Therefore, the Case 1 on-ramp could accommodate approximately 1.49 times the capacity of the Case 2 on-ramp.

The curves for the second comparison are shown in Figure 14. The Pearson Type III distribution was fitted to the data collected at the Griggs-Wayside location (a Case 1 configuration) and the Scott-Cullen location (a Case 2 configuration). The probability of an acceptable gap was 0.51 for the Case 1 configuration and 0.30 for the Case 2 configuration. The ratio of these probabilities was 1.70. Therefore, the Case 1 on-ramp could accommodate approximately 1.70 times the capacity of the Case 2 on-ramp.

Conclusion

In the first comparison the Case 1 on-ramp could accommodate approximately 1.49 times the capacity of the Case 2 on-ramp, and in the second comparison the Case 1 on-ramp could accommodate approximately 1.70 times the capacity of the Case 2 on-ramp. Therefore, it was concluded that the Case 1 configuration (an off-ramp upstream of an on-ramp) offers considerable capacity advantages.

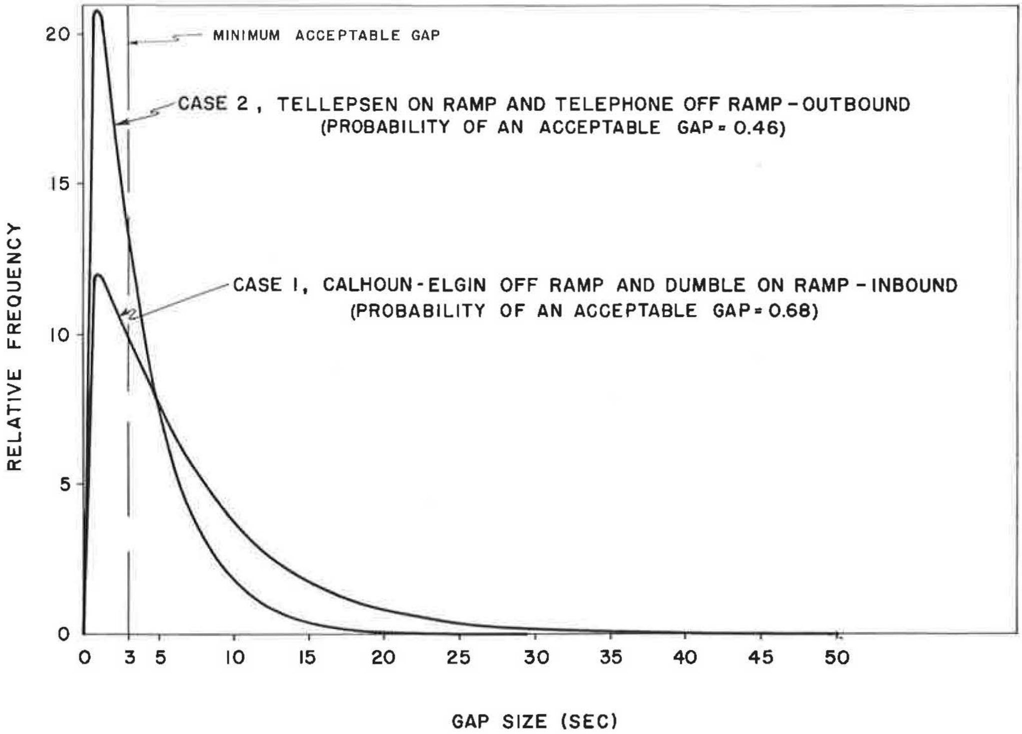


Figure 13. Comparison 1 of acceptable gap probability.

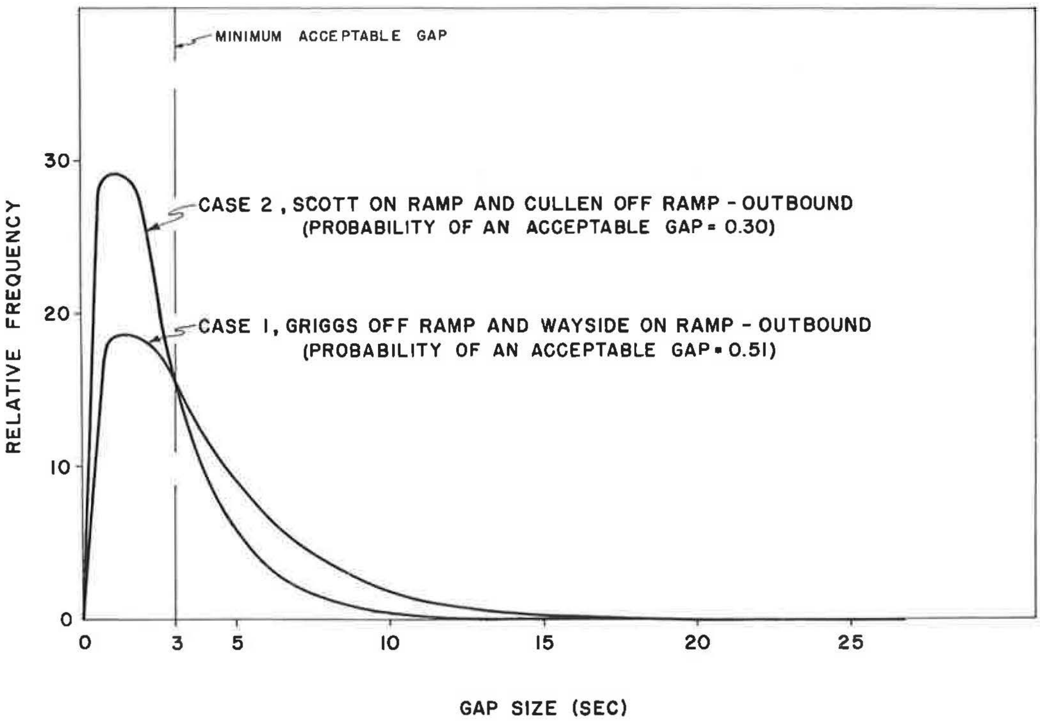


Figure 14. Comparison 2 of acceptable gap probability.

FREEWAY RAMP SPACING

The effect on the amount of acceptable gap time as the distance downstream of an off-ramp increased was investigated in an attempt to develop criteria for ramp spacing. It was concluded earlier that the Case 1 configuration (an off-ramp upstream of an on-ramp) was the most desirable. The critical factor in the desired configuration was the distance between the ramps. The ramps in a Case 1 configuration could not be less than certain distance limitations in order to maintain current design standards (to be discussed later), but no limitation has been set on the maximum spacing which could be used without forfeiting the benefit of the greater capacity (greater acceptable gap time) of the Case 1 configuration.

Method of Study

The study procedure used in this investigation was to determine the probability of acceptable gaps just downstream of an off-ramp and at points located at intervals downstream of the off-ramp (Fig. 15). Theoretical distributions were fitted to the observed data so that the probability of acceptable gaps could be determined. Background information and the reasons for choosing the Pearson Type III and the Erlang distributions were previously discussed. The chi-square test at the 5 percent level of significance was used to test the hypotheses that the theoretical distributions fitted the observed data.

Data Collection

The ramp spacing studies were conducted between the Wayside off-ramp and the Griggs on-ramp—inbound. This location, shown in Figure 15, was called Brays Bayou since the bayou passes through the study section. Both peak and off-peak studies were conducted. The lane 1 gaps were recorded with the 20-pen recorder just downstream of the gore of the Wayside off-ramp and at 5 points located every 500 ft downstream of the gore of the off-ramp. The Esterline-Angus 20-pen recorder was used to record the data as follows:

C, to record the lane 1 freeway gaps and to begin the speed trap.

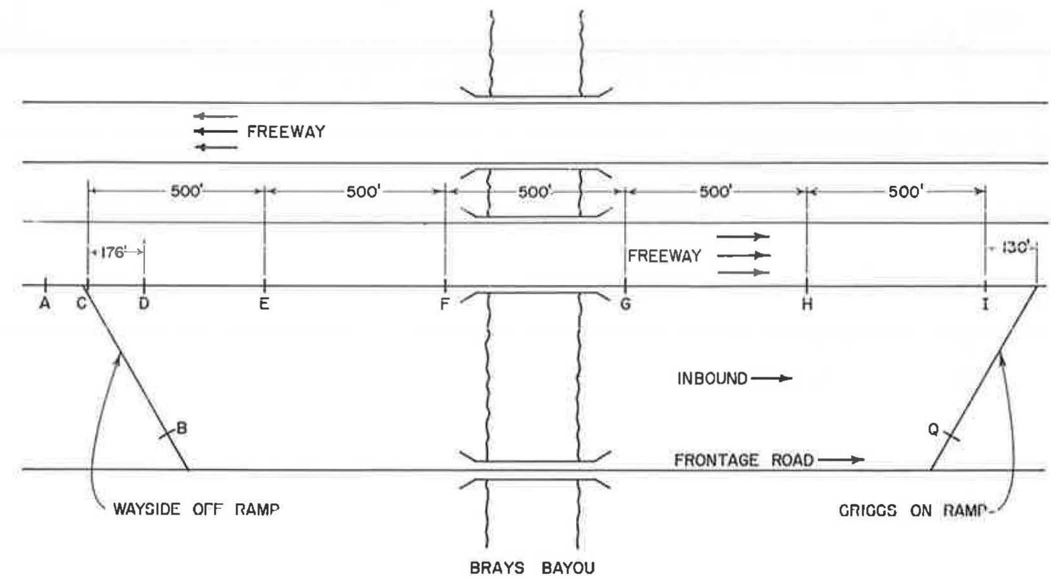


Figure 15. Freeway ramp spacing study location.

2. Pen No. 2 was used at Point D, 176 ft downstream for the end of the speed trap in conjunction with Point C.

3. Pens No. 3 through No. 7 (Points E, F, G, H, and I, respectively) were used at the locations downstream of the off-ramp. Pen No. 3 was used at the Point E 500 ft downstream of the gore of the Wayside off-ramp, and each pen in turn was located an additional 500 ft downstream.

4. Pen No. 15 was used at Point B to record the Wayside off-ramp volume.

5. Pen No. 17 was used at Point Q to record the Griggs on-ramp volume.

6. Pen No. 20 was used at Point A to record the 3-lane freeway volume just upstream of the Wayside off-ramp.

The recorded information was reduced and placed on IBM cards for the data analysis as in the ramp configuration studies. The frequency of the gaps is given in Appendix B.

TABLE 4

BRAYS BAYOU CHI-SQUARE TESTS RESULTS

Date	Point	On-Ramp Closed	Time Period	Data Interval (sec.)	Chi-Square Tests Results			
					Pearson Type III	d.f.	Erlang	d.f.
Jan. 25	C	No	1:30-3 PM	2	12.99[a]	12	46.79	12
	E	No	1:30-3 PM	2	6.08[a]	9	41.91	10
	F	No	1:30-3 PM	2	12.72[a]	9	44.50	8
	G	No	1:30-3 PM	2	10.75[a]	9	42.70	10
	H	No	1:30-3 PM	2	3.75[a]	9	33.00	9
	I	No	1:30-3 PM	2	15.96[a]	10	42.08	11
Feb. 16	C	Yes	7:05-7:10 AM	2	7.05	1	9.58	1
	F	Yes	7:05-7:10 AM	2	9.33	1	8.92	1
	G	Yes	7:05-7:10 AM	2	1.27[a]	1	2.17[a]	1
	H	Yes	7:05-7:10 AM	2	3.25[a]	1	3.60[a]	1
	I	Yes	7:05-7:10 AM	2	7.19	1	10.73	1
	C	Yes	7:05-7:10 AM	1	23.23	3	32.96	3
	F	Yes	7:05-7:10 AM	1	18.28	3	29.19	3
	G	Yes	7:05-7:10 AM	1	35.61	3	34.41	3
	H	Yes	7:05-7:10 AM	1	14.89	3	13.06	3
	I	Yes	7:05-7:10 AM	1	9.78	3	12.37	3
Feb. 18	C	No	7:20-8:00 AM	2	32.04	4	12.65	3
	F	No	7:20-8:00 AM	2	51.67	3	54.80	3
	G	No	7:20-8:00 AM	2	78.12	3	50.48	3
	H	No	7:20-8:00 AM	2	72.84	3	86.54	3
	I	No	7:20-8:00 AM	2	66.32	3	56.60	3
	C	No	7:20-7:25 AM	1	9.72	3	6.82[a]	3
	F	No	7:20-7:25 AM	1	7.69[a]	3	4.45[a]	3
	G	No	7:20-7:25 AM	1	4.67[a]	3	4.51[a]	3
	H	No	7:20-7:25 AM	1	13.47	3	25.29	3
	I	No	7:20-7:25 AM	1	6.03[a]	4	5.91[a]	4
	C	Yes	7:05-7:10 AM	1	7.75[a]	3	9.99	3
	F	Yes	7:05-7:10 AM	1	25.95	3	27.07	3
	G	Yes	7:05-7:10 AM	1	12.14	3	15.21	3
	H	Yes	7:05-7:10 AM	1	17.74	3	14.71	3
	I	Yes	7:05-7:10 AM	1	14.72	3	15.04	3
	C	Yes	7:05-7:10 AM	2	3.54[a]	1	3.84[a]	1
	F	Yes	7:05-7:10 AM	2	1.92[a]	1	4.90	1
	G	Yes	7:05-7:10 AM	2	5.66	1	6.87	1
	H	Yes	7:05-7:10 AM	2	2.16[a]	1	2.83[a]	1
	I	Yes	7:05-7:10 AM	2	3.44[a]	1	3.39[a]	1

[a]Significant at the 5 percent level.

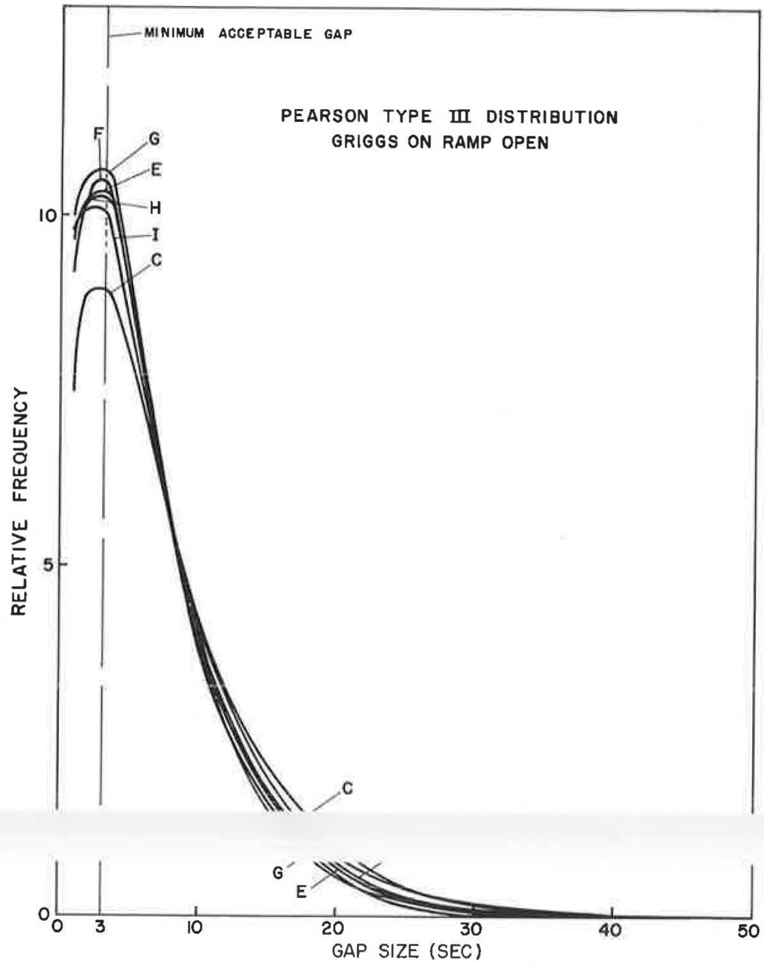


Figure 16. Gap distributions, Brays Bayou, 1:30-3:00 PM, Jan. 25.

Data Analysis

Using a data observation interval of 2 sec, the Pearson Type III distribution was found to fit the data observed from 1:30 to 3:00 p.m. (off-peak data) on January 25. The Erlang distribution, for the same data interval, did not fit these observed data for any point. The time period of the data, the data observation interval, the value of chi-square, the degrees of freedom (d.f.) and the significance of the chi-square tests are given in Table 4. The curves of the Pearson Type III distributions are shown in Figure 16. The area under each of the curves, for gaps of 3 sec and greater, was the probability of an available, acceptable gap at the point each curve represents. These probabilities of acceptable gaps being available are given in Table 5.

An attempt was made to fit a theoretical distribution to the data collected from 7:20 to 8:00 a.m. on February 18, using

TABLE 5

PROBABILITY OF ACCEPTANCE GAPS AT BRAYS BAYOU

Point	1:30-3:00 p.m. Jan. 25, 1965	7:20-7:25 a.m. Feb. 18, 1965	7:05-7:10 a.m. Feb. 18, 1965
C	0.78	0.33	0.33
E	0.74	No data	No data
F	0.77	0.34	0.24
G	0.72	0.34	Not significant
H	0.76	Not significant	0.22
I	0.74	0.43	0.26

a 2-sec data observation interval. Both the Pearson Type III and the Erlang distributions failed to fit the observed data (Table 4). It was decided that the peak-period conditions varied too much during this long time period, and a fit was attempted using the 7:20 to 7:25 a.m. data in one-second data observation intervals. The Erlang distribution fitted these data for 4 of the 5 points, and the Pearson Type III distribution fitted these data for 3 of the 5 points. The Erlang distribution was used since it fitted more data than did the Pearson Type III distribution. Figure 17 shows the curves of the Erlang distribution. The probability of an acceptable gap being available at each point was determined and is given in Table 5.

The results of these data (see Discussion of Results) showed an effect of the Griggs on-ramp (approximately 130 ft downstream of Point I) which made it necessary to study data collected when the on-ramp was closed due to the freeway control study. (As a part of the freeway control study, the Griggs on-ramp was closed for a 15-min period each weekday morning.) A 5-min period of data, collected from 7:05 to 7:10 a.m. when the Griggs on-ramp was closed, was used in one-second data observation intervals in an attempt to fit a distribution to these data. A fit was obtained for only one point; thus, another attempt was made using 2-sec data observation intervals. The

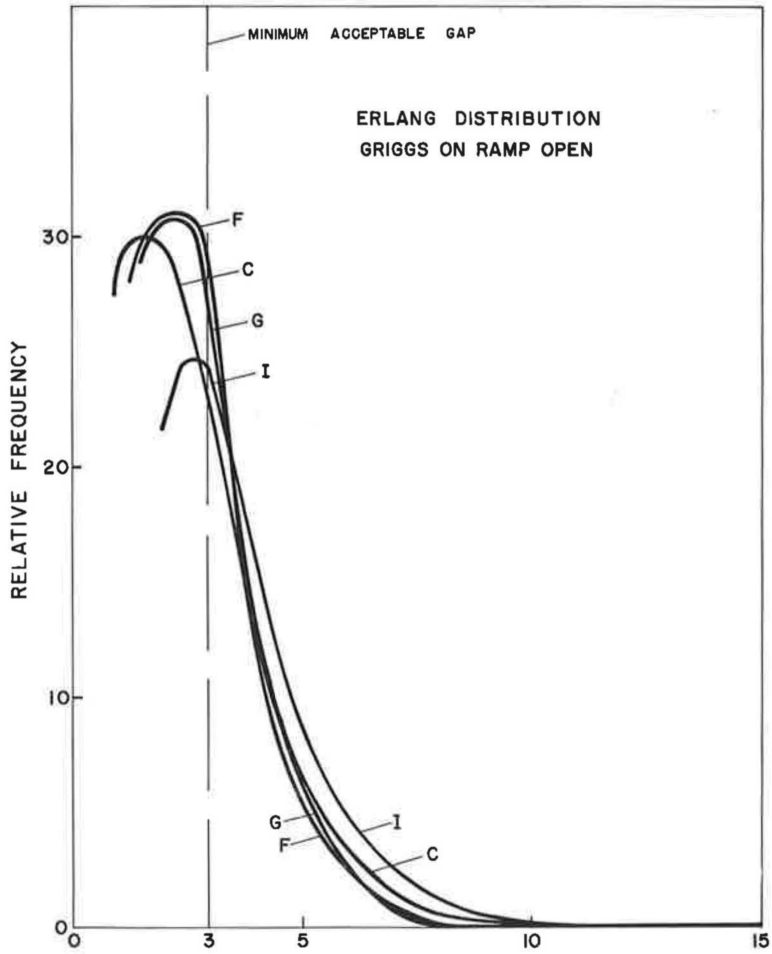


Figure 17. Gap distributions, Brays Bayou, 7:20-7:25 AM, Feb. 18.

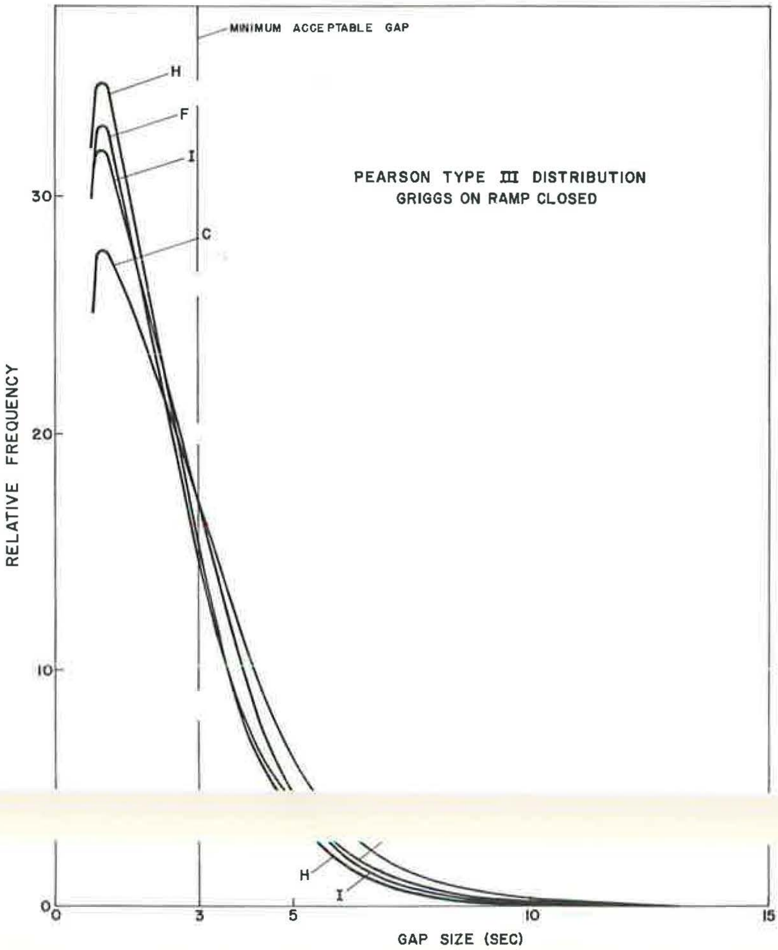


Figure 18. Gap distributions, Brays Bayou, 7:05-7:10 AM, Feb. 18.

Pearson Type III distribution fitted the observed data for 4 of the 5 points, and the Erlang distribution fitted the observed data for 3 of the 5 points. The Pearson Type III distribution was used since it fitted the most data. The curves of the Pearson Type III distribution are shown in Figure 18. The probability of an available, acceptable gap at each point is given in Table 5.

Additional data, collected from 7:05 to 7:10 a.m. on February 16, when the Griggs on-ramp was closed, were analyzed in an attempt to obtain another set of probabilities for peak-period data with the on-ramp closed. These data were used in 2-sec data intervals in an attempt to fit a distribution to the data. The Erlang and the Pearson Type III distribution fitted these data for the same 2 of the 5 points. Since Point C (at the off-ramp) was not one of the locations for which a distribution was fitted to the data, these probabilities could not be used. Thus, one-second data observation intervals were used, and a distribution could not be fitted to any of these data. Hence, none of the data collected on February 16 could be used in the results.

Discussion of Results

The curves of the probabilities of available, acceptable gaps as related to the distance from the gore of the off-ramp are shown in Figure 19. The highest curve represented the probabilities of the data collected from 1:30 to 3:00 p.m. (off-peak data) on

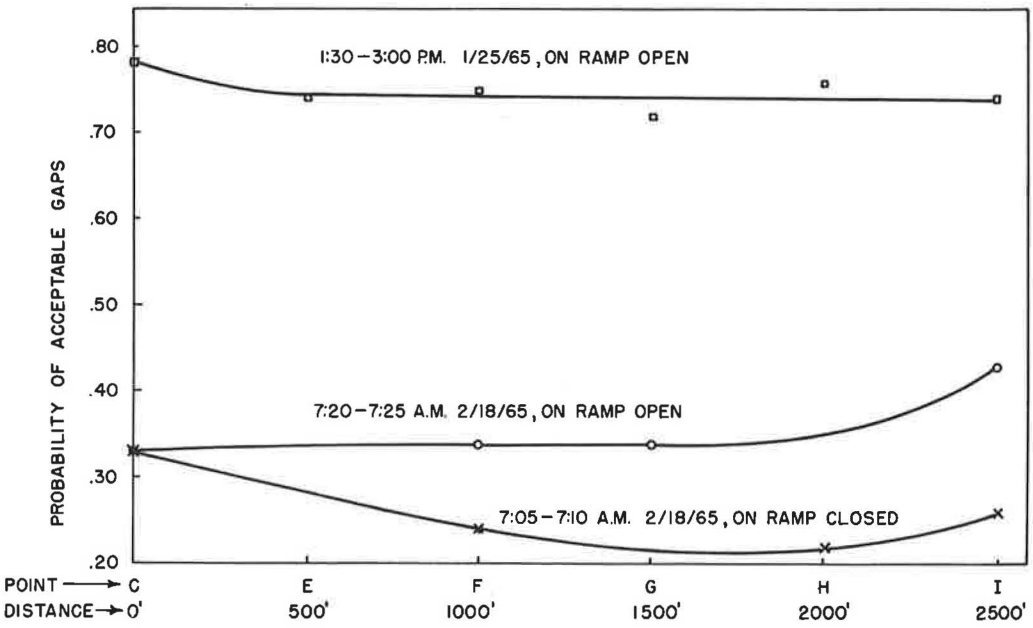


Figure 19. Effect of distance between ramps on acceptable gap probability.

January 25. This curve was essentially a straight line which showed no effect of the distance between the ramps on the probability of available, acceptable gaps.

The center curve represented the probabilities of the data collected from 7:20 to 7:25 a.m. on February 18 when the Griggs on-ramp was open. The curve shows a very slight increase at Points F and G and a marked increase at Point I. This study would usually have been expected to result in a decrease in the probability of available, acceptable gaps as the distance between the ramps increased. But, while the data were being collected, it was noted that vehicles were leaving lane 1 for the center lane as they approached the Griggs on-ramp. This was occurring because the drivers in lane 1 had a very good view of the Griggs on-ramp because it was located at an upgrade, and at 7:20 a.m. they saw vehicles queued on the on-ramp and the frontage road, waiting to enter the freeway. This curve verified the observation that vehicles were leaving lane 1 in the vicinity of the Griggs on-ramp since it shows an increase in the probability of acceptable gaps. Therefore, the decision was made to study data that were collected when the Griggs on-ramp was closed, to eliminate its effect.

The lowest curve represented the probabilities of the data collected from 7:05 to 7:10 a.m. on February 18, when the Griggs on-ramp was closed. This curve showed a decrease in the probability as the distance increased up to Point F as was expected. But, since the probability increases at Point I and possibly at Point H, the remainder of the curve showed that the Griggs on-ramp still had an effect even though it was closed. It was presumed that this effect was caused by repeat drivers who did not realize that the Griggs on-ramp was closed and left lane 1 to avoid the Griggs on-ramp traffic.

Conclusion

The peak period studies of the effect on the amount of acceptable gap time as the distance downstream of an off-ramp increased were inconclusive. No peak-period data were available which could be used to develop criteria for ramp spacing due to the failure to eliminate the effect of the Griggs on-ramp even when it was closed to traffic. It was decided that studies must be conducted at a location where no on-ramp exists for a distance substantially greater than 2600 ft downstream of an off-ramp, in order to obtain data suitable for developing criteria for ramp spacing on this basis.

FEASIBILITY OF STACKED RAMPS

Previously discussed results indicated that at several interchanges both an on-ramp and an off-ramp were desired at the same location (upstream or downstream of the arterial street), and it was concluded that a Case 1 configuration (an off-ramp upstream of an on-ramp) was the most desirable configuration. These results could be satisfied by an off-ramp located upstream of an on-ramp and by stacked ramps (a modification of an off-ramp upstream of an on-ramp with grade-separated ramps). In this section, the results of an investigation of the feasibility of stacked ramps are presented.

Method of Study

Stacked ramps and an off-ramp located upstream of an on-ramp were designed to evaluate their relative costs, the right-of-way required, weaving, and the potential for stage construction.

For the design of the stacked ramps the following factors were assumed:

1. The facility was a 6-lane freeway which had an inside shoulder on each side of the median and an outside shoulder.
2. The centerline of the freeway and the frontage road were at the same elevation.

For the design of the stacked ramps the following criteria were used:

1. The Texas Highway Department recommended designs were used for the ramps (17).
2. The on-ramp horizontal and vertical curves (18) were designed for 40 mph.
3. The off-ramp vertical curves were designed for 35 mph (18).

In this design, one lane of the frontage road was dropped as the freeway on-ramp left the frontage road in order to obtain maximum usage of the available right-of-way. A lane was added to the frontage road as the freeway off-ramp joined the frontage road. In this design the on-ramp crossed over the off-ramp. A 90-ft bridge span was required to cross the off-ramp and provide adequate side clearance. The vertical dis- [illegible] on-ramp grades, and retaining walls were required for the off-ramp depression. This design provided 875 ft between the 2 ramps (from the physical off-ramp gore to the on-ramp nose as in Figure 20). The right-of-way requirement for this design was 360 ft for a minimum distance of 2325 ft along the freeway.

For the normal design of an off-ramp upstream of an on-ramp the following factor was assumed in addition to those assumed for the stacked ramp design: The combined

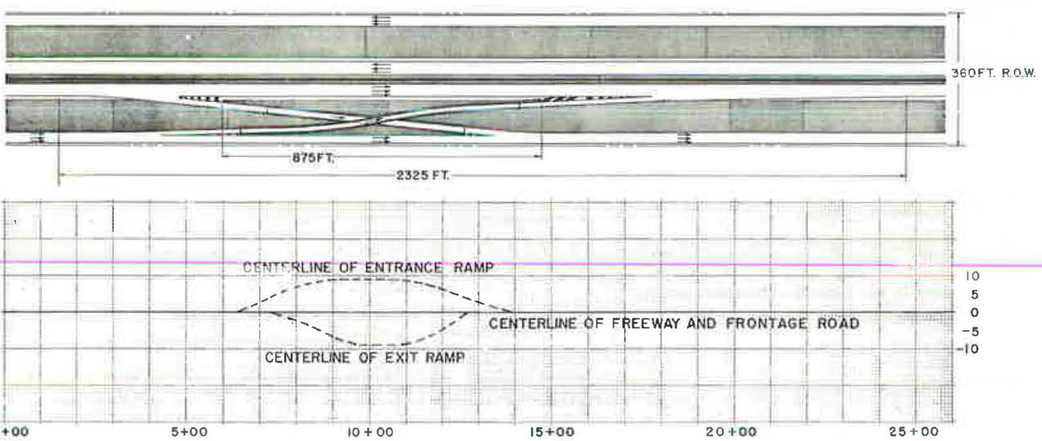


Figure 20. Plan profile of stacked ramps.

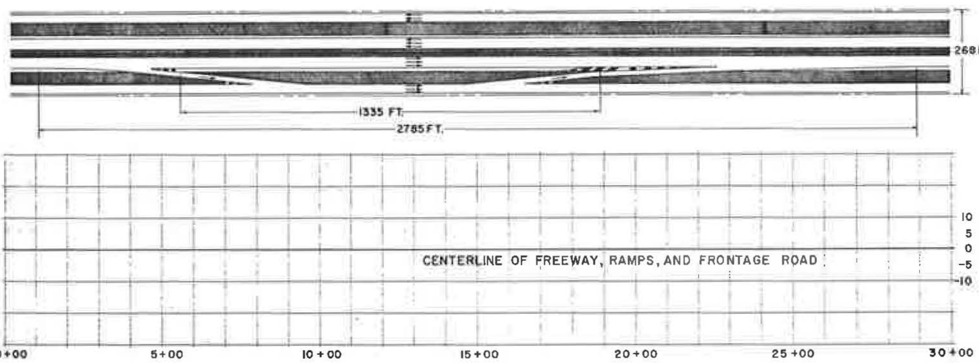


Figure 21. Plan profile at an off-ramp upstream of an on-ramp.

volume of the 2 ramps during the peak hour was 1250 vehicles per hour or 625 vehicles per hour per ramp.

The criteria used for this design were (a) the Texas Highway Department recommended designs were used for the ramps (17), and (b) the weaving distance on the frontage road was designed for volumes of 1250 vehicles per hour to operate at a speed of 35 mph (17). In this design a 50-ft outer separation was adequate to provide a 350-ft deceleration lane. A weaving distance of 500 ft was provided on the frontage road between the 2 ramps to accommodate 1250 weaving vehicles per hour at an operating speed of 35 mph. The plan profile of this design is shown in Figure 21. This design provided 1335 ft between the 2 ramps. The right-of-way requirement for this design was 268 ft for a distance of 2785 ft along the freeway.

Discussion of Results

The results of the designs indicated that the stacked ramp design required 360 ft of right-of-way and a distance of 2325 ft along the freeway, and the off-ramp located upstream of an on-ramp design required 268 ft of right-of-way and a distance of 2785 ft along the freeway. These respective designs are shown in Figures 20 and 21. The stacked ramp design required 460 ft less along the freeway than does the alternate design.

The estimated cost of the stacked ramp design would have been many times greater than the cost of the alternate design due to the additional right-of-way required, the bridge required to raise and lower the on-ramp, the 90-ft span to cross over the off-ramp, and the retaining walls required in the off-ramp depression.

Weaving would be completely eliminated from the frontage road in the stacked ramp design since the vehicles cross paths at a grade separation. The off-ramp located upstream of an on-ramp configuration could create weaving problems on the frontage road. This weaving could be accommodated by an adequately designed weaving distance without too much distance being required, due to the relatively low operating speed on the frontage road. A weaving volume of 1250 vehicles per hour can be accommodated at an operating speed of 35 mph in a distance of 500 ft (17).

The off-ramp located upstream of an on-ramp configuration had the potential for stage construction because adding the second ramp would not physically affect the first ramp constructed. Stage construction would be considered in the original design so that the first ramp would be located so as to furnish the distance along the freeway required by the addition of another ramp. The stacked ramp configuration did not have great potential for stage construction because the existing ramp would have to be reconstructed to cross the ramp to be added, additional right-of-way would be required, and the frontage road would have to be moved to increase the width of the outer separation.

Conclusion

The high cost, the lack of potential for stage construction, and the additional right-of-way required, indicate that the construction of stacked ramps may not be generally feasible to gain the advantages of no weaving on the frontage road and less distance (460 ft) required along the freeway to fit in the design. The stacked ramp arrangement could be expected to provide a high level of service, however, and in many cases might warrant consideration.

INTERCHANGE LAYOUTS

The suitability of various interchange layouts in fulfilling drivers' desires, providing access to freeway and arterial street traffic, and reducing the interference to freeway and arterial street traffic was investigated to determine the merits of 2 proposed types of interchange layouts. Each of the types of interchange layouts investigated was formed on the basis of the results discussed earlier in this report.

Method of Study

The types of interchange layouts considered are shown in Figure 22. The ramps in the layouts were shown as dashed lines to indicate the location of the ramps if they were desired. One of the previous conclusions stated that the desired movement of traffic could be fulfilled by providing ramps based on these desires. Therefore, each of the interchange layouts which was investigated had the potential to fulfill drivers' desires. A Case 1 configuration (an off-ramp located upstream of an on-ramp) which was concluded to be the most desirable ramp configuration was used twice in the Type 1 layout and once in the Type 2 layout. The Type 1 layout had a Case 1 configuration upstream and downstream of the arterial street, and the Type 2 layout had a Case 1 configuration spanning the arterial street. The Case 1 configurations in each layout were an off-ramp located upstream of an on-ramp since it was concluded that the use of stacked ramps may not be feasible in all cases.

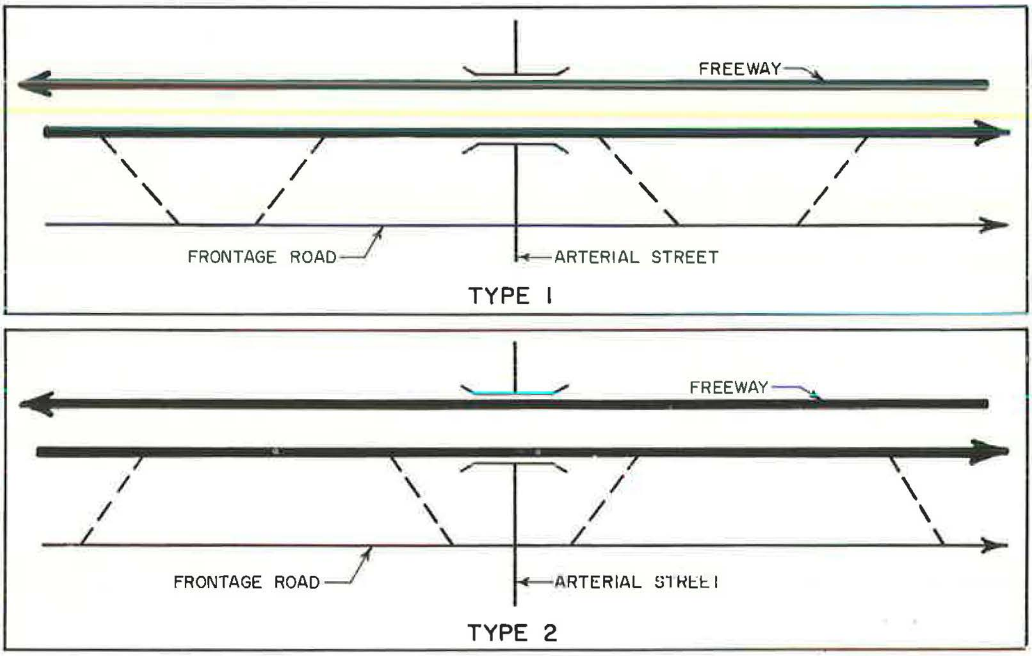


Figure 22. Types of interchange layouts.

The types of interchange layouts were compared using the following considerations: (a) potential for stage construction, (b) fulfillment of drivers' desires, (c) critical distance (off-ramp to arterial street), (d) maximum access to abutting property, (e) maximum access to the freeway, (f) freeway with reduced capacity at interchange, (g) freeway without reduced capacity at interchange, (h) minimum interference to the arterial street, (i) weaving on the freeway, and (j) interstate signing standards.

Discussion of Results

In Figure 22 the ramps are shown as dashed lines to indicate the location of the respective ramps if they were desired. All of the ramps should be included in the original design, but only the desired ramps would be built in the original construction. Therefore, if a ramp were not desired at the time the interchange was constructed, adequate space would be provided in the interchange layout for the stage construction of the other ramps which might be desired at some future date. Each of the types of interchange layouts provides for the potential of stage construction of ramps.

In the Type 2 interchange layout, a critical distance between the terminal of the off-ramp located upstream of the arterial street and the arterial street was introduced. This distance needed to be sufficient to provide an adequate storage space for vehicles stopped for the signal in addition to an adequate weaving distance in which the off-ramp traffic could weave across the frontage road to make a right turn at a signal. This distance was dependent on the frontage road volume, the signalized intersection capacity for this approach, the number of frontage road lanes, and the number of off-ramp vehicles desiring to make a right turn.

Maximum access to abutting property was provided by locating an off-ramp just downstream of an arterial street. And, an on-ramp located just upstream of an arterial street maximized direct access to the freeway, from abutting property, and minimized the volume of traffic required to cross straight through the intersection to

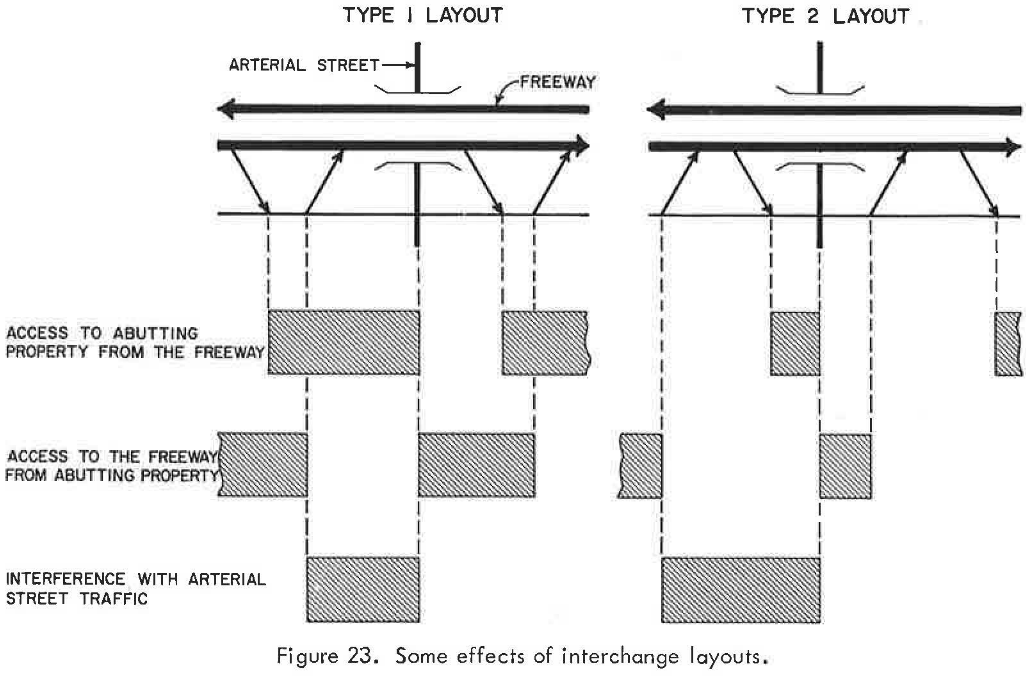


Figure 23. Some effects of interchange layouts.

TABLE 6

COMPARISON OF TYPES OF INTERCHANGE LAYOUTS

Factors	Type of Interchange Layout	
	Type 1	Type 2
Potential for stage construction	X	X
Fulfill drivers' desires	X	X
Critical distance (off-ramp to arterial street)		X
Maximum access to abutting property	X	
Maximum access to the freeway	X	
Freeway with reduced capacity at interchange		X
Freeway without reduced capacity at interchange	X	X
Minimum interference to the arterial street	X	
Weaving on the freeway	X	X
Meets Interstate signing standards	X	X

gain access to the freeway. The Type 1 interchange layout provided for ramps to be located in this manner, and therefore, it furnished the maximum access to both the freeway and abutting property (Fig. 23).

Minimum interference to the arterial street traffic was provided by locating an on-ramp just upstream of the arterial street. This ramp reduced the volume on the frontage road approach to the signalized intersection by the number of vehicles that desire to enter the freeway. Thus, a minimum effect was felt by the arterial street, and a greater portion of the "green time" at the signalized intersection could be used [illegible] terial street (Fig. 23).

Minimum interference to the freeway was determined by the design as the freeway and the arterial street crossed. If the design was such that the capacity of the freeway was reduced (for example, by introducing a sharp increase in freeway grade) as it crossed the arterial street, the Type 2 interchange layout should be used. In this instance the freeway volume would have been reduced as the capacity of the freeway was reduced. If the capacity of the freeway was not reduced by the design, either type of interchange layout could be used with minimum interference to the freeway.

Freeway signing, following Interstate Highway standards, could be used for either type of interchange layout, since the distance between interchange layouts approached one mile as a minimum.

Conclusions

Considering the foregoing factors, the Type 1 interchange layout was the better layout with one exception. This exception was that the Type 2 interchange layout would be required when the capacity of the freeway was reduced as the freeway crossed the arterial street. A comparison of the types of interchange layouts as related to the factors discussed is given in Table 6.

FREEWAY LAYOUTS

Interchange Spacing

The minimum spacing of interchanges was investigated since the freeway designer is usually faced with the task of designing a new facility which can service existing arterial streets that are often closely spaced. The 2 types of interchange layouts discussed in the previous section were considered to investigate the interchange spacing that would result from their use.

Method of Study

Two types of freeway layouts were determined: (a) Type I, which resulted from combining two of the Type I interchange layouts (Fig. 22) closely together, and (b) Type II, which resulted from combining 2 of the Type 2 interchange layouts (Fig. 22) closely together to form a section of freeway.

To develop the Type I freeway layout using a pair of Type 1 interchange layouts, the following were assumed: (a) all ramp volumes were 625 vehicles per hour, (b) the freeway volume between an on-ramp and an off-ramp was 5400 vehicles per hour, and (c) the freeway did not have a reduction in capacity as it crossed the arterial street.

The design of an off-ramp upstream of an on-ramp as described earlier was used in the freeway layout.

Moskowitz and Newman's procedure (6) was used to determine the distance required between the physical nose of an on-ramp and the physical gore of a downstream off-ramp. This calculation is given in Appendix C.

A special case of the Type I freeway layout was determined by overlapping the 2 pairs of ramps between the arterial streets in the Type I freeway layout. Thus, in the special case of the Type I freeway layout there were only 2 ramps between the arterial street.

The Type II freeway layout using a pair of Type 2 interchange layouts was made assuming the same values as were assumed for the Type I freeway layout. This freeway layout used the same ramp designs as the off-ramp upstream of an on-ramp, but the spacing on the frontage road between the ramps was different due to the signalized intersection within this area. A special case of the Type II interchange layout was determined by overlapping the 2 pairs of ramps in the Type II freeway layout.

Discussion of Results

The Type I freeway layout and its special case (overlapping the 2 pairs of ramps between the arterial streets) are shown in Figure 24. The minimum interchange spacing resulting from combining two Type 1 interchange layouts was 5670 ft, or just over one mile.

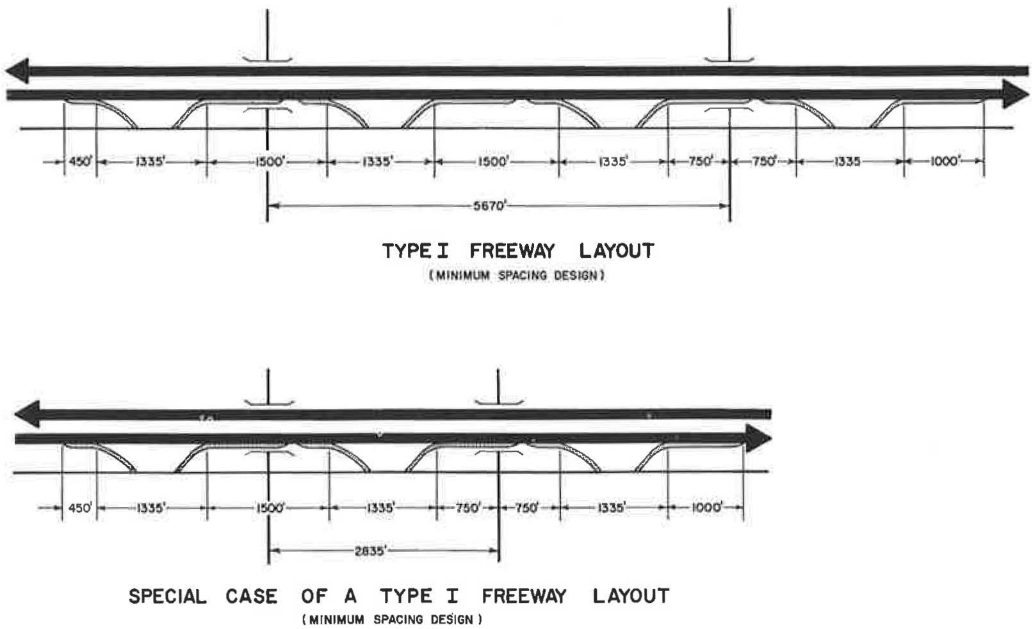


Figure 24. Type I freeway layouts.

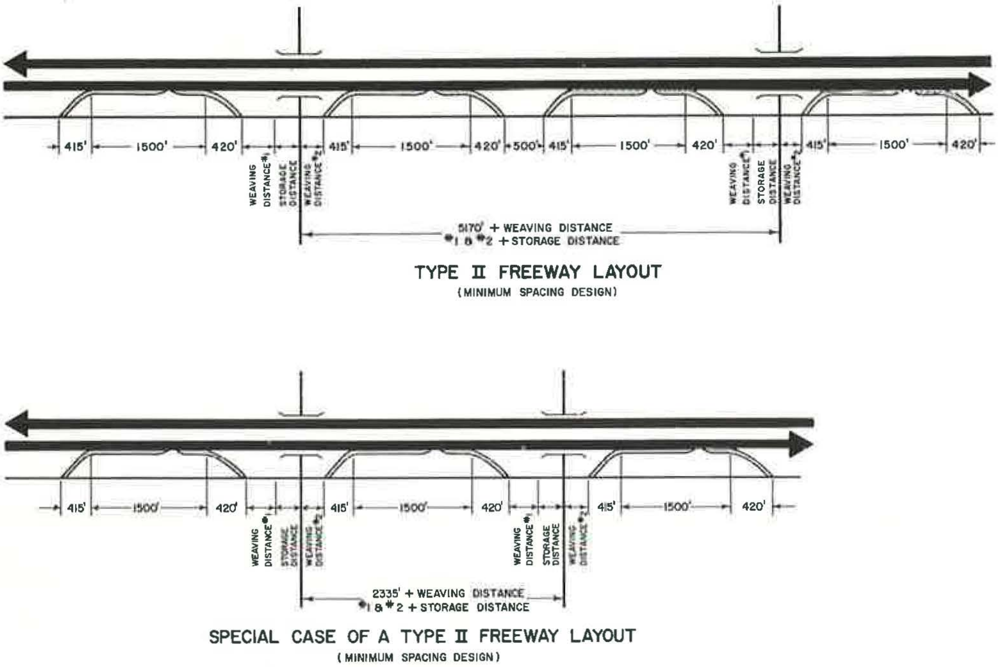


Figure 25. Type II freeway layouts.

The minimum interchange spacing for the special case of the Type I freeway layout was one-half of the previous distance, 2835 ft, or just over 0.5 mile. One disadvantage

[illegible] mum interchange spacing resulting from combining 2 of the Type 2 interchange layouts was 5170 ft plus 2 different weaving distances and a vehicle storage distance. The weaving distance No. 1 is dependent on the off-ramp traffic which desires to turn right at the signal, and the frontage road volume. The storage distance is dependent on the frontage road volume, the "green time" for the frontage approach, and the number of approach lanes on the frontage road. The weaving distance No. 2 is dependent on the number of drivers desiring to enter the freeway who made right turns onto the frontage road, and the existence of a free right turn which might enter the frontage road at a point some distance downstream of the intersection.

The minimum spacing of the special case of the Type II freeway layout would be somewhat greater than one-half of the Type II freeway layout minimum interchange spacing. This occurred because it was certain that the sum of the 3 unknown distances would be greater than the 500 ft between the 2 ramps in the center of the Type II freeway layout. Signing problems may also occur for this short interchange spacing.

Conclusion

Minimum interchange spacing was provided by the Type I freeway layout which consisted of the combination of 2 interchange layouts with an off-ramp located upstream of an on-ramp both before and after an arterial street (Type 1 interchange layout). This gives additional emphasis to the durability of the Type 1 interchange layout.

CONCLUSIONS

1. Standard interchange designs cannot always fulfill the various desired movements at different interchanges. To obtain the most efficient operation at a specific interchange, it may be desirable to use a diamond type, an X-type, or possibly a combina-

tion of both of these. Considerable effort should be made to predict the desired movements at any given interchange and to design the ramp arrangements accordingly.

2. The configuration of a off-ramp located upstream of an on-ramp has considerable advantages over the reverse configuration. The studies indicated that an approximate 50 to 70 percent increase in on-ramp capacity could be obtained by removing traffic in advance of adding traffic to the freeway.

3. The construction of stacked ramps rather than an off-ramp upstream of an on-ramp was not generally feasible due to the high probable cost, the lack of potential for stage construction and the additional right-of-way required. The stacked ramps, however, offer the advantages of elimination of weaving on the frontage road and less distance (approximately 460 ft) required along the freeway to fit in the design. The desirability of the stacked ramp use would have to be evaluated in each specific case considering the topography, the need for this type ramp as indicated by traffic volumes and other individual factors.

4. With one exception, the type of interchange layout which has an off-ramp located upstream of an on-ramp both upstream and downstream of the arterial street is the most desirable. The exception would exist when the freeway capacity is reduced by the design as the freeway crosses the arterial street. On the basis of this study, it appears that considerable attention should be given to the use of an X-type interchange which would provide the desired interchange layout.

5. Minimum interchange spacing was provided by the combination of 2 interchange layouts with an off-ramp located upstream of an on-ramp both before and after an arterial street (Type I interchange layout). This gives additional emphasis to the desirability of the Type I interchange layout.

REFERENCES

1. Keese, C. J., Pinnell, C., and McCasland, W. R. A Study of Freeway Traffic Operation. HRB Bull. 235, pp. 73-132, 1960.
2. Pinnell, C., and Keese, C. J. Freeway Ramps. Report by Texas Transportation Institute, 1960.
3. Fukutome, I., and Moskowitz, K. Traffic Behavior and On-Ramp Design. HRB Bull. 235, pp. 35-72, 1960.
4. Hess, Joseph W. Capacities and Characteristics of Ramp-Freeway Connectors. Highway Research Record 27, pp. 69-115, 1963.
5. Fisher, R. L. Accident and Operating Experience at Interchanges. HRB Bull. 291, pp. 124-138, 1961.
6. Moskowitz, K., and Newman, L. Notes on Freeway Capacity. Highway Research Record 27, pp. 44-68, 1963.
7. Conklin, Robert D. A Comparison of Vehicle Operating Characteristics Between Parallel Lane and Direct Taper Types of Freeway Off Ramps. Proc. Inst. Traffic Engineers, pp. 19-20, 1959.
8. Fukutome, I., and Moskowitz, K. Traffic Behavior and Off-Ramp Design. Highway Research Record 21, pp. 17-31, 1963.
9. Jouzy, N. C., and Michael, H. L. Use and Design of Acceleration and Deceleration Lanes in Indiana. Highway Research Record 9, pp. 25-51, 1963.
10. Lipscomb, John. The Effect of Ramps on the Lateral Distribution of Vehicles on a Six Lane Freeway. Thesis, Texas A and M Univ., 1962.
11. Gerlough, D. L. Traffic Inputs for Simulation on a Digital Computer. HRB Proc., Vol. 38, pp. 480-492, 1959.
12. Schuhl, Andre. The Probability Theory Applied to the Distribution of Vehicles on Two-Lane Highways. In Poisson and Traffic, The Eno Foundation, 1955, pp. 59-72.
13. Haight, Frank A. Mathematical Theories of Traffic Flow. Academic Press, New York, 1963.
14. May, Adolf D. Gap Availability Studies. Highway Research Record 72, pp. 101-136, 1965.

15. Drew, Donald R. A Study of Freeway Traffic Congestion. PhD dissertation, Texas A and M Univ., May 1964.
16. Gerlough, D. L. Use of Poisson Distribution in Highway Traffic. In Poisson and Traffic, The Eno Foundation, 1955, p. 17.
17. Design Manual for Controlled Access Highways. Texas Highway Department, 1960, revised Sept. 1963.
18. A Policy on Geometric Design of Rural Highways. AASHO, pp. 395-398, 1964.

Appendix A

TABLE 7

FREQUENCY OF GAPS—FREEWAY RAMP CONFIGURATION STUDIES

Gap Size (sec.)	Tellepsen On-Ramp 1:15-2:05 p.m.	Dumble On-Ramp 2:30-3:20 p.m.	Scott On-Ramp 4:10-4:15 p.m.	Wayside On-Ramp 4:55-5:00 p.m.
0-2	300	99	61	29
2-4	243	113	33	21
4-6	125	58	8	15
6-8	66	42	7	6
8-10	32	37	3	6
10-12	16	26	0	2
12-14	9	26	0	2
14-16	5	15	0	0
20-22	0	6	0	0
22-24	0	3	0	0
24-26	2	0	0	0
26-28	0	2	0	0
28-30	0	4	0	0
30-32	1	0	0	0
32-34	0	0	0	0
34-36	0	2	0	0
36-38	0	0	0	0
38-40	0	0	0	0
40-42	0	0	0	0
42-44	0	0	0	0
44-46	0	0	0	0
46-48	0	1	0	0
48-50	0	0	0	0

Appendix B

TABLE 8

FREQUENCY OF GAPS—FREEWAY RAMP SPACING STUDIES
(Brays Bayou, Jan. 25, 1:30-3:00 p.m., On-Ramp Open)

Gap Size (sec.)	C	E	F	G	H	I
0-2	91	133	132	144	133	126
2-4	122	148	171	175	161	170
4-6	91	121	126	130	128	122
6-8	88	92	81	89	87	81
8-10	55	75	71	63	73	69
10-12	57	53	51	56	54	49
12-14	28	44	46	43	47	41
14-16	35	24	34	22	27	26
16-18	21	16	19	25	17	18
18-20	10	17	13	12	16	9
20-22	11	9	7	6	8	6
22-24	11	4	4	5	6	12
24-26	11	3	3	4	3	5
26-28	4	0	0	1	5	2
28-30	2	4	2	1	1	2
30-32	0	0	1	1	1	2
32-34	2	1	1	1	2	1
34-36	1	0	1	1	0	1
36-38	2	0	0	0	0	1
38-40	0	0	0	0	0	0
40-42	0	0	0	0	0	0
42-44	0	0	0	0	0	0
44-46	0	0	0	0	0	0
46-48	0	1	0	0	0	0
48-50	0	0	0	0	0	0

TABLE 9

FREQUENCY OF GAPS—FREEWAY RAMP SPACING STUDIES
(Brays Bayou, Feb. 16, 7:05-7:10 a.m., On-Ramp Closed)

Gap Size (sec.)	C	F	G	H	I
0-1	4	10	8	6	4
1-2	22	56	62	51	50
2-3	19	38	28	30	36
3-4	10	13	10	12	18
4-5	11	4	7	8	5
5-6	2	2	4	7	4
6-7	2	2	3	1	1
7-8	1	0	1	1	1
8-9	0	1	0	0	1
9-10	2	2	0	1	0
10-11	0	0	0	0	0
11-12	1	0	0	0	0
12-13	0	0	0	0	0
13-14	1	0	1	0	0
14-15	0	0	0	0	0
15-16	0	0	0	0	0
16-17	1	0	0	0	0
17-18	0	0	0	0	0
18-19	1	0	0	0	0
19-20	0	0	0	0	0
20-21	0	0	0	0	0
21-22	0	0	0	0	0
22-23	0	0	0	0	0
23-24	0	0	0	0	0
24-25	0	0	0	0	0

TABLE 10

FREQUENCY OF GAPS—FREEWAY RAMP SPACING STUDIES
(Brays Bayou, Feb. 16, 7:05-7:10 a.m., On-Ramp Closed)

Gap Size (sec.)	C	F	G	H	I
0-2	26	66	70	57	54
2-4	29	51	38	42	54
4-6	13	6	11	15	9
6-8	3	2	4	2	2
8-10	2	3	0	1	1
10-12	1	0	0	0	0
12-14	1	0	1	0	0
14-16	0	0	0	0	0
16-18	1	0	0	0	0
18-20	1	0	0	0	0
20-22	0	0	0	0	0
22-24	0	0	0	0	0
24-26	0	0	0	0	0
26-28	0	0	0	0	0
28-30	0	0	0	0	0
30-32	0	0	0	0	0
32-34	0	0	0	0	0
34-36	0	0	0	0	0
36-38	0	0	0	0	0
38-40	0	0	0	0	0
40-42	0	0	0	0	0
42-44	0	0	0	0	0
44-46	0	0	0	0	0
46-48	0	0	0	0	0
48-50	0	0	0	0	0

TABLE 11

FREQUENCY OF GAPS—FREEWAY RAMP SPACING STUDIES
(Brays Bayou, Feb. 18, 7:05-7:10 a.m., On-Ramp Closed)

Gap Size (sec.)	C	F	G	H	I
0-1	12	17	7	12	7
1-2	39	58	61	61	58
2-3	27	29	35	20	26
3-4	10	8	13	16	15
4-5	7	6	6	8	9
5-6	4	4	6	4	6
6-7	5	3	0	1	3
7-8	4	2	1	2	0
8-9	0	1	0	0	0
9-10	0	0	0	0	0
10-11	0	0	0	0	0
11-12	0	0	0	0	0
12-13	0	0	0	0	0
13-14	0	0	0	0	0
14-15	0	0	0	0	0
15-16	0	0	0	0	0
16-17	0	0	0	0	0
17-18	0	0	0	0	0
18-19	0	0	0	0	0
19-20	0	0	0	0	0
20-21	0	0	0	0	0
21-22	0	0	0	0	0
22-23	0	0	0	0	0
23-24	0	0	0	0	0
24-25	0	0	0	0	0

TABLE 12

FREQUENCY OF GAPS—FREEWAY RAMP SPACING STUDIES
(Brays Bayou, Feb. 18, 7:05-7:10 a.m., On-Ramp Closed)

Gap Size (sec.)	C	F	G	H	I
0-2	51	75	68	73	65
2-4	37	37	48	36	41
4-6	11	10	12	12	15
6-8	9	5	1	3	3
8-10	0	1	0	0	0
10-12	0	0	0	0	0
12-14	0	0	0	0	0
14-16	0	0	0	0	0
16-18	0	0	0	0	0
18-20	0	0	0	0	0
20-22	0	0	0	0	0
22-24	0	0	0	0	0
24-26	0	0	0	0	0
26-28	0	0	0	0	0
28-30	0	0	0	0	0
30-32	0	0	0	0	0
32-34	0	0	0	0	0
34-36	0	0	0	0	0
36-38	0	0	0	0	0
38-40	0	0	0	0	0
40-42	0	0	0	0	0
42-44	0	0	0	0	0
44-46	0	0	0	0	0
46-48	0	0	0	0	0
48-50	0	0	0	0	0

TABLE 13

FREQUENCY OF GAPS—FREEWAY RAMP SPACING STUDIES
(Brays Bayou, Feb. 18, 7:20-7:25 a.m., On-Ramp Open)

Gap Size (sec.)	C	F	G	H	I
0-1	8	3	5	3	2
1-2	44	36	30	33	24
2-3	22	29	27	30	29
3-4	19	15	15	12	15
4-5	9	12	14	3	9
5-6	7	1	2	3	8
6-7	0	2	0	0	2
7-8	1	1	1	1	2
8-9	1	1	1	0	1
9-10	1	0	0	1	0
10-11	0	0	0	1	0
11-12	0	0	0	0	1
12-13	0	0	0	0	0
13-14	0	0	0	0	0
14-15	0	0	0	0	0
15-16	0	0	0	0	0
16-17	0	0	0	0	0
17-18	0	0	0	0	0
18-19	0	0	0	0	0
19-20	0	0	0	0	0
20-21	0	0	0	0	0
21-22	0	0	0	0	0
22-23	0	0	0	0	0
23-24	0	0	0	0	0
24-25	0	0	0	0	0

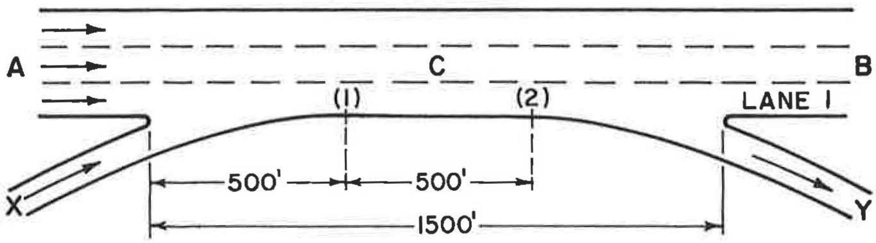
TABLE 14

FREQUENCY OF GAPS—FREEWAY RAMP SPACING STUDIES
(Brays Bayou, Feb. 17, 7:20-8:00 a.m., On-Ramp Open)

Gap Size (sec.)	C	F	G	H	I
0-2	251	280	254	216	138
2-4	276	378	375	364	401
4-6	112	102	104	104	125
6-8	50	45	24	25	35
8-10	18	11	13	15	10
10-12	5	3	7	10	4
12-14	6	1	3	2	0
14-16	4	1	0	0	1
16-18	2	1	1	1	1
18-20	0	1	1	0	0
20-22	1	0	0	0	1
22-24	0	0	0	0	0
34-36	0	0	0	0	0
26-28	0	0	0	0	0
28-30	0	0	0	0	0
30-32	0	0	0	0	0
32-34	0	0	0	0	0
34-36	0	0	0	0	0
36-38	0	0	0	0	0
38-40	0	0	0	0	0
40-42	0	0	0	0	0
42-44	0	0	0	0	0
44-46	0	0	0	0	0
46-48	0	0	0	0	0
48-50	0	0	0	0	0

Appendix C

CALCULATION OF THE MINIMUM DISTANCE BETWEEN AN ON-RAMP AND AN OFF-RAMP



C = 5400
A to B = 4150
X to Y = 0
X to B = 625
Y to B = 625

Find lane volumes

a. Average lane volume = 5400 ÷ 3 = 1800

b. Check lane 1 volume at (1)

1. Thru traffic in right lane = 14% = .14 (4150)	=	580
2. On-ramp traffic in right lane = 1.00 (625)	=	625
3. Off-ramp traffic in right lane = .94 (625)	=	587
Total in right lane at (1)	=	1792

c. Check lane 1 volume at (2)

1. Thru traffic in right lane	=	580
2. On-ramp traffic in right lane (.60 x 625)	=	375
3. Off-ramp traffic in right lane (1.00 x 625)	=	625
Total in right lane at (2)	=	1580

Since the right lane volumes at both (1) and (2) are less than 1800 vehicles per hour, this design is satisfactory to accommodate the assumed volumes.

Research and Development of a Guide Rail System For a High-Speed Elevated Expressway

GILLES G. HÉNAULT, Director of Technical Services, The Warnock Hersey Company Ltd., Montreal, Quebec, and
HENRI PERRON, Director of Traffic Engineering Bureau, Highway Department, Province of Quebec

This paper covers the analysis of all accidents on an elevated expressway in Montreal and recommendations. It also covers procedure, testing, results, analysis, recommendations and an analysis of accidents after the construction of the new guide rail.

•SAFETY is one of the most important factors in the design of a modern highway, and all possible effort should be made to protect both the user and the vehicle. In order to determine the main causes of accidents, the Province of Quebec decided to study the high accident rate on a 7-mi long, 6-lane elevated expressway.

The preliminary study started in April 1962, and was conducted during the next 3 months. Many causes for the high rate of accidents were found, but mainly it was due to poor performance of the guide rail system. Therefore, it was decided that the most urgent study to be undertaken would be the design of a new guide rail system to meet the actual traffic conditions.

[illegible] and results were ob- [illegible] rail system was done between July and November 1963.

This paper gives a brief discussion of the main causes of accidents and the details of the study performed on various types of guide rails. Also, it gives the accident rate before and after modifications were made.

PRELIMINARY STUDY

The preliminary study was made primarily from the accident records together with some field measurements and other data obtained from the Highway Department.

Traffic Data

The Montreal Metropolitan Boulevard (now part of the Trans-Canada Highway) is located in the northern part of the city and is 6.5 mi long (Fig. 1). It is an urban 6-lane elevated expressway with six 12-ft lanes and many exits and accesses, and it was completed in January 1961, although one 3-mi portion was opened in September 1959.

The traffic density has greatly increased since its opening (Fig. 2). These data are representative of summer days and the AADT is 80 percent of them. Truck traffic is 17 percent of the AADT. The weekly traffic variation is very small, except for Sunday when there is a reduction of 35 percent, and Saturday, 25 percent.

The reduction in traffic density during 1963 is the result of the reconstruction of the guide rail system, during which one lane was closed to traffic. The directional traffic density per hour for summer days in 1965 is shown in Figure 3.

Paper sponsored by Committee on Guardrails and Guide Posts and presented at the 45th Annual Meeting.

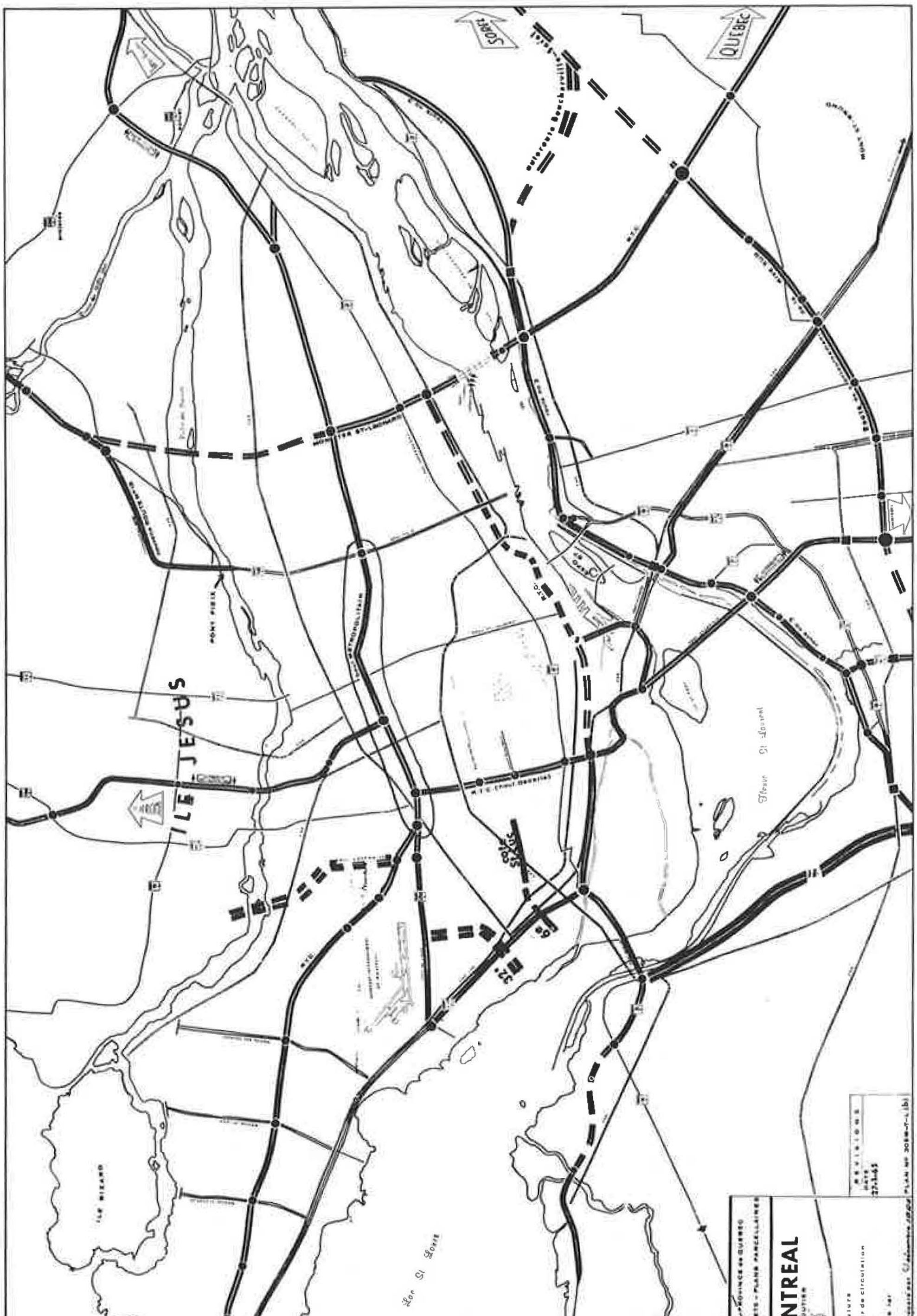


Figure 1. Metropolitan Boulevard.

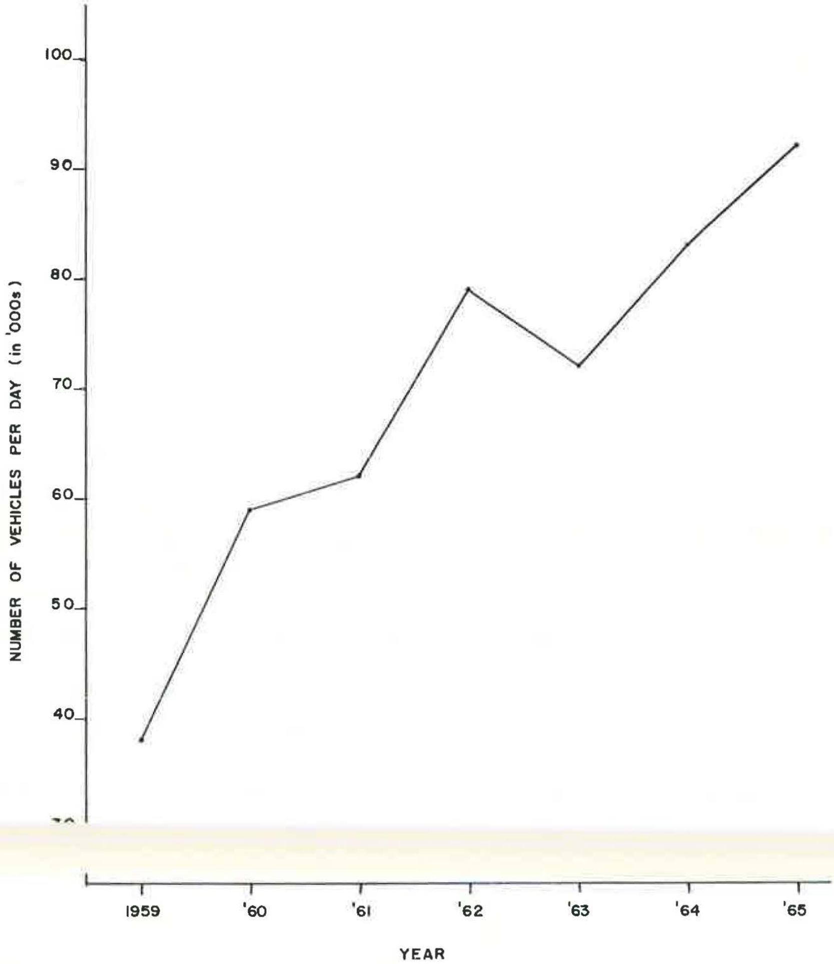


Figure 2. Average daily traffic (summer day).

Accident Analysis

A summary of the accident records is given in Tables 1 and 2. An analysis of all the accident records shows a high number of accidents involving a vehicle hitting another vehicle, the parapet or the lighting poles. Also, it indicated that more than 45 vehicles had fallen off the elevated highway between its opening and this study.

Since the main aim of this paper is to discuss the guide rail system, we will concentrate on the analysis of accidents involving the safety barrier as it then existed. These accidents occurred under such a wide variety of conditions that no definite conclusions could be reached as to their exact cause. However, a common factor throughout seems to have been the carelessness of drivers. It was also noted that there was a concentration of accidents along the curved sections near Decarie and St. Hubert, and also to a lesser degree when the surface of the pavement was slippery.

A summary of the accidents from April 1, 1961, until March 31, 1962, shows that there were 675 accidents per 100 million vehicle-miles (MVM), of which 282 caused damage to the Boulevard. During this period 50 percent of the fatal accidents occurred as a result of vehicles breaking through the guide rail. On the other hand, 50 percent of all the accidents which caused damage to the Boulevard resulted from vehicles which struck the guide rail directly or indirectly, thus demonstrating the necessity of an effective guide rail.

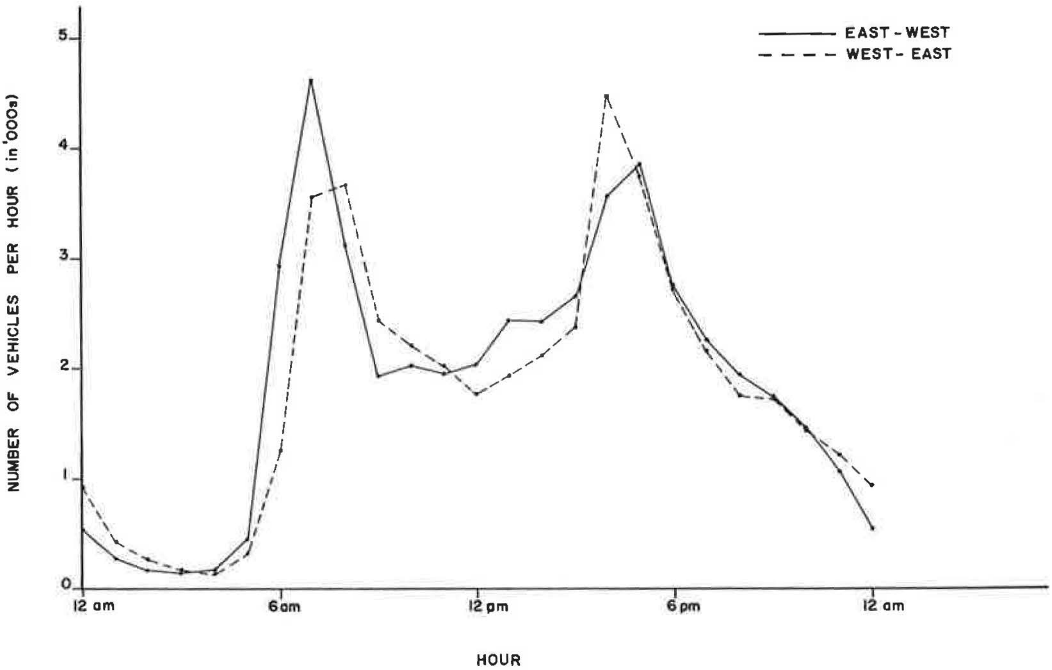


Figure 3. Hourly traffic in 1965 (summer day).

A preliminary analysis of all the accidents to date did not bring to light any specific cause of these accidents, but an analysis of those cases in which only the vehicles broke through the barrier provides the following data: (a) 6 drivers out of 12 were less than 25 years old, (b) 9 drivers out of 12 were less than 30 years old, (c) 7 accidents out of 12 occurred on a damp or slippery pavement, (d) 6 accidents out of 14 occurred between 1 a.m. and 4 a.m., (e) 4 accidents out of 14 occurred between 7 a.m. and 12 p.m., and (f) 4 accidents out of 14 occurred between 2 p.m. and 10 p.m. From this information it appears that these accidents have resulted from many causes, but principally the carelessness of the drivers and their youth. For this period, however, the Expressway has a fair accident record of 4.1 per 100 MVM.

By comparison with the U.S. expressways, the accident rate for the Montreal Expressway is higher. This is normal because for the same number of vehicles, the number of vehicle-miles is much higher on the average expressway than on the Montreal Expressway. There is also a greater potential for accidents than on expressways at large because of the large number of entrances and exits and because of the heavy traffic at certain times of the day. However, the accident rate on the Montreal Expressway is below the average for all U.S. highways, excluding expressways.

But, for the subsequent period, the number of accidents increased drastically and something had to be done to reduce the number of vehicles falling off the Expressway. This was important, because beneath the Expressway there are service roads on each side. Fortunately, no vehicle has fallen off the Expressway onto another passing or stopped vehicle below.

RECOMMENDATIONS

After the complete analysis of these data, it was recommended that, as soon as possible, a new guide rail system should be built with specific characteristics which will be described later. The following recommendations were made with the suggestion that they should be implemented immediately:

TABLE 1

NUMBER OF ACCIDENTS BY TYPE

PERIOD / TYPE			APRIL TO DECEMBER 1961	1962	1963	JANUARY TO MARCH 1964	APRIL 1, 1964 TO MARCH 31 1965
VEHICLE / VEHICLE	NORTH	DAY	127	228	279	56	340
		NIGHT	33	74	79	36	68
	SOUTH	DAY	130	197	265	51	304
		NIGHT	47	72	92	43	141
	INJURIES		48	75	62	20	79
	FATALITIES		5	2	1	0	1
VEHICLE / PARAPET	NORTH	DAY	22	41	67	17	51
		NIGHT	18	34	24	17	11
	SOUTH	DAY	11	30	44	10	56
		NIGHT	22	26	29	12	25
	INJURIES		20	31	28	14	13
VEHICLE / POLE	NORTH	DAY	13	18	34	16	38
		NIGHT	8	20	16	9	27
	SOUTH	DAY	9	15	29	7	19
		NIGHT	13	22	37	9	15
	INJURIES		23	26	39	10	33
	FATALITIES		3	4	1	2	2
MISCELLANEOUS	NORTH	DAY	1	2	3	4	20
		NIGHT	2	2	0	0	11
	SOUTH	DAY	2	0	2	1	9
		NIGHT	4	2	0	0	15
	INJURIES		3	2	1	0	17
	FATALITIES		0	0	0	0	0
TOTAL	INJURIES		94	134	130	44	142
	FATALITIES		11	8	2	2	3
			461	783	1000	289	1166

TABLE 2

SUMMARY OF ACCIDENTS

DESCRIPTION	YEAR (MAY TO MAY)			
* PER 100 M.V.M.	'61 '62	'62 '63	'63 '64	'64 '65
NUMBER OF ACCIDENTS *	675	856	970	1010
NUMBER OF VEHICLES * FALLING OFF THE EXPRESSWAY	13.2	23.2	2.9	1.8
NUMBER OF FATALITIES * RESULTING FROM VEHICLES FALLING OFF THE EXPRESSWAY	2.0	4.6	0	0
NUMBER OF ACCIDENTS * CAUSING DAMAGE TO THE EXPRESSWAY	282	318	274	285
DAMAGE TO THE EXPRESSWAY	\$45,100.	\$100,500.	\$70,300.	\$80,100.
ESTIMATED DAMAGE TO VEHICLES	\$300,000.	\$390,000.	\$450,000.	\$660,000.
NUMBER OF INJURIES *	117	141	139	127
NUMBER OF FATALITIES *	4.1	11.1	2.9	2.7

1. The type of accidents showed that an increase in policing the Expressway would reduce their number. The policing should mainly be directed at reducing the number of users exceeding the speed limit and/or crossing unnecessarily from lane to lane.

2. The speed limit should be reduced to 45 mph from 55 mph.

3. The analysis showed a large number of accidents occurring because the pavement was slippery. The application of an anti-stripping pavement surface was recommended.

4. Lighting measurements taken during a day in January showed a very low light intensity on the pavement. The light poles are located on the median strip of the 80-ft wide expressway at a spacing of 120 ft. The light intensity measurements at the pole location varied between 1.2 and 2.2 foot-candles, close to the pole, and 0.6 and 1.2 on the outside lanes. At mid-distance between poles the light intensity varied between 0.5 and 0.6 foot-candles on the inside lanes, and 0.3 and 0.4 on the outside lanes. The variations at the same location were mainly dependent on the age and cleanliness of the lighting systems.

Although the light intensity measurements were taken at a temperature of 36 F, it was felt that the lighting of the Expressway was much below the requirements of the I.E.S., which requires an average value of 2.0 foot-candles in the worst conditions. It was also noted from the accident records that some occurred because of poor visibility, especially during poor weather conditions. Therefore, it was suggested that the lighting system be changed for one giving a better performance.

5. The accident analysis also showed that a re-marking of the pavement lane width would be appropriate. Suggested plans were sent with the original reports.

GUIDE RAIL SYSTEM RESEARCH AND DEVELOPMENT

Once the main recommendation of the preliminary study was approved, a research and development study of a new guide rail system was undertaken. It was limited to 8 different types of guide rails which were suggested for study by the Department of Highways and by guide rail manufacturers. The first phase was to make a review of the available research, followed by the establishment of criteria for the evaluation of

Previous Research

A review was made of the technical literature available on the question of safety on elevated expressways with particular attention given to safety barriers. The main researchers on that subject were found in California and New York, at the General Motors Company, and in Sweden.

With this background and our own experience, the characteristics of a well-designed safety barrier for the relevant conditions were arrived at as follows: (a) strength of the whole unit, (b) minimum rebound, (c) moderate deceleration of the impacting vehicle, (d) strength of the guide rail, (e) uniform sliding along the barrier, (f) minimum damage to the guide rail, (g) minimum cost of construction, (h) easy maintenance, (i) minimum damage to the impacting vehicle, (j) appearance, and (k) ease of modification of existing guide rail.

In the calculations for the rails and anchorages, the following factors should be taken into account: weight of vehicle, speed of vehicle, angle of impact, resilience of vehicle at its point of impact—tires and body work, resilience of posts and anchorages, resilience of rail, resilience of rail bracket, and effect of braking.

The problem is complex and it is theoretically impossible to determine the absolute value of many of these factors; thus, the most practical and valid method for our conditions proved to be the performance of destructive tests.

Conditions for the Tests

The most important conditions for the tests were the speed and impact angle, which were kept constant throughout the tests.

Speed.—The test speed was set at 55 mph which was the maximum legal speed on the Boulevard. A different speed could have been used but taking into account the angle of impact chosen, it was felt that this speed was the most valid.

Angle of Impact. —The angle of impact is the angle at which the vehicle strikes the barrier. It is a function of the speed of the vehicle and the lateral distance between the vehicle and the guide rail.

An analysis was made from the accident reports to determine the optimum angle of impact for these tests, taking into account the conditions which prevail on the Expressway; but it did not provide a conclusive figure. An angle of impact was therefore chosen which corresponds to the worst conditions and also took account of the results of similar tests carried out in the United States.

From the American tests and the conditions on Metropolitan Boulevard, it was concluded that a speed of 55 mph and a lateral displacement of 28 ft would given an angle of impact of about 28 deg. This figure is only valid if the vehicle is in perfect condition and the coefficient between the tires and the pavement is high.

The Californian tests carried out in 1953 showed that the vehicle reaction is much the same for all angles of impact between 5 deg and 10 deg. Their accident records for guide rail showed that in most cases the angle of impact was between 15 deg and 20 deg. The chosen angle of impact of 21 deg was intended to allow for these results as well as for the actual conditions on the Expressway. The worst accident conditions were also considered; for example, the case of a driver losing control of his vehicle when at full speed before and even immediately after the impact.

Control System

Projection System. —Preliminary tests were made to determine the best method for projecting the impacting vehicle in order for it to strike the barrier at the required speed and angle of impact.

The technique for doing this was developed on an unused runway where practical methods were developed for controlling these factors. The testing was then continued on the Expressway. The first test was carried out to check the effectiveness of the projection system and to verify the test conditions. After this test had proved successful, the program was completed on an unused section of the elevated expressway west of Decarie Circle, where there was no possibility of obstructing the flow of traffic.

The projection system used consisted of a wooden trough securely attached to the pavement and providing a positive trajectory for the impacting vehicle. The front and back wheels on one side of the impacting vehicle travel in the trough during the approach (Figs. 4, 5).

Installation of the Guide Rail. —The suppliers of guide rails provided and installed their prototypes and modified the existing installation, in some cases changing it completely but they had to have the original system in place.

Figure 4. Projection system for the impacting vehicles for all the tests (start).

Figure 5. Projection system for the impacting vehicles for all the tests, except for two tests on guide rail Model H (just before impact).

Apparatus During Test. —Before beginning the actual tests, it was arranged that films would be made of each test in order to obtain an accurate and permanent record on which the calculations and conclusions could be based. The films enabled a close step-by-step study to be made of the tests after the actual testing was completed. It was possible to examine the behavior of the vehicle at the moment of impact and immediately afterwards when the rear of the vehicle struck the rail. These films proved to be extremely helpful and permitted all the required data to be obtained.

Figure 6 shows the installation of the platforms for the cameras, the installation of the radar for measuring speed and of the painted grid on the road for pinpointing the position of the vehicle at all times.

DIFFERENT TYPES OF GUIDE RAILS

The results obtained are summarized in Table 3. Figure 7 shows the different types of guide rails tested and also the path followed by the impacting vehicles.

The first test was mainly performed to recheck the calibration data, which were set for each vehicle during preliminary testing on the runway. It also shows that at high speed and a large angle the curb alone has very little effect (Fig. 8).

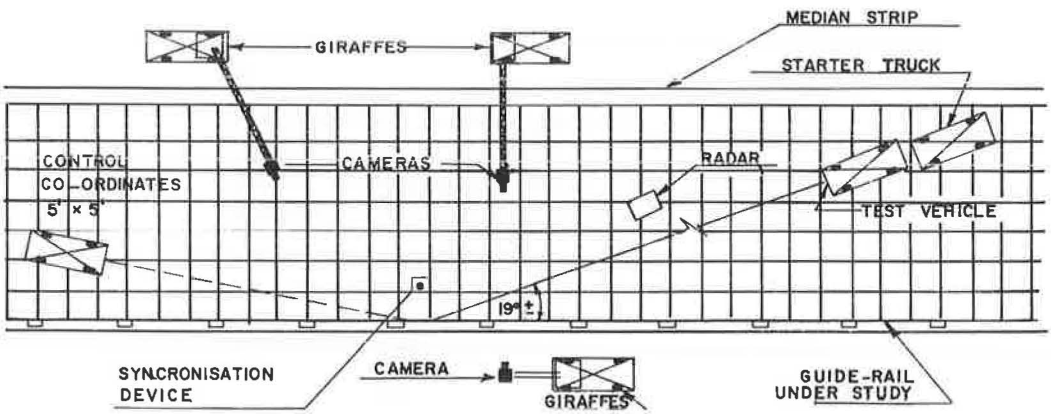
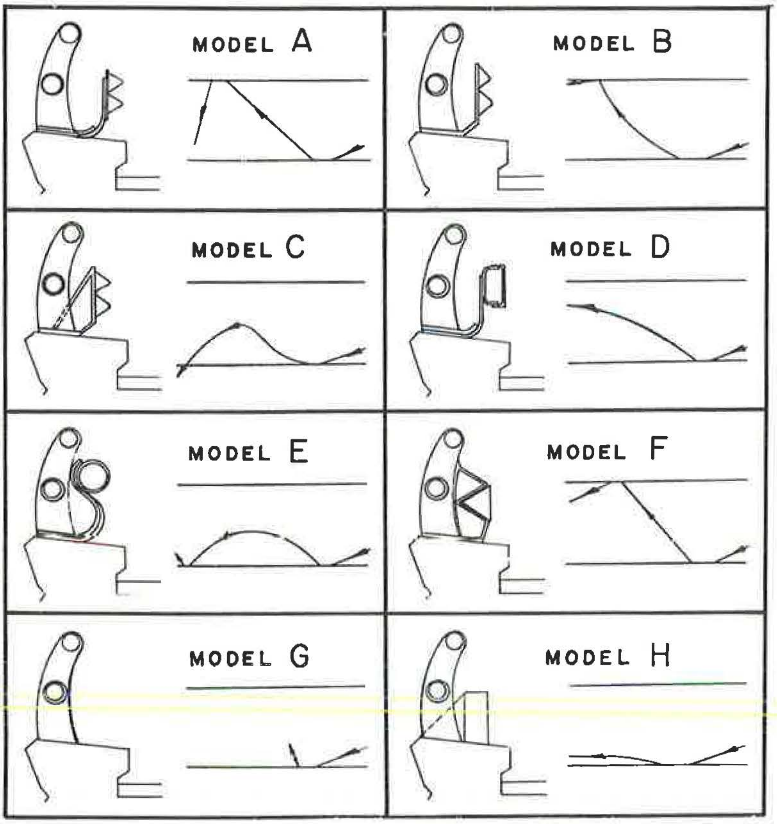


Figure 6. General arrangement of test site.

TABLE 3

RESULTS OF TESTS ON DIFFERENT TYPES OF GUIDE RAIL

Test No	Test Vehicle No	Identification	VEHICLE Make	Year	Weight	SPEED (MPH) Before Impact Radar	Impact Film	After Film	IMPACT ANGLE Expected	Film	REBOUND-ANGLE After Impact	At Max. Point	Maximum Rebound (Ft.)	Contact Distance With The Curb.	Average Deceleration Lateral Ft / Sec^2	Longitudinal Ft / Sec^2
1	1	Curb.	Plymouth	1955	3700	64	64	-	21°	20°	14°	20°	36'	15'	80	33
2	3	Median	Pontiac	1952	3500	56	53	35	21°	19°	14°	14°	36'	19'	92	58
3	2	Model "A"	Chevrolet	1956	3450	57	59	45	21°	19°	14°	20°	36'	15'	80	33
4	4	Model "B"	Plymouth	1956	3580	44	42	36	21°	19°	8°	9°	13'	9'	70	14
5	12	Model "B"	Monarch	1953	3200	57	57	35	21°	20°	9°	24°	36'	28'	69	28
6	6	Model "C"	Plymouth	1956	3670	55	57	44	21°	18°	11°	9°	20'	40'	124	47
7	5	Model "D"	Ford	1955	3570	50	53	38	21°	18°	13°	9°	28'	12'	113	67
8	8	Model "E"	Buick	1953	4000	50	58	44	21°	19°	10°	10°	17'	17'	210	169
9	11	Model "F"	Chevrolet	1952	3650	50	50	32	21°	20°	15°	25°	36'	30'	67	43
10	10	Model "G"	Studebaker	1957	3420	60	52	-	21°	19°	100°	100°	16'	13'	160	460
11	7	Model "H"	Ford	1954	3570	54	56	44	17°	15°	6°	9°	11'	37'	84	48
12	9	Model "H"	Mercury	1955	3980	25	29	15	21°	22°	9°	4°	7'	50'	11	10
13	9 A	Model "H"	Meteor	1956	-	50	53	35	21°	20°	8°	7°	10'	22'	90	57



Median Strip

The existing median strip was tested to determine its performance. Figure 9 shows the results obtained.

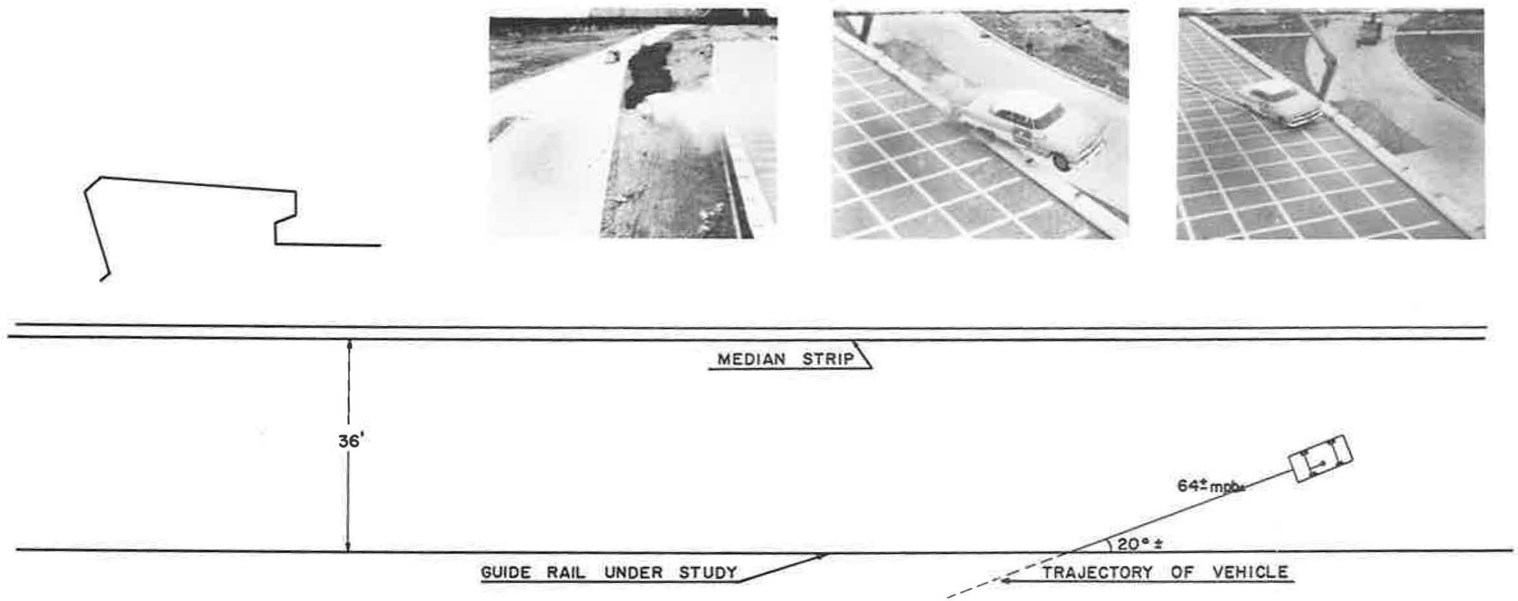
This median strip is made of concrete with a curb 10 in. high and 6 in. wide. On top of this curb is a vertical wall sloping toward the outside of the lane for a height 14 in. over the curb. This test produced a very large rebound which was considered highly unsatisfactory.

Steel Guide Rail—Model A

Model A (Leclerc et Fils) is a 12-gage steel railing connected to the post by spring steel metallic supports ½ in. thick and 6 in. wide. An additional ½-in. plate is inserted under the posts for extra rigidity. This system is relatively flexible, provides for easy setup, and could be added to the actual system without removing the post. The installation required loosening the nuts sufficiently to allow the slotted reinforcing plate to be properly positioned under the posts and final tightening of the nuts. The height of rails could be adjusted in a similar manner.

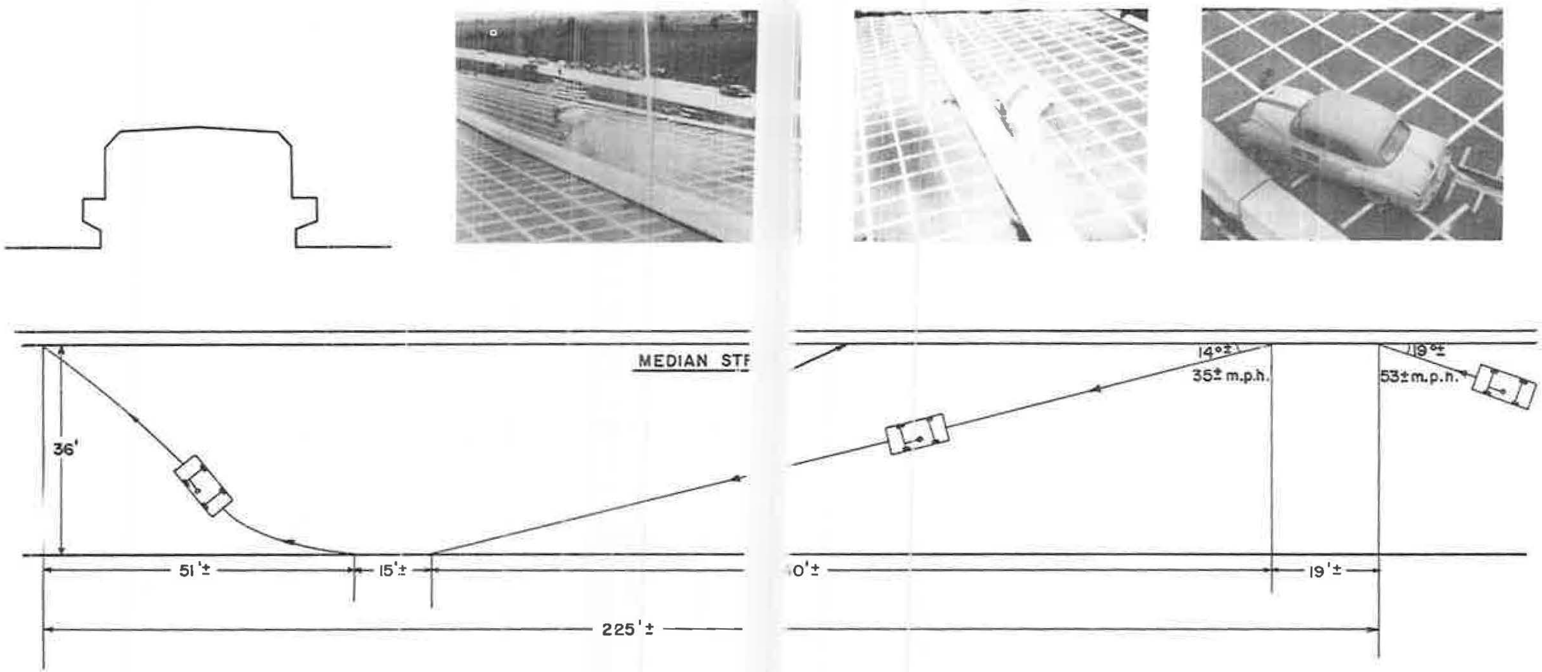
The plate joining the railing to the supporting tends to assist in maintaining the original railing section under vehicle impact. This is important, since the resistance under the impact is thus increased. The rail centerline is approximately 20 in. above the pavement.

The test on this L-shaped system of a rail fastened to flexible supports produced a considerable rebound. The damage to the safety barriers resulting from this test was a broken post dropping to the ground below, 4 aluminum tubes disjointed and the railing left bent (Fig. 10).



SCALE : 20' = 1"

Figure 8. Curb test No. 1 of calibration data.



S LE : 20'= 1"

Figure 9. Te lo. 2 of median strip.

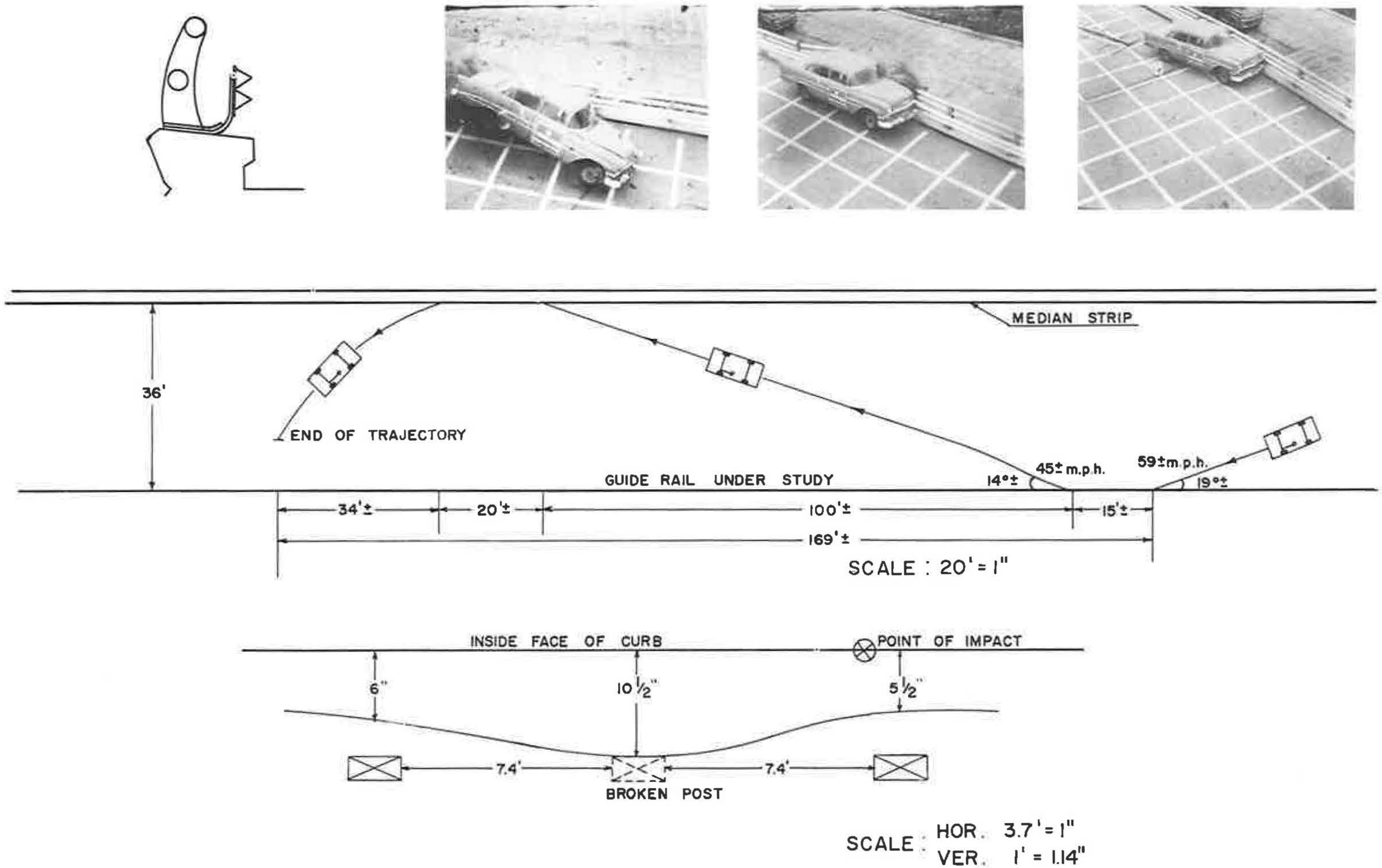


Figure 10. Test No. 3 of steel guide rail model A.

Steel Guide Rail—Model B

Model B (Roads Department, J. Lacroix) is a 12-gage railing with relatively flexible supports connecting it to the existing anchor bolts. Both components are SAE 1020 steel. The supports are $\frac{5}{8}$- × 8-in. angle-shaped plates directly connected to the rail. No reinforcing plate is employed. Installation requires removal of the existing posts since only 4 holes are provided in the base of the supports for bolting to the post.

The holes are oval, permitting limited horizontal adjustment of the supports. This system does not provide for vertical adjustment, so it is impossible to maintain uniform rail height on vertical curves. The rail centerline is 21 in. from the pavement, while the rail itself is 5 in. from the curb. Two tests were conducted with this system, as the vehicle used in the first test did not reach the required speed.

First Test. —This test, at 42 mph, caused the following damage to the safety barrier: one partially broken post, 2 bent rails and a few bent supports (Fig. 11).

Second Test. —This test was performed at 59 mph and the damage to the barrier was one twisted post, one pulled pipe, 3 bent supports and 2 bent rails. The rebound (Fig. 12) was important.

Steel Guide Rail—Model C

Model C (Roads Department) is a modification of Model B. For increased rigidity, 2 reinforcing plates were added to each side of the angle bracket support. During the tests, there was a considerable amount of rebound (Fig. 13). Following the initial impact of the test vehicle on the rail at the desired location, it bounced further on into the original barrier, broke it, and then fell to the ground.

The damage to the barrier was one post and 3 tubes broken and projected down below, 2 bent rails and one partially pulled out support.

Steel Guide Rail—Model D

Model D (Aluminum Company of Canada) consists of a U-shaped aluminum alloy, the rail is connected to the posts by means of a 12-in. wide angle-shaped aluminum covered steel strip. The base of this connector is reinforced by a similar type steel plate, and the top is reinforced by another U-shaped aluminum alloy section 48 in. in length. The rail centerline is 23 in. above the pavement while the front of the rail is 2 in. behind the face of the curb. The installation of this type safety barrier requires the entire raising of the existing posts.

During the test, considerable rebound was noted (Fig. 14). The damage caused to the safety barrier was one broken post, 2 distorted supports, a bent rail, and 26 disjointed pipes because of faulty original installation.

Aluminum Guide Rail—Model E

Model E (Aluminum Company of Canada) is a B51STA-type tubular aluminum rail of 0.187-in. wall thickness and $8\frac{5}{8}$-in. diameter. It is connected to the existing safety barrier post by an aluminum-coated double steel strip 18 in. in width by $\frac{5}{8}$ in. thick. This strip consists of leaves separated by a $\frac{3}{16}$-in. thick hard rubber membrane. The tubes are attached together by sleeves. The posts must be raised to install this type barrier. The rail centerline is at an elevation of 24 in. above the pavement and the front of the rail is 3 in. behind the face of the curb.

Following initial impact, the vehicle rebounded and struck another barrier 180 ft farther on (Fig. 15). This barrier did not fail. After this second impact, the vehicle continued and came to rest a little farther down the road. On examination of the railing Model E, we noticed that the damage was relatively light. Only the tubular section had been flattened out and also bent slightly.

The rail struck by the second vehicle impact was a Model G.

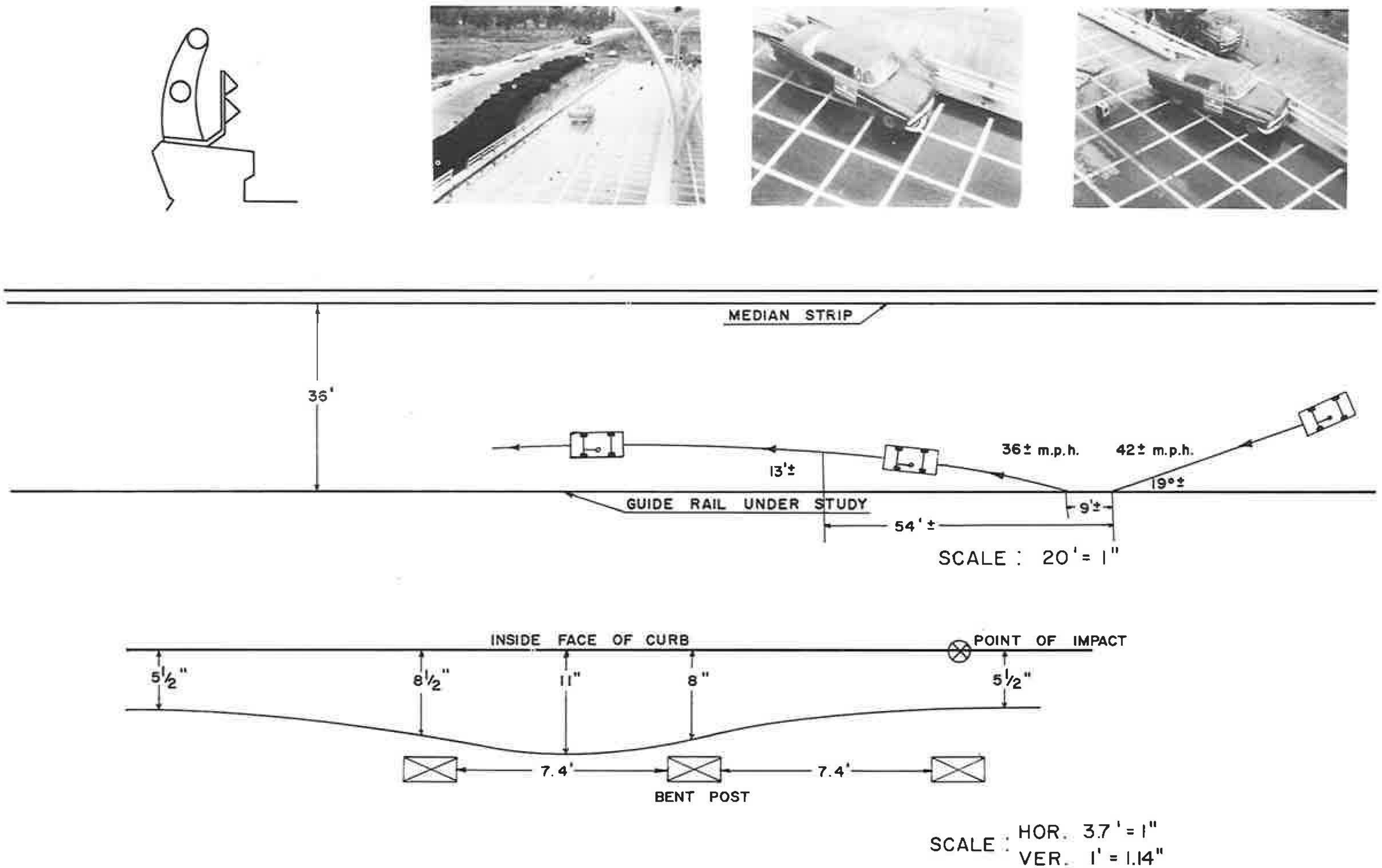


Figure 11. Test No. 4 of steel guide rail at 42 mph (model B).

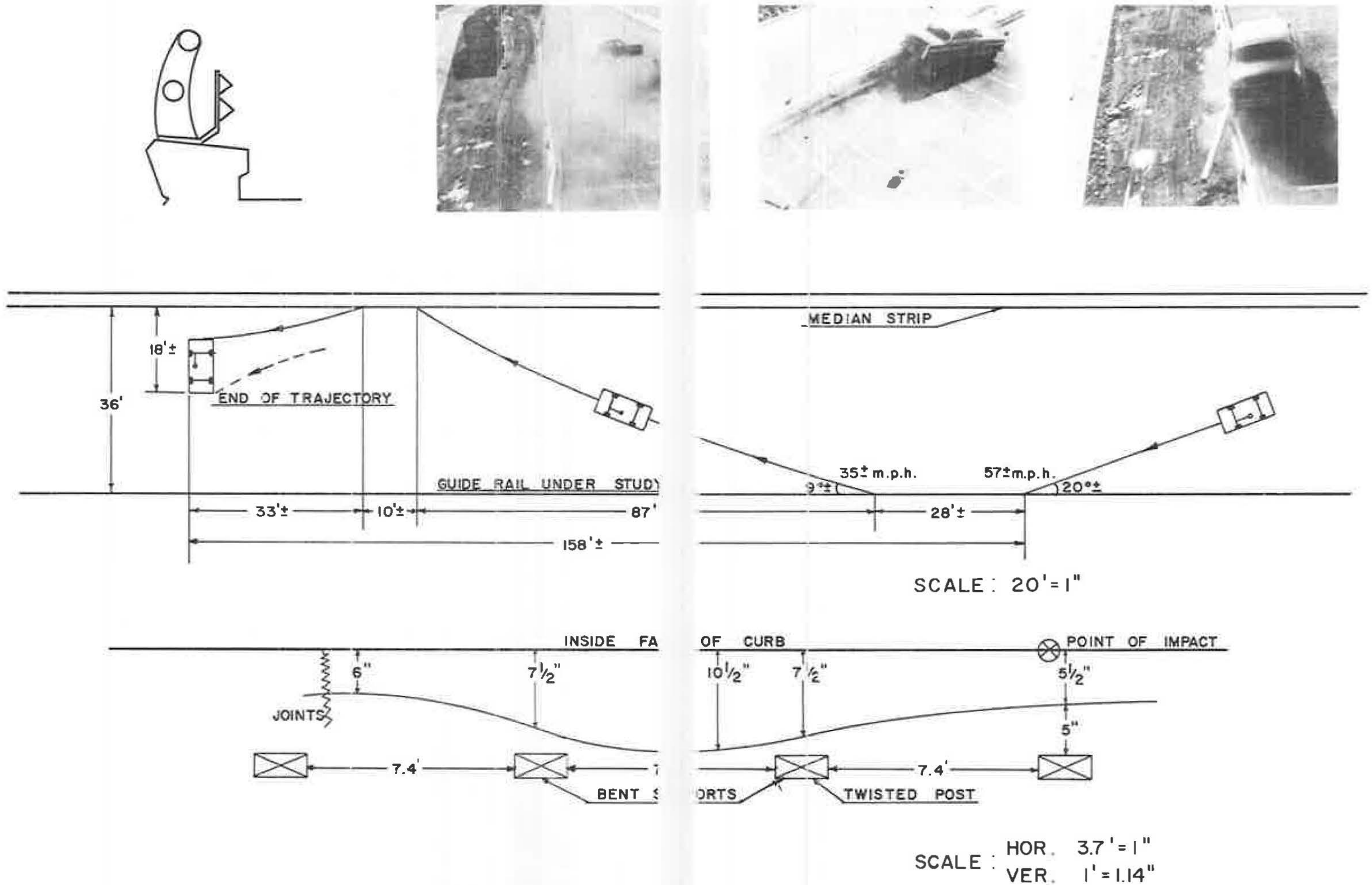


Figure 12. Test No. 5 o el guide rail at 59 mph (model B).

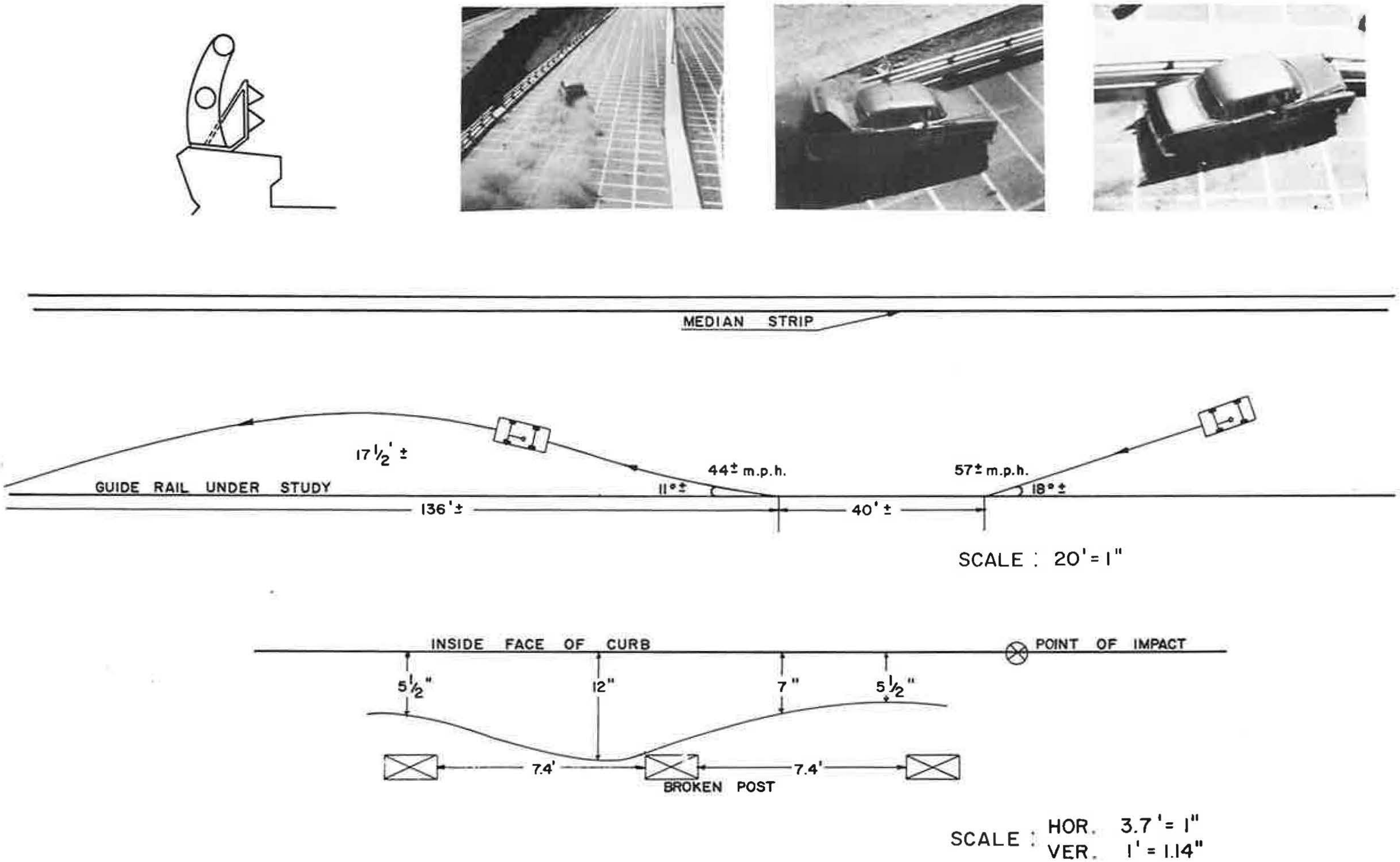


Figure 13. Steel guide rail test No. 6 showing rebound (model C).

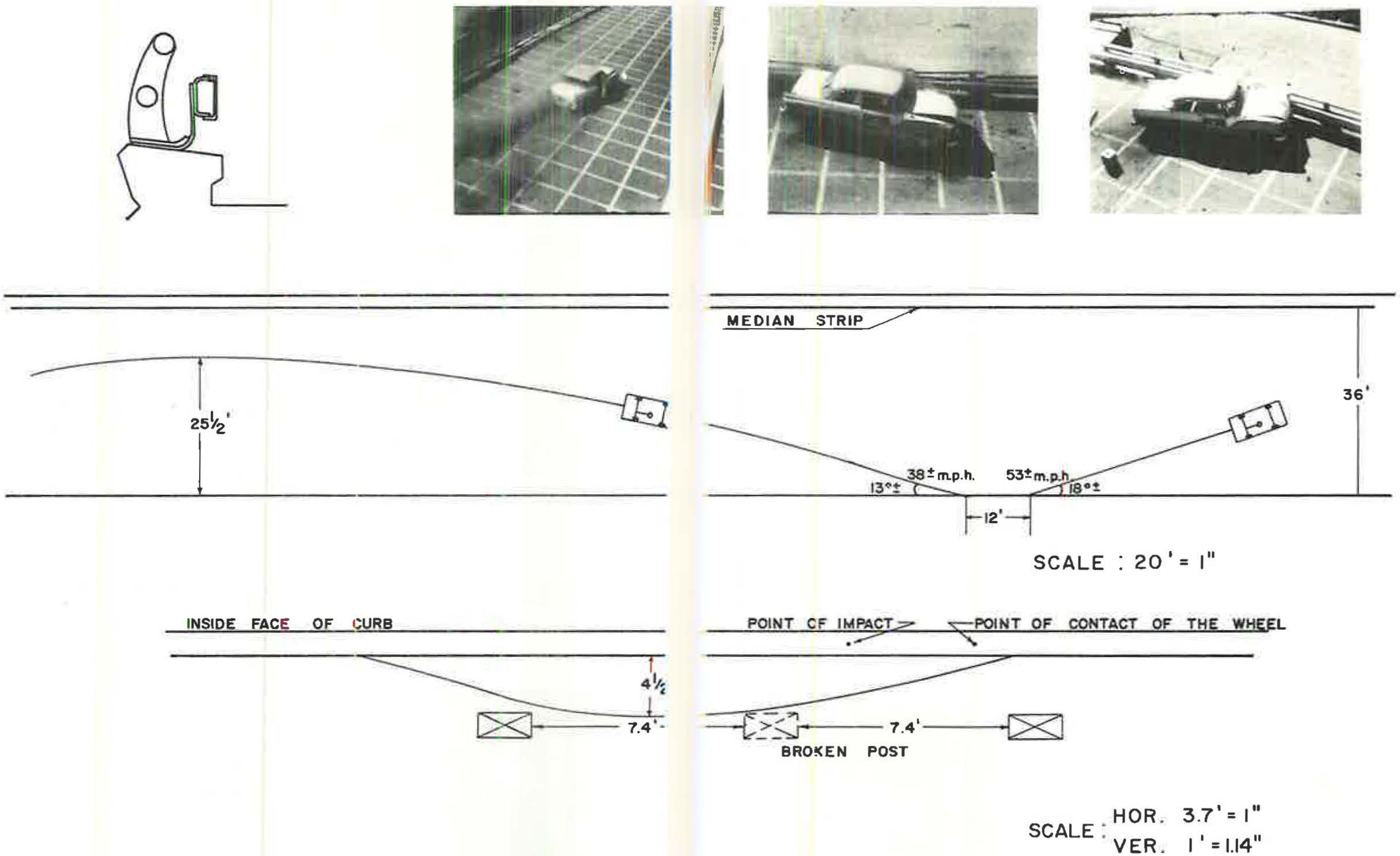


Figure 14. Aluminum guide r est No. 7 showing rebound on model D.

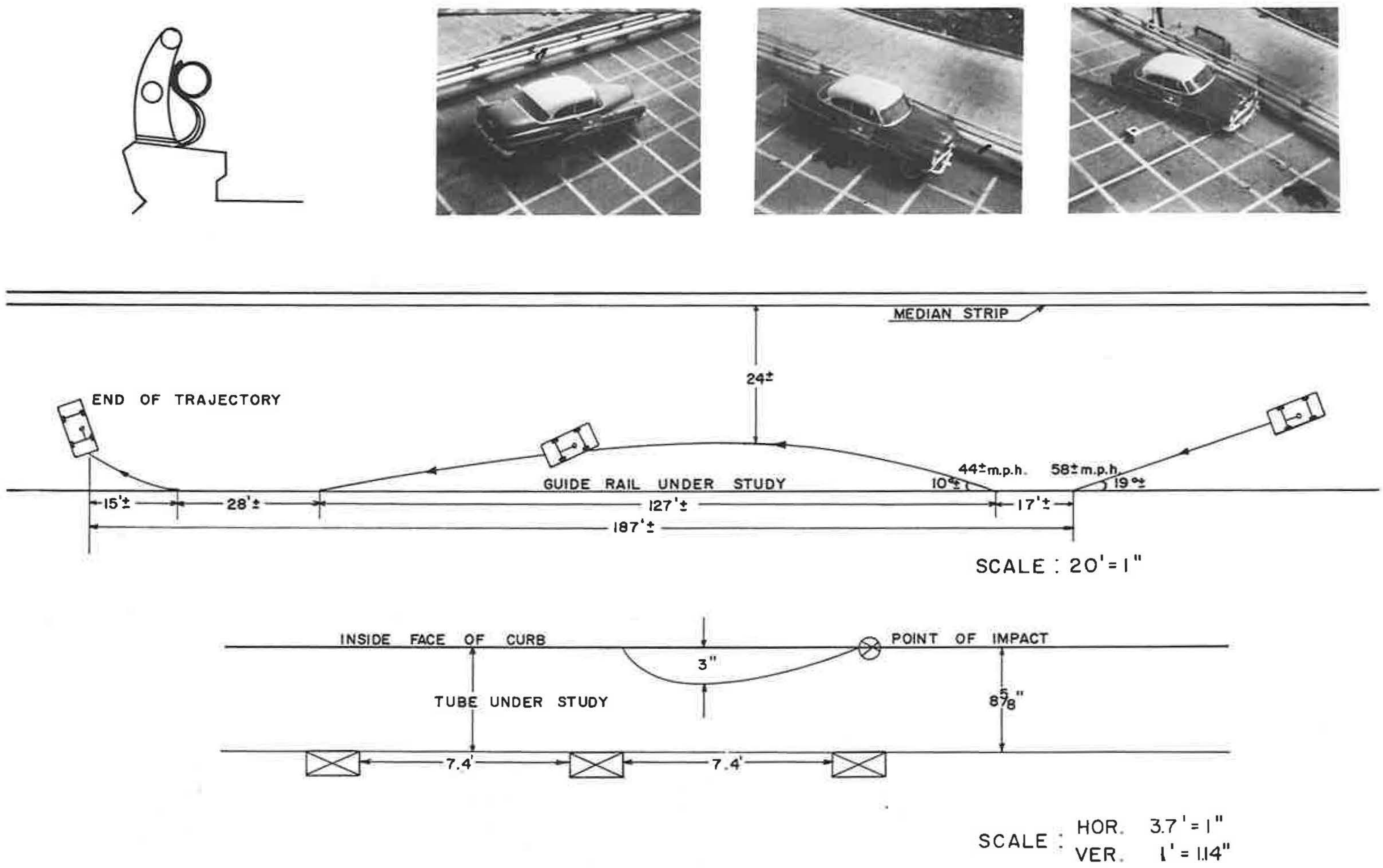
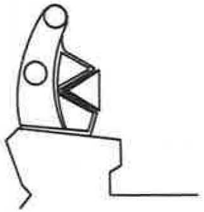
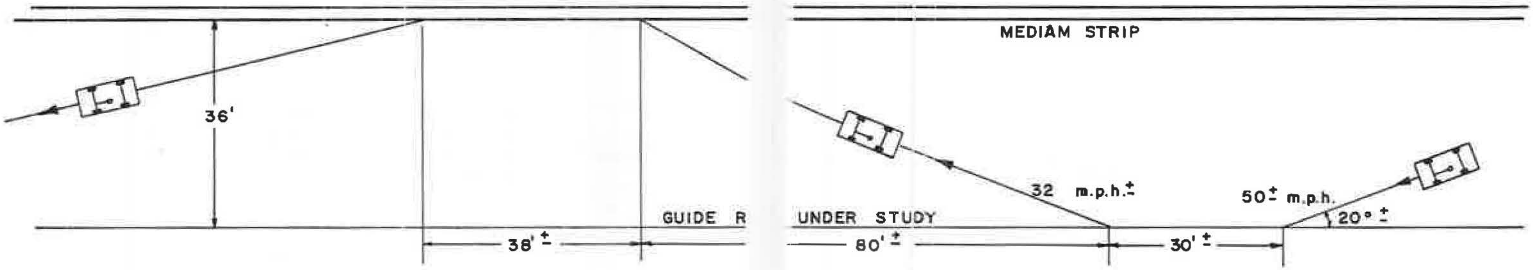
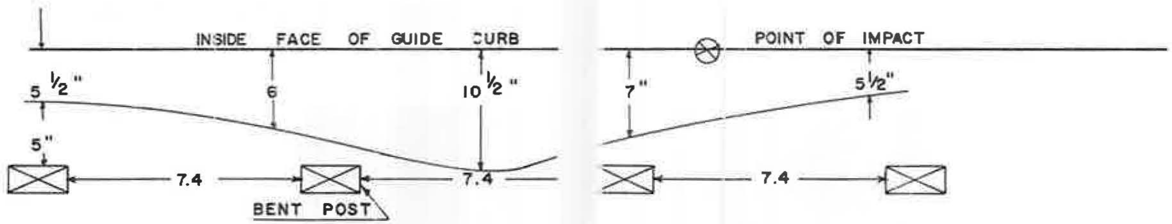


Figure 15. Aluminum guide rail test No. 8 showing rebound on model E.





SCALE : 20 = 1"



SCALE : HOR. 3.7' = 1"
VER. 1' = 1.14"

Figure 16. Aluminum guide test No. 9 of rebound on model F.

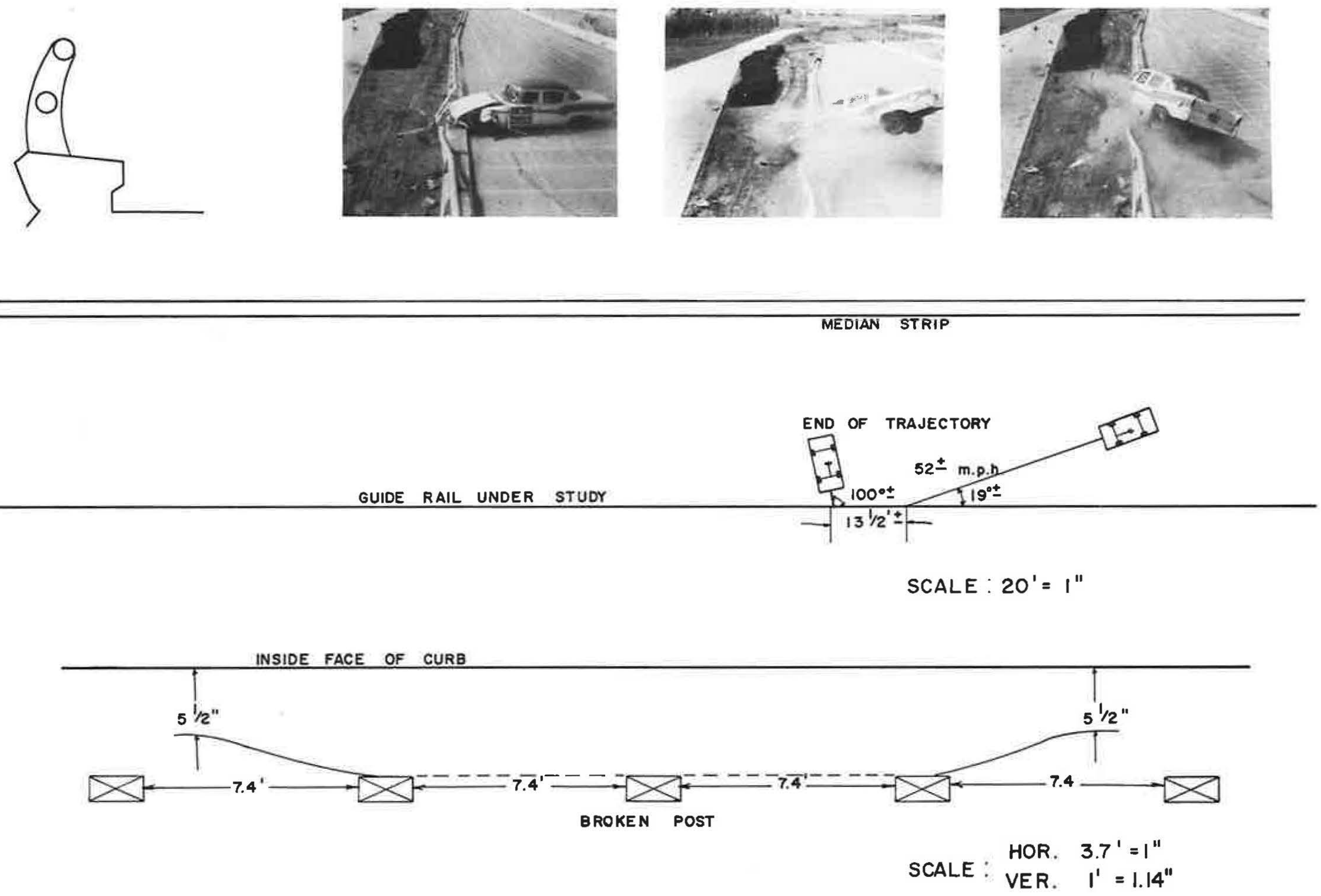


Figure 17. Aluminum guide rail test No. 10 showing abrupt halt of auto by model G.

Aluminum Guide Rail—Model F

The installation of Model F (Aluminum Construction, Inc.), although time-consuming, is nevertheless relatively simple, except at the supports. It consists of units of identical triangular section fastened and assembled so as to form a single railing. Each unit is reinforced by interior ribs to increase its section modulus. This railing is fastened to the support by means of $^3/_8$- × 6-in. aluminum strip.

The method developed for fixing the aluminum strip to the support is relatively new. A rubber and plastic composition is used to cold weld the top part to the support. The lower section is fastened directly to the concrete with anchor bolts. The rail is then bolted to the aluminum strip slotted with 2-in. oval holes, by two $^5/_8$-in. bolts, one in each triangular unit resting against the support.

These bolt holes are to allow for expansion and adjustment of the rail. Model F has a good appearance. The height of the rail and its distance from the face of the curb are similar to the other types, being approximately 22 in. and 5 in., respectively.

After initial impact the rebound was of sufficient intensity to: (a) hurl the vehicle across the traffic lanes, (b) strike the center wall, (c) slide along it for a short time, and (d) come to rest a little farther away (Fig. 16). The damage to the barrier was one post partially broken, and one bent railing.

Aluminum Guide Rail—Model G

Model G is a modification of the existing aluminum safety barrier and consists of post and tubes. The ACD 25 posts are sectional aluminum, bent and welded. The tubes are 5 in. in diameter and 0.125 in. in wall thickness. They are joined by dowels and tied by fasteners.

After initial impact, the test car was abruptly halted by the guide rail (Fig. 17). The damage was 3 posts twisted or partially broken at their base and the tubes torn and twisted.

Concrete Guide Rail—Model H

Model H, built to the Quebec Roads Department Specifications for Bridges and Structures, consists of a small reinforced continuous concrete wall poured directly on the existing curb (Fig. 18). To insure a perfect bond, care was taken to anchor the wall with bars at 18-in. centers and chip one to 2 in. from the existing curb surface. This wall is trapezoidal, 12 in. in height, 14 in. at the base and 6 in. across the top. At each post a gutter is provided to prevent water accumulation. The wall is located $4^1/_2$ in. from the face of the curb with a maximum height of 24 in. above the pavement. The construction of this type of guide rail is rather slow and cumbersome.

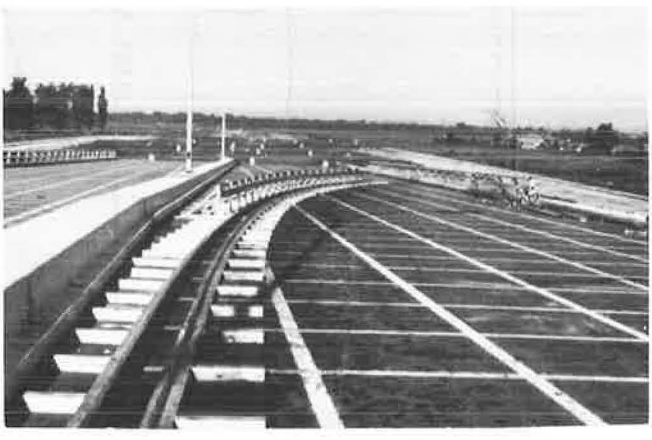
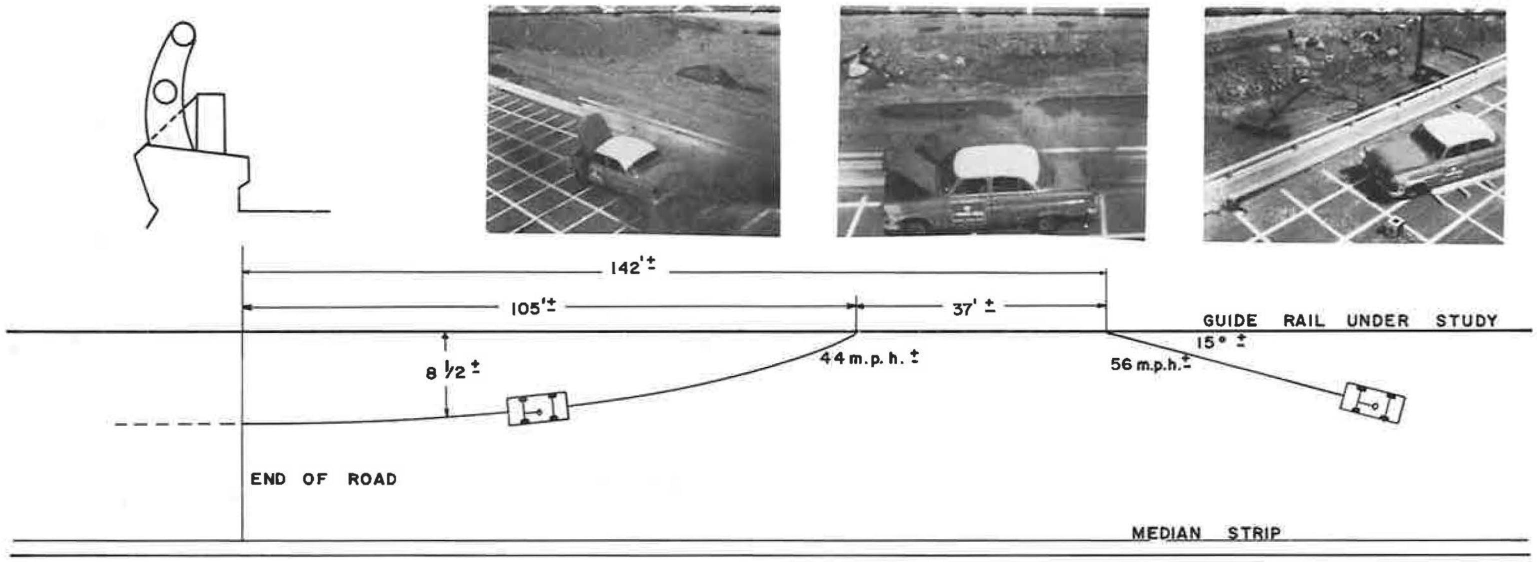


Figure 18. Method of propulsion with additional guide rails.



SCALE : 20'=1"

Figure 19. Concrete guide rail test No. 11 of model H. No damage.

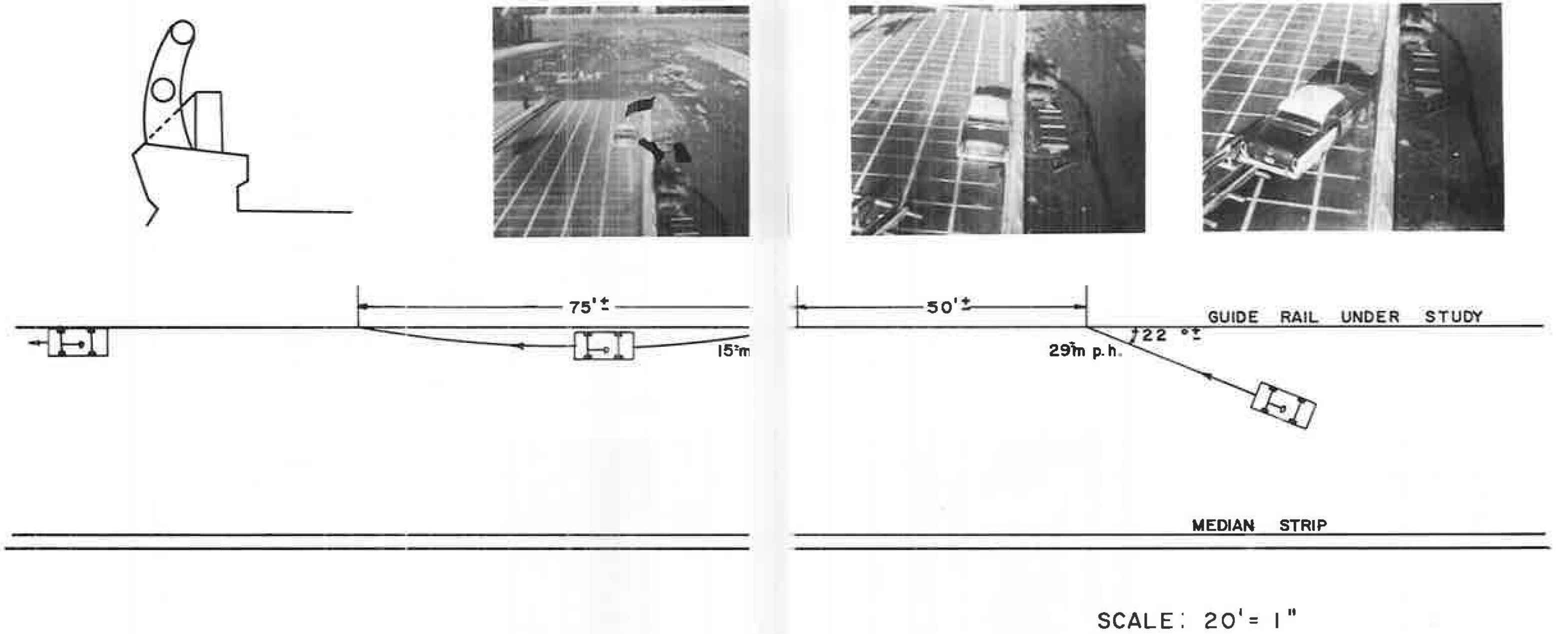


Figure 20. Concrete guide rail te lo. 12 of model H at 21 deg. No damage.

Figure 21. Damages to guide rail model H after the three tests.

Three tests were made on the concrete wall-type guard railing, one at a 17-deg impact angle and 2 at a 21-deg angle. Each impact on this rail produced only a slight rebound. To obtain the 21-deg impact angle, it was necessary to erect a supplementary guiding system, as the test car's centripetal force would have thrown the vehicle out of the ordinary guiding system (Fig. 18). Two tests at the 21-deg impact angle were made because the gas pump of the first car failed about 100 ft from the starting point.

First Test (17° impact)—No damage was noted on the guide rail (Fig. 19).

Second Test (21° impact, 28 mph)—This test caused no damage to the guide rail (Fig. 20).

Third Test (21° impact, 52 mph)—In the course of this test the top horizontal tube between the posts was partially displaced, but the concrete wall suffered no damage (Figs. 21, 22).

ANALYSIS OF RESULTS

The same conditions were maintained for all the tests on the guide rails except for Nos. 4, 11 and 12 which were repeats. The angle of impact and speed were kept generally the same irrespective of which side of the boulevard was used for the tests.

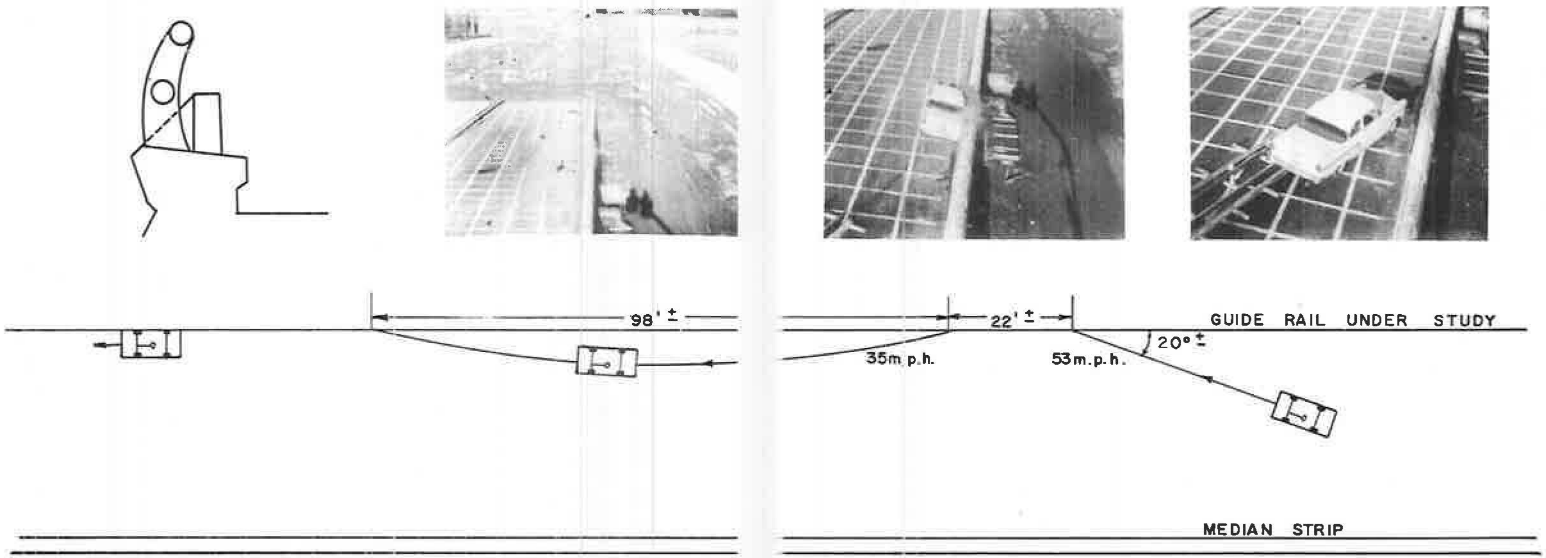
The relative behavior of the systems was observed in detail during the studies, but the stresses in the components of the barriers were not measured. The speeds and angles of impact were carefully controlled and repeat tests were made if these factors varied significantly. Similarly, the weight of the impacting vehicles was kept more or less the same to simplify the interpretation of the results. The slightly different types of cars could have caused variations in the results, but not of any significance.

Strength of Barrier

It is always possible to obtain a barrier which is sufficiently strong by strengthening its components.

Minimum Rebound

The films and other data showed that the more rigid the guide rail the lower the rebound. When the impacting vehicle strikes a rigid barrier, the front of the vehicle makes contact and is projected back onto the highway; but almost simultaneously the back of the vehicle strikes at nearly the same place as the front and is, in its turn, rebounded. The effect is to straighten the vehicle and direct it along the road close to the barrier. If, on the other hand, the rail is deformed under the impact, the back of the vehicle does not rebound with the same force but tends to follow the contour of the



SCALE : 20' = 1"

Figure 22. Concrete guide test No. 13 of model H. No damage.

deformation. Instead of sliding along the barrier, the vehicle takes a diagonal trajectory, crosses the road and in some cases strikes the median strip.

Among all the types of guide rails tested, the concrete wall is the strongest and the rebound is the least. In other cases, the amount of rebound seems to be inversely proportional to the strength of the barrier.

The setback of the guide rail with respect to the curb is an important factor. If it is too much, the vehicle has a tendency to be overturned by its contact with the curb (assuming that it does not ride over it). This occurred during a test on the central median strip. If the setback is too little, only the guide rail is brought into play. The body and the wheels should strike at almost the same time to obtain the best performance.

Deceleration

The calculated average decelerations obtained during impact are approximate and they appear to be reasonable except in the case of Model G. To use these values to determine the seriousness of the accidents, it would have been necessary to install accelerometers inside dummies in the vehicles. The tests show that the concrete wall gives the least average deceleration.

Uniform Sliding

All the models tested, except Model G, resulted in a fairly uniform sliding along the rail.

Damage to Guide Rail

The existing guide rail breaks up as soon as it is hit by a vehicle which is out of control; the pieces could cause considerable danger particularly on the service roads below the elevated section. The installation of a second line of defense such as an additional guide rail should reduce this to a minimum especially if the latter consists of a concrete wall.

Maintenance of Barrier

The cost of repair of the barrier after an accident varies considerably. In the case of the concrete wall, the cost of repairs is minimal while in the case of steel and aluminum, especially the latter, it is considerable. It is difficult to make a positive comparison but the figures obtained on this question appear reasonable.

Damage to Vehicles

The damage caused to the impacting vehicles varied depending on the kind of guide rail under test. However, the concrete wall caused the least damage.

Appearance

Without question, Model F has the best appearance while Model H is the least attractive; the other types of rails vary in their appearance. The appearance of the barrier has some influence on drivers; drivers will tend to keep farther away from Model H than Model F.

Cost of Guide Rail

The cost of construction of the guide rail varies considerably depending on the type. If it was necessary to choose between 2 types of guide rails of the same performance, a direct cost comparison would have been necessary. However, this was not the case; steel guide rail is a little cheaper than concrete which, in turn, is cheaper than aluminum.

Summarizing, the tests showed that the concrete wall fulfills the largest number of the requirements; this is not to say that concrete must be employed to the exclusion of other alternatives, but only that it gave the best results under the test conditions.

CONCLUSIONS

The installation recommended involved the retention of the existing safety barrier and the addition to it of a second line of protection, namely, a small concrete wall built on the existing curb (Fig. 23). This recommendation was based mainly on the very small rebound by the impacting vehicle from this kind of wall, a particularly important feature for elevated highways. Too great a rebound would set in motion a series of accidents in which the vehicle is thrown into the paths of other oncoming vehicles, resulting in a chain of accidents.

On the ramps, where vehicles travel in a single lane, the question of rebound is not so important as on the elevated section. For this reason, it was recommended that the type of guide rail built should be capable only of restraining impacting vehicles (Fig. 24). The cheapest installation consists of a steel guide rail, with the rail itself of W form, supported by steel brackets. This type must resist impact with a minimum of deflection and must therefore have its end encased. Also, it is essential to eliminate the hazards which arise if the ends are not protected.

Summarizing, the tests made on 8 different kinds of guide rails for use on a heavily traveled elevated expressway showed that a concrete wall gives the best performance under the test conditions.

PERFORMANCE OF CHANGES

Since the new guide rail system was finished in November 1963, the legal maximum speed was reduced to 45 mph and the new lane marking system was painted. There has

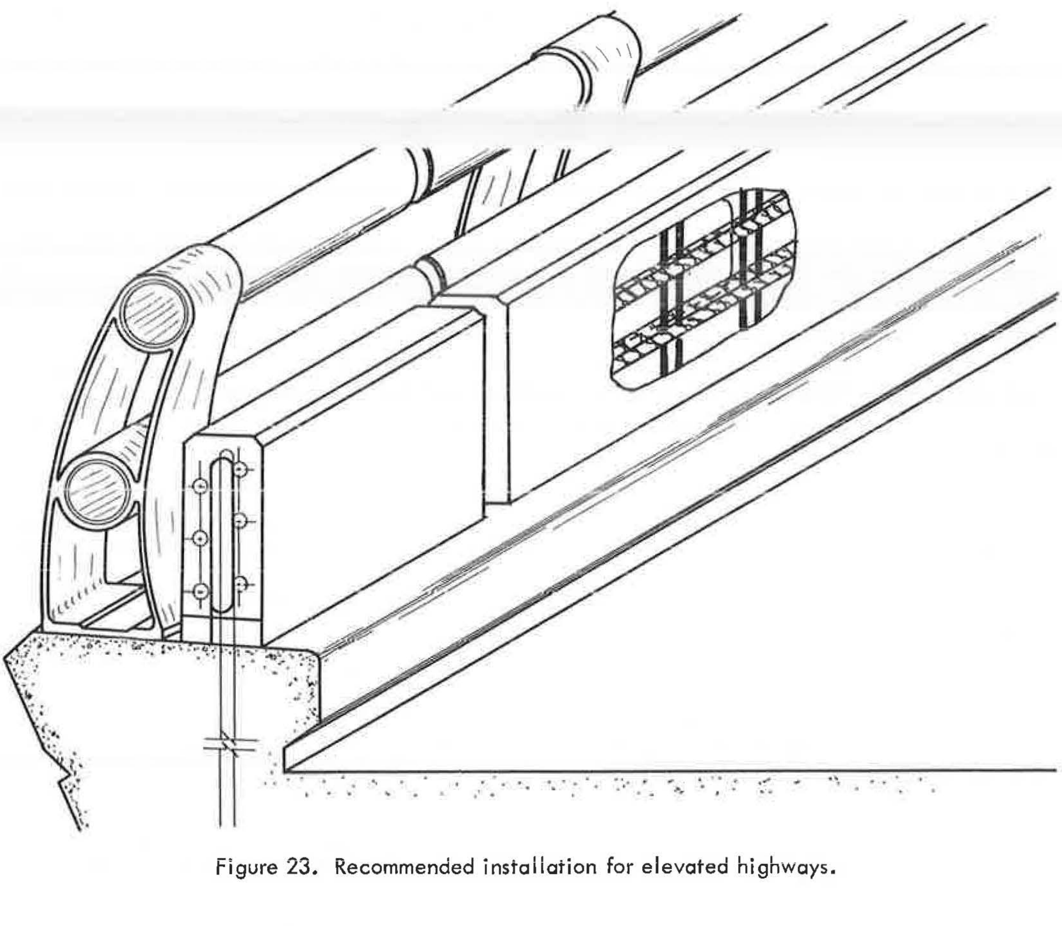


Figure 23. Recommended installation for elevated highways.

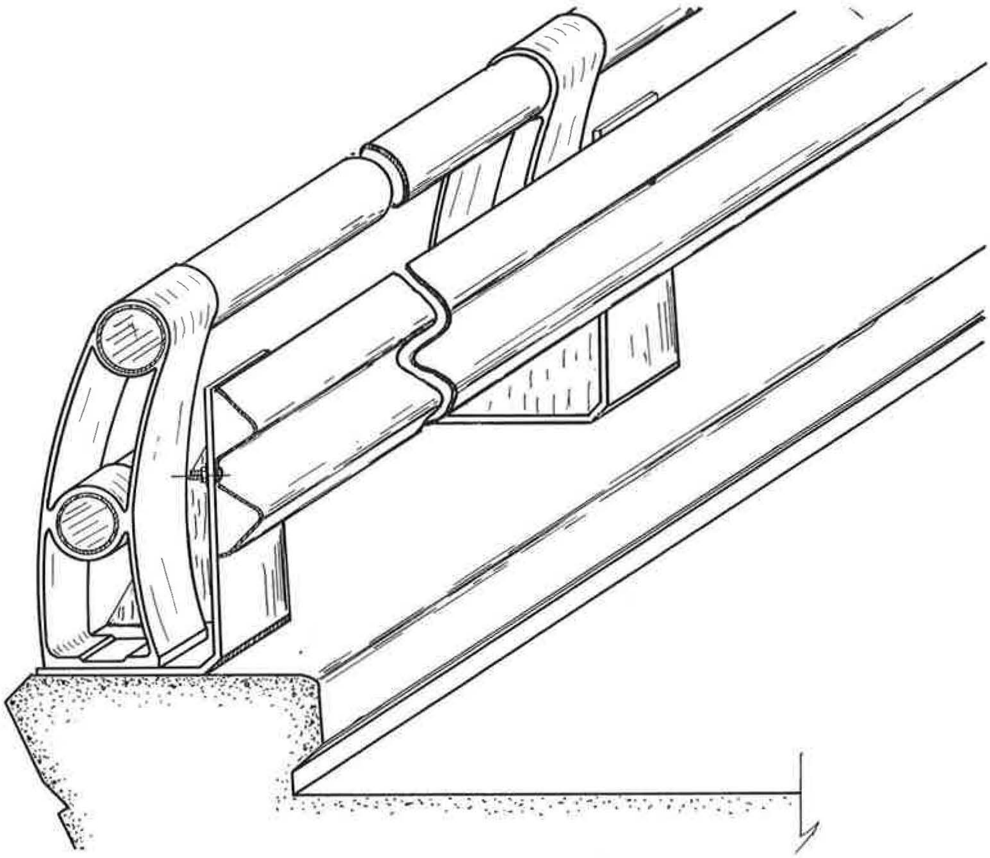


Figure 24. Recommended installation for ramps.

been a substantial change in the number of fatalities and the seriousness of injuries, as shown in Tables 1 and 2.

The performance of the concrete guide rail has been exactly as expected. Even at more than 60 mph the main results sought have been obtained, namely, very small rebound and low deceleration. This design of guide rail is now standard for elevated expressways in the Province of Quebec.

Study of Freeway Access Violations

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•A FREEWAY is a divided arterial highway for through traffic, with full control of access and grade separations at all crossings; but a motorist defines a freeway by the quality of service it provides rather than its physical characteristics. The motorist views the freeway as a superhighway which eliminates annoyances, hazards of left turns, blind intersections, dangerous curves and distractions close to the highway. Freeways are expected to be a motorists' highway with motorists' needs anticipated and fulfilled to a much higher degree than on conventional highways. Experience has shown, however, that it is not enough to merely build freeways. To deliver the promised safety, comfort, and convenience, freeways must have a high degree of operational attention. Control of access, one feature of freeway design, implies that the rights to light, air, view, and access are controlled by public authority. This feature provides a fundamental change in the concept of modern highways. There were indications, however, that the access control feature of freeway design was being violated and that additional controls may be required to insure access control.

The general objectives of this project were to determine the extent and causes of access violations on controlled access facilities, and to provide data that would be useful in controlling existing access violations and in anticipating and eliminating future violations. The specific objectives of this study were: (a) to catalog the types of ac- [illegible] of access violations; and (c) to determine the effectiveness of various design and control features presently being utilized to prevent access violations.

STUDY PROCEDURE

Data Collection

To determine the extent of access violations, it was decided to collect data on controlled access facilities across Texas. Data were collected on approximately 770 mi of freeway which included all Interstate Highways within the state. The locations from which data were actually collected are shown in Figure 1. Since the data collection was to be accomplished on a statewide basis, it was determined that the Texas Highway Department would request each district's maintenance personnel to collect the data using a standard data collection form. Although district maintenance personnel were requested to complete the data collection form, in many districts the traffic engineering personnel completed or supervised the completion of the data collection forms.

The "Shoulder and Rest Area Use Procedure Guide" (1) was helpful in making a data collection form. A first form was made and evaluated in the field to determine its shortcomings. The necessary changes were made and the form was again reviewed and cleared for statewide data collection. Material in addition to the data collection forms sent to each Highway Department district were: an extract of the project statement, data collection procedure sheets, and two completed sample data collection forms. The extract from the project statement explained the specific aim of the investigation,

Paper sponsored by Committee on Operational Effects of Geometrics and presented at the 45th Annual Meeting.

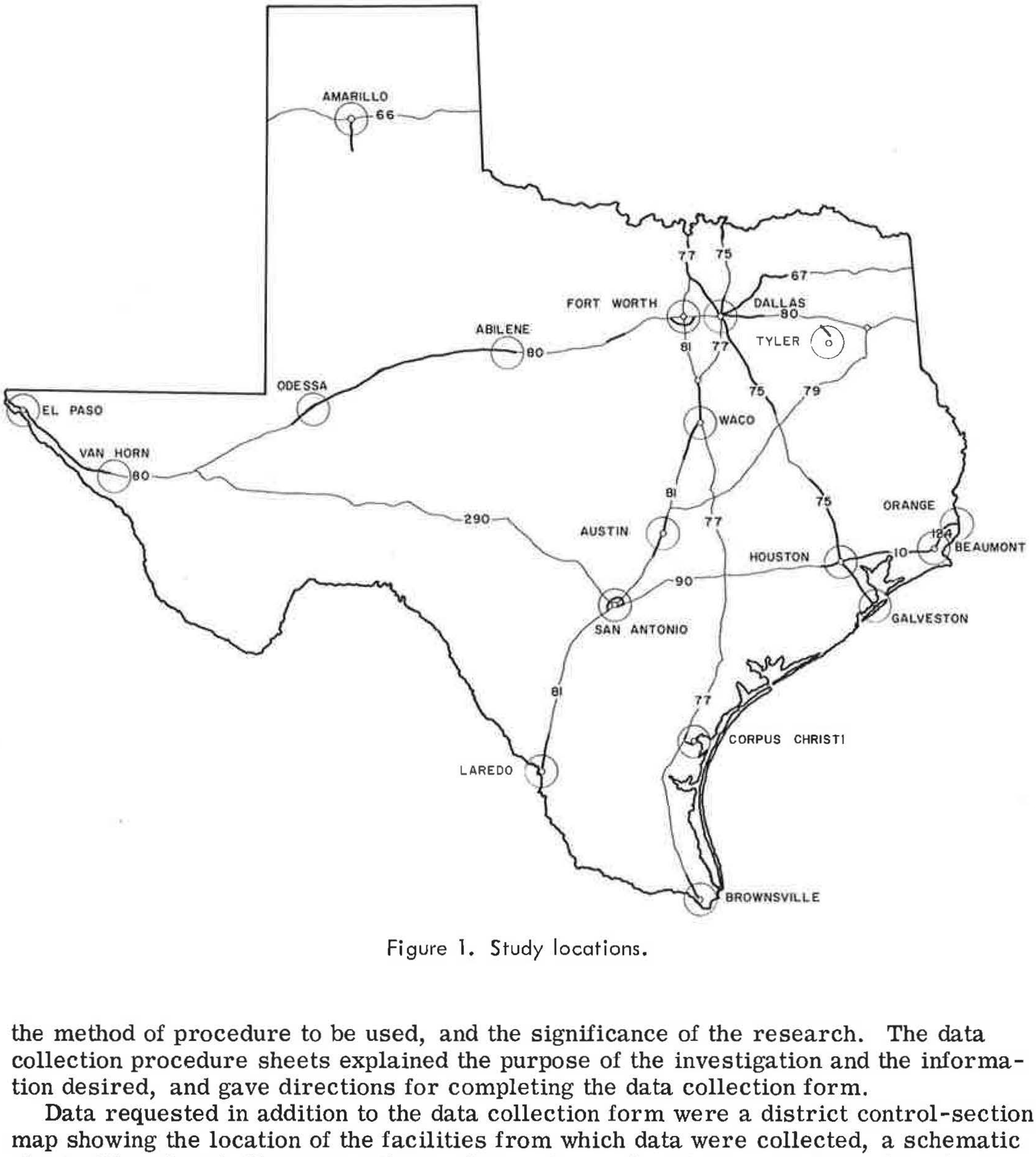


Figure 1. Study locations.

the method of procedure to be used, and the significance of the research. The data collection procedure sheets explained the purpose of the investigation and the information desired, and gave directions for completing the data collection form.

Data requested in addition to the data collection form were a district control-section map showing the location of the facilities from which data were collected, a schematic sheet with a sketch illustrating the conditions for each violation, photographs, if possible, and any pertinent information not covered in the form. The data collection personnel had 4 methods of determining access violations: (a) to see a violation actually take place; (b) to see the tracks of violations that had occurred previously; (c) to remember violations that had been seen on other occasions; and (d) to note past violations which have been eliminated by corrective measures.

Data collection was accomplished on a one-time basis by all districts during the month of March or April 1964. This means that the personnel drove through the facility one time only, completing the data collection forms and taking photographs. Aerial photographs were made of the major types of violations and some of the causes of violations for inclusion in this report.

Analysis of Data

After receiving the data, the forms were checked against the sketch of the violation to insure that each form had been correctly completed. Any errors in the data collec-

1 VIOLATION NUMBER: ____________________
District ____________________ County ____________________
Highway Number ____________________ Between ____________ And ____________
Control ____________________ Section ____________________
Station (approximate) ____________ Urban ____________ Rural ____________
Location with respect to nearest access (Interchange, Grade Separation or nearby road): ____________________

____

____

____

2. DATE: ____________________
3. INSPECTOR: ____________________
4. VIOLATOR: (Check appropriate block)
( ) Pedestrian ( ) Animal
( ) Vehicle ( ) Other: ____________
(Note any specific group of violators such as telephone or power companies, school children, etc.)

____

____

____

____

5. TYPE OF VIOLATION: (Check appropriate block or blocks)
( ) Median Crossing ( ) Incorrect Use of Entrance Ramp
( ) Separation Strip Crossing ( ) Incorrect Use of Exit Ramp
( ) Nose Crossing ( ) Entrance Where No Entrance Ramp Exists
( ) Crossing Entire Freeway System ( ) Exit Where No Exit Ramp Exists
( ) Unattended Vehicle on Shoulder ( ) Wrong Way on Frontage Road
( ) Parking on Median ( ) Animal Crossing
( ) Hitch-Hiking ( ) Loading or Unloading Passengers
( ) Other: ____________________

____

6. FREQUENCY OF VIOLATION: (Estimate the number of violations per week if possible:
( ) Very Often ( ) Occasionally
( ) Often ( ) Seldom

7. PURPOSE OF VIOLATION:
( ) "U" Turn ( ) Access to Home, Farm, or Business
( ) Leisure Stop ( ) Access from Home, Farm, or Business
( ) Business Stop ( ) Access to or from New Development
( ) Emergency Stop ( ) Other: ____________

8. CAUSE OF VIOLATION:
( ) No Ramp ( ) Most Convenient Route
( ) No Grade Separation ( ) Frontage Road Ends
( ) No Frontage Road ( ) End of Corrective Measure
( ) Other: ____________________

____

(Also describe and show on skematic sheet geometric factors contributing to the violation in addition to those above. Describe what proper route if any is available to traffic and estimate the additional time and distance required. Use the dashed lines to show the proper route on the skematic sheet.)

____

____

____

____

____

ADDITIONAL TIME ____________________ ADDITIONAL DISTANCE ____________________

Figure 2. Data collection form for analysis of access violations on controlled access facilities.

9. DURATION OF VIOLATION: (Estimate the time required to complete the violation maneuver.)

(___) less than 5 minutes
(___) 5 to 15 minutes
(___) 15 minutes to 1 hour
(___) 1 hour to 5 hours
(___) over 5 hours
(___) variable

10. SEVERITY OF VIOLATION:

(___) Very Dangerous (___) Dangerous (___) Relatively Safe

11. PRESENCE OF VIOLATIONS: (Estimate how long the violation has existed and note if it is of a temporary nature due to roadside construction, etc)

(___) Since highway opened in ______________________
(___) Temporary for ______________________ months
(___) Other: ______________________
(___) Since corrective measure was placed

12. CORRECTIVE MEASURES: (Describe the effectiveness of corrective measures used in the past.)

(___) Signs	(___) Effective	(___) Ineffective
(___) Posts with Barrier Cable	(___) Effective	(___) Ineffective
(___) Ditches	(___) Effective	(___) Ineffective
(___) Curbs	(___) Effective	(___) Ineffective
(___) Chain Line Fences	(___) Effective	(___) Ineffective
(___) Guard Fences	(___) Effective	(___) Ineffective
(___) None	(___) Effective	(___) Ineffective
(___) Guard Posts	(___) Effective	(___) Ineffective
(___) Other: ______________	(___) Effective	(___) Ineffective

____

____

(Describe any suggested or anticipated measures for elimination of this violation.)

____

____

____

13. Has this violation been eliminated? Yes (___) No (___)

14. ENFORCEMENT: (Rate enforcement level in this vicinity.)

(___) High (___) Low (___) Medium

15. ACCIDENT HISTORY: (Describe and sketch on plan or skematic sheet any accidents at this point which were the result of this violation.)

(___) None Reported
(___) History: ______________________

____

____

____

____

____

____

16. SKETCH: (Illustrate the conditions described in items 5, 8, and 12 above on approximately 1" = 200' skematic sheet. Ground photos should be provided when justified by the severity of the violation.)

Note profile of violation area as:

(___) Relatively Flat
(___) Other: (Sketch cross section below)

17. VOLUME: (Give average daily traffic on controlled access facility.)

ADT = ______________________

Figure 2. (Continued).

tion form which could be determined were corrected. This review revealed that some changes should be made in the form. Many of the questionnaires had blocks with "other" checked and the same comment entered. These forms were altered to add separate blocks for these comments before the data were removed. The revised form which included these alterations is shown in Figure 2.

The data were then punched into IBM cards with the coding shown in Table 1 and sorted on the desired columns using an IBM sorting machine. Next, the contents of the sorted cards were printed on paper using the IBM 407 accounting machine. This machine printed a list of the card contents sorted on a certain column and a count of the

TABLE 1

CODE OF DATA COLLECTION FORM

Col.	1-2	Last two digits of TTI project number 65 = Project 1065
Col.	3-4	Number of month data was collected 3 = March 4 = April
Col.	5	Last digit of year data was collected 4 = 1964
Col.	7-8	Texas Highway Department district number
Col.	10-16	Violation number for each district (letters preceding numbers indicate that the district had more than one number 1).
Col.	18	Urban or rural Urban = 1 Rural = 3
Col.	20-22	Location to nearest access in tenths of a mile Blank = Unknown or does not apply
Col.	24	Violator 1 = Pedestrian 3 = Animal 2 = Vehicle 4 = Other
Col.	25	Specific group of violator 1 = No specific group 2 = Specific group
Col.	27-30	Type of violation (up to four may be checked for one violation) A = Median crossing B = Separation strip crossing C = Nose crossing D = Crossing entire freeway system E = Unattended vehicle on shoulder F = Parking on median G = Hitch-hiking H = Other I = Incorrect use of entrance ramp J = Incorrect use of exit ramp K = Entrance where no entrance ramp exists L = Exit where no exit ramp exists M = Wrong way on frontage road N = Animal crossing O = Loading or unloading passengers
Col.	34	Purpose of violation 1 = U turn 2 = Leisure stop 3 = Business stop 4 = Emergency 5 = Access to home, farm, or business 6 = Access from home, farm, or business 7 = Access to or from new development 8 = Other
Col.	36-37	Cause of violation (up to two may be checked for one violation) 1 = No ramp 2 = No grade separation 3 = No frontage road 4 = Other 5 = Most convenient route 6 = Frontage road ends 7 = End of corrective measure
Col.	39-40	Additional time in minutes required for legal route Zeros = No additional time required Blank = Does not apply
Col.	42-44	Additional distance to tenths of a mile required for legal route Zeros = No additional distance required Blank = Does not apply
Col.	46	Duration of violation 1 = Less than 5 minutes 4 = 1 hour to 5 hours 2 = 5 to 15 minutes 5 = Over 5 hours 3 = 15 minutes to 1 hour 6 = Variable
Col.	48	Severity of violation 1 = Very dangerous 2 = Dangerous 3 = Relatively safe
Col.	50	Presence of violation 1 = Since highway opened. . . 3 = Other 2 = Temporary 4 = Since corrective measure was placed
Col.	52 55 58	Corrective measures 0 = Unknown 1 = Signs 2 = Posts with barrier cable 3 = Ditches 4 = Curbs 5 = Chain link fences 6 = Guard fences 7 = None 8 = Other 9 = Guard posts
Col.	53 56 59	Effectiveness of corrective measures 0 = Unknown 1 = Effective 2 = Ineffective
Col.	64	Suggested corrective control measures 1 = No suggestions 2 = Suggested control measures
Col.	66	Violation eliminated or still in existence 1 = Violation still in existence 2 = Violation eliminated
Col.	68	Enforcement level 1 = High 2 = Low 3 = Medium
Col.	70	Accident history 1 = None reported 2 = History
Col.	72	Profile 1 = Relatively flat 2 = Other
Col.	74-78	Average daily traffic Blanks = No data submitted
Col.	80	Ground photo 1 = No photo 2 = Fair photo 3 = Good photo

number of cards printed. Thus, a permanent record of the sorting process was achieved for use in graphically illustrating the project results.

RESULTS

Types of Access Violations

The first objective of this project was to catalog the types of access violations occurring on controlled access facilities. Access violations were cataloged into types of violations as determined by the path or route of the violator. Each type of violation described the freeway areas crossed during the violation maneuver and the violator's direction of travel. Figure 3 defines the freeway areas as used in naming the types of violations. An example of one type of violation was a "separation strip crossing, exit where no exit ramp exists," which means that the separation strip was crossed in making an illegal departure from the freeway facility.

For a better understanding, many of the types of violations are illustrated in Figures 4 through 7. Types of violations in addition to those illustrated were an unattended vehicle on the shoulder, parking on the median, hitch-hiking, animal crossing, loading and unloading passengers, and the general group catalogs as "other." The classification "other" was used for violations which did not have enough occurrences to be considered as an individual type of violation. A list of all of the types of violations is given in Table 2. Aerial photographs of some types of access violations existing in June 1964 are shown in Figures 8, 9, and 10. While taking the aerial photographs, the photographer noted a blanket salesman selling his goods within the Interstate right-of-way (Fig. 9).

Extent and Causes of Access Violations

The extent or frequency of the types of access violation locations are given in Table 2, which gives both the number of violation locations and percent of all violation locations. The total number of violation locations reported was 986 making the percent for each type roughly one-tenth of the number of violation locations. The 986 violation locations occurred over approximately 770 mi of freeway, a ratio of 1.3 access violation locations per mile of freeway. Twenty-five percent of these violation locations occurred on the 130 mi of urban freeway studied, a ratio of 1.9 access violation locations per mile of urban freeway. The remainder of the violation locations occurred on 640 mi of rural freeway, a ratio of 0.85 access violation locations per mile of rural freeway.

Although there were 28 different types of violations, 5 types were predominant, which accounted for 63.2 percent of the violation locations reported. The predominant or major types of access violations were:

1. Separation strip crossing, exit where no exit ramp exists;
2. Median crossing;
3. Separation strip crossing, entrance where no entrance ramp exists;
4. Unattended vehicle on shoulder; and
5. Crossing entire freeway system.

The frequency of these types of violations and their respective percentages are shown in Figure 11. The type of violation, loading or unloading passengers, was not marked as one occurring in the state, yet it was included because the author noted and photographed this taking place on a Houston freeway. It was believed that a continuous surveillance method of data collection, rather than the once-over method used, would indicate the frequency of this type of violation (Fig. 10).

The number of violation locations shown in Table 2 does not take into account how often each violation was repeated. These data were required to determine the true extent of access violations. Since data were collected on a one-time basis, the frequency of each violation was estimated by the personnel completing the questionnaire in the general terms of seldom, occasionally, often or very often. This estimation is shown in Figure 12. Noting that "often" was marked for 44 percent of the violation locations

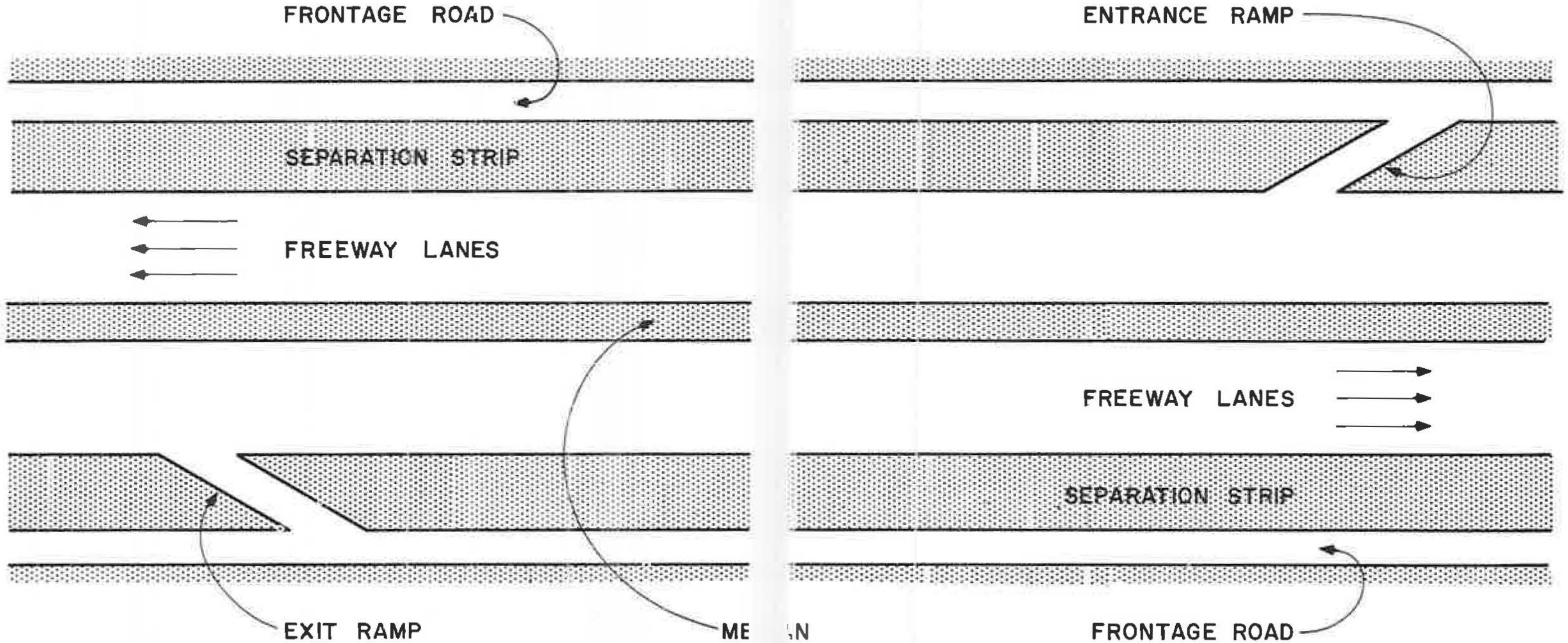


Figure 3 reeway definitions.

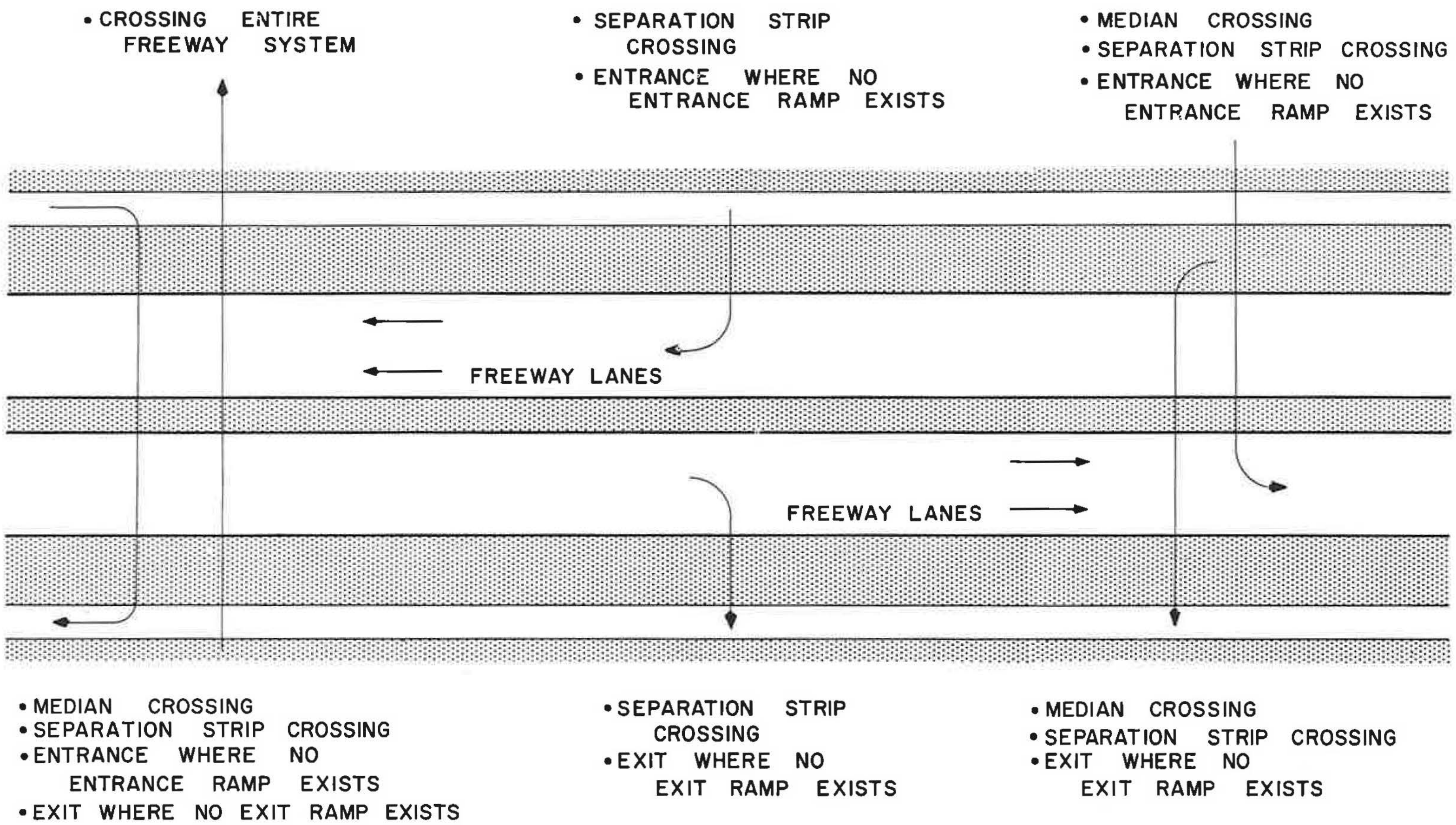


Figure 4. Types of access violations.

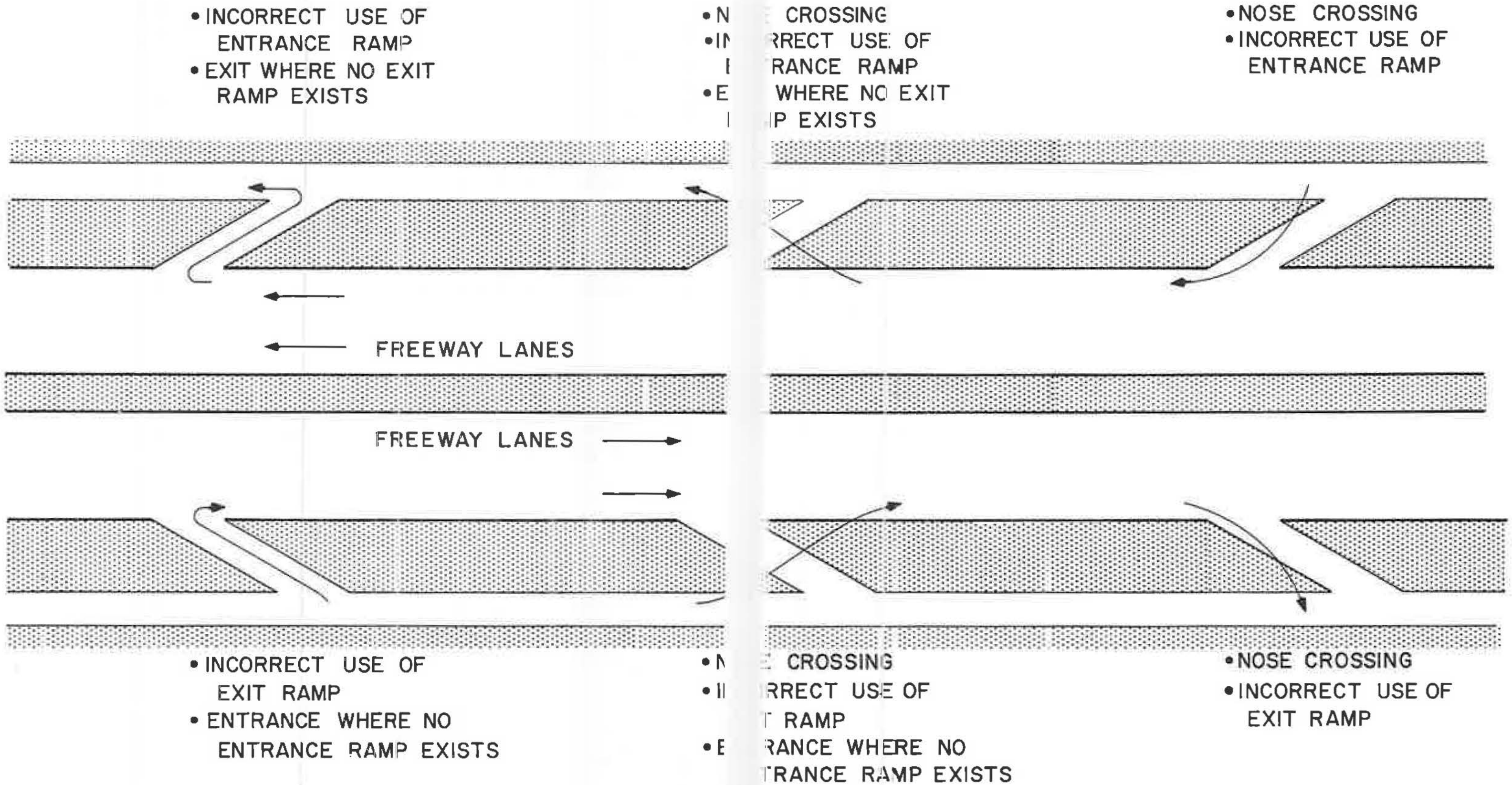


Figure 5. Ty of access violations.

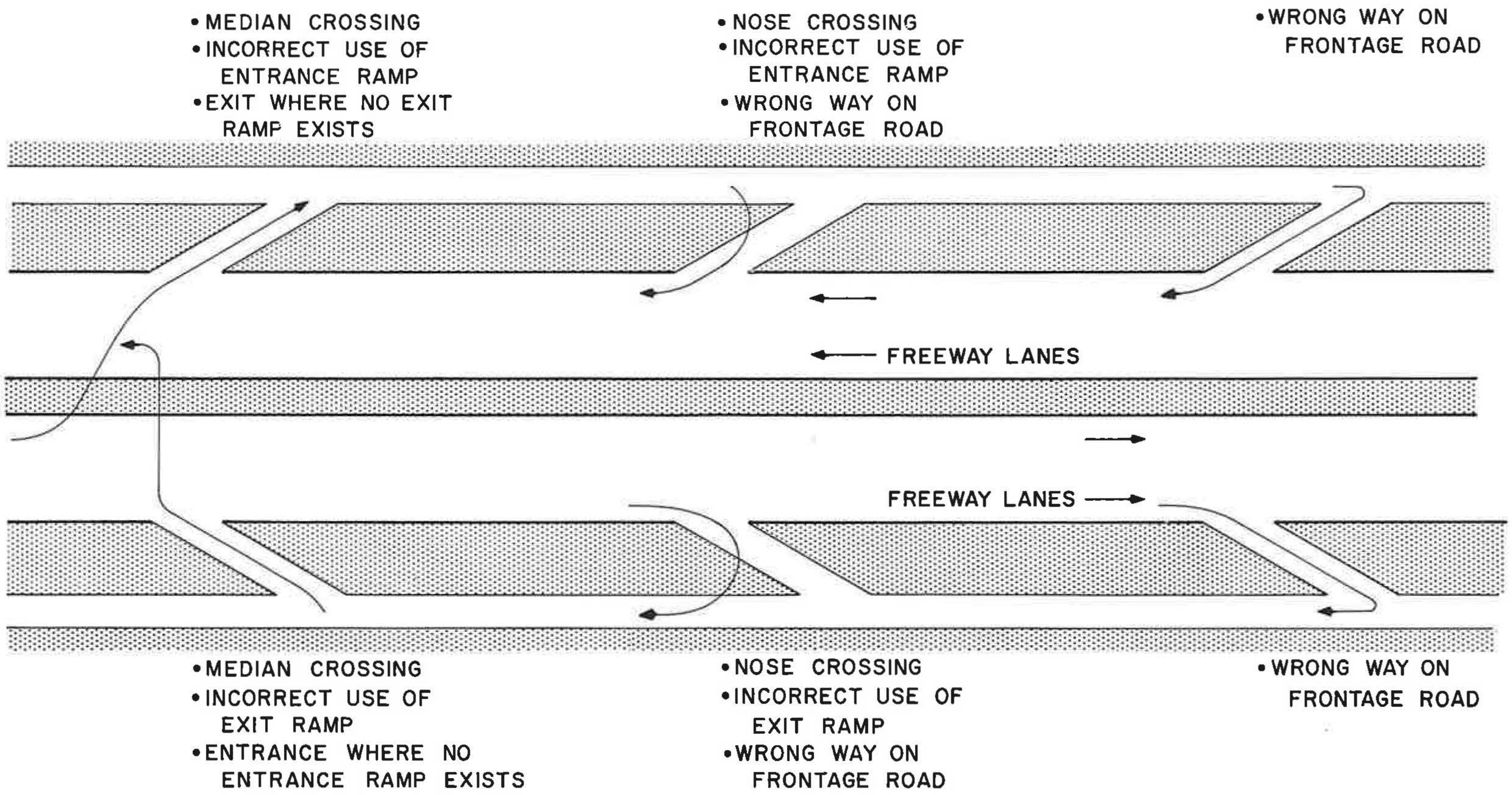


Figure 6. Types of access violations.

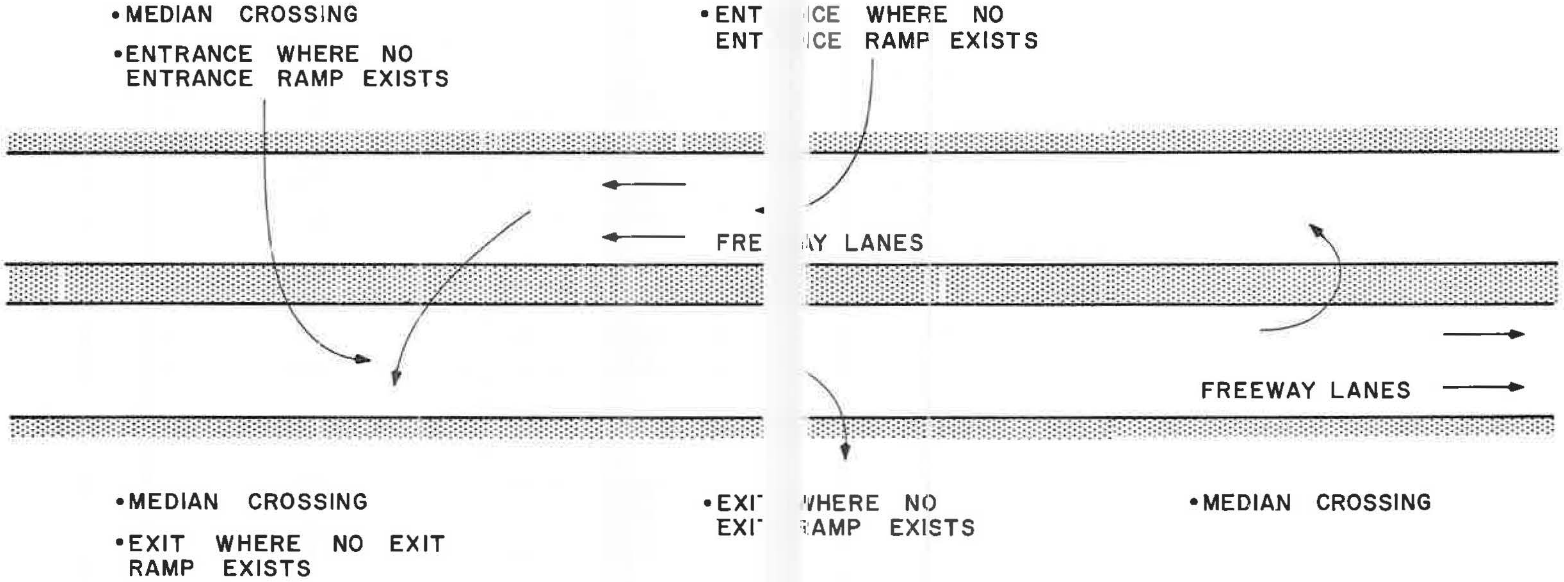
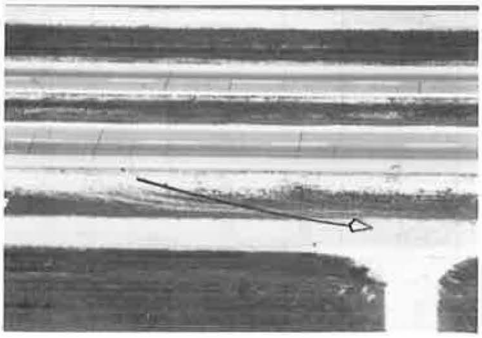


Figure 7. Typ f access violations.

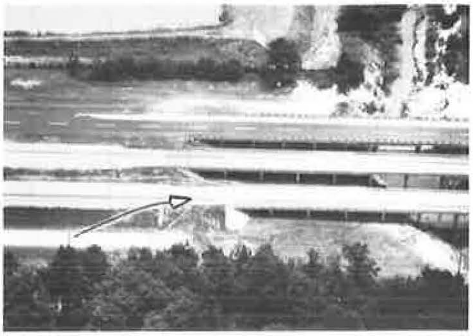
TABLE 2

TABLE OF THE FREQUENCY OF TYPES OF VIOLATION LOCATIONS

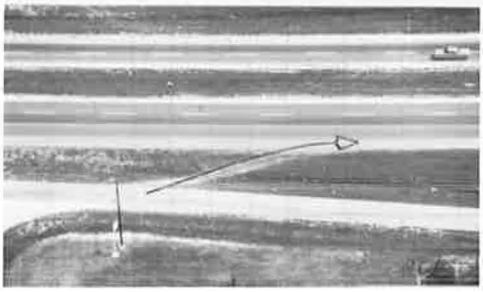
TYPE OF VIOLATION	NUMBER OF VIOLATION LOCATIONS	PERCENT OF VIOLATION LOCATIONS	TYPE OF VIOLATION	NUMBER OF VIOLATION LOCATIONS	PERCENT OF VIOLATION LOCATIONS
MEDIAN CROSSING	180	18.2	CROSSING ENTIRE FREEWAY SYSTEM	58	5.9
MEDIAN CROSSING SEPARATION STRIP CROSSING ENTRANCE WHERE NO ENTRANCE RAMP EXISTS	35	3.6	MEDIAN CROSSING INCORRECT USE OF ENTRANCE RAMP EXIT WHERE NO EXIT RAMP EXISTS	9	0.9
MEDIAN CROSSING SEPARATION STRIP CROSSING EXIT WHERE NO EXIT RAMP EXISTS	48	4.9	MEDIAN CROSSING INCORRECT USE OF EXIT RAMP ENTRANCE WHERE NO ENTRANCE RAMP EXISTS	4	0.4
NOSE CROSSING INCORRECT USE OF ENTRANCE RAMP WRONG WAY ON FRONTAGE ROAD	3	0.3	NOSE CROSSING INCORRECT USE OF ENTRANCE RAMP EXIT WHERE NO EXIT RAMP EXISTS	40	4.1
NOSE CROSSING INCORRECT USE OF EXIT RAMP WRONG WAY ON FRONTAGE ROAD	5	0.5	NOSE CROSSING INCORRECT USE OF EXIT RAMP ENTRANCE WHERE NO ENTRANCE RAMP EXISTS	37	3.8
MEDIAN CROSSING ENTRANCE WHERE NO ENTRANCE RAMP EXISTS	5	0.5	NOSE CROSSING INCORRECT USE OF ENTRANCE RAMP	8	0.8
MEDIAN CROSSING EXIT WHERE NO EXIT RAMP EXISTS	8	0.8	NOSE CROSSING INCORRECT USE OF EXIT RAMP	12	1.2
SEPARATION STRIP CROSSING EXIT WHERE NO EXIT RAMP EXISTS	204	20.7	INCORRECT USE OF ENTRANCE RAMP EXIT WHERE NO EXIT RAMP EXISTS	30	3.0
SEPARATION STRIP CROSSING ENTRANCE WHERE NO ENTRANCE RAMP EXISTS	112	11.4	INCORRECT USE OF EXIT RAMP ENTRANCE WHERE NO ENTRANCE RAMP EXISTS	20	2.0
UNATTENDED VEHICLE ON SHOULDER	68	7.0	MEDIAN CROSSING SEPARATION STRIP CROSSING ENTRANCE WHERE NO ENTRANCE RAMP EXISTS EXIT WHERE NO EXIT RAMP EXISTS	10	1.0
PARKING ON MEDIAN	6	0.6			
HITCH-HIKING	2	0.2			
ENTRANCE WHERE NO ENTRANCE RAMP EXISTS	21	2.1	ANIMAL CROSSING	2	0.2
EXIT WHERE NO EXIT RAMP EXISTS	20	2.0	LOADING OR UNLOADING PASSENGERS	0	0.0
WRONG WAY ON FRONTAGE ROAD	19	1.9	OTHER	22	2.2
			TOTAL	986	100.0



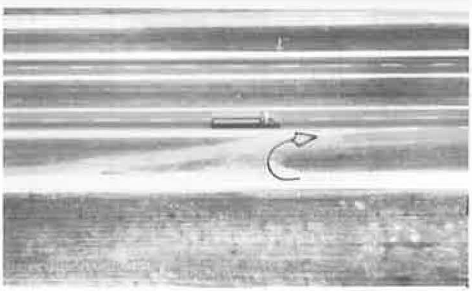
- SEPARATION STRIP CROSSING
- EXIT WHERE NO EXIT RAMP EXISTS.



- SEPARATION STRIP CROSSING
- ENTRANCE WHERE NO RAMP EXISTS.



- SEPARATION STRIP CROSSING
- ENTRANCE WHERE NO RAMP EXISTS.

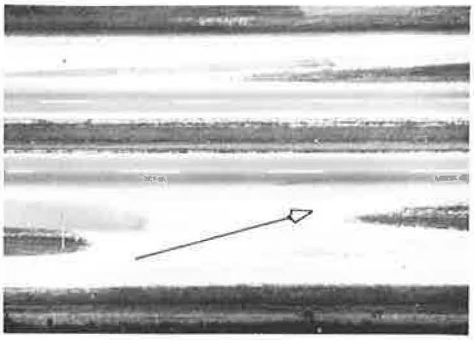


- NOSE CROSSING
- INCORRECT USE OF ENTRANCE RAMP.

Figure 8. Types of access violations.

- UNATTENDED VEHICLE ON SHOULDER.

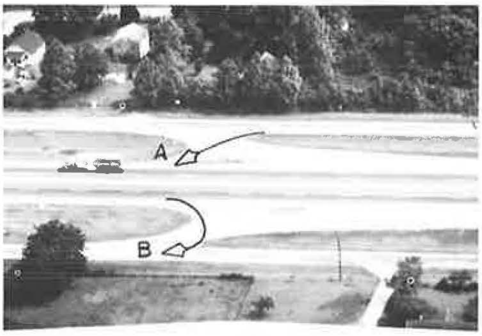
- LOADING OR UNLOADING PASSENGER.



- NOSE CROSSING
- INCORRECT USE OF EXIT RAMP.
- ENTRANCE WHERE NO ENTRANCE RAMP EXISTS.

- BLANKET SALES ON RIGHT OF WAY.

Figure 9. Types of access violations.

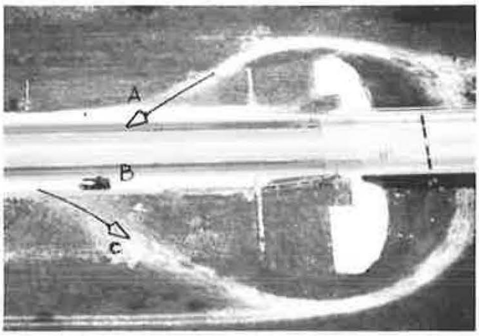


A

- NOSE CROSSING
- INCORRECT USE OF EXIT RAMP
- ENTRANCE WHERE NO ENTRANCE RAMP EXISTS.

B

- NOSE CROSSING
- INCORRECT USE OF ENTRANCE [illegible]
- EXIT WHERE NO EXIT RAMP EXISTS.



A

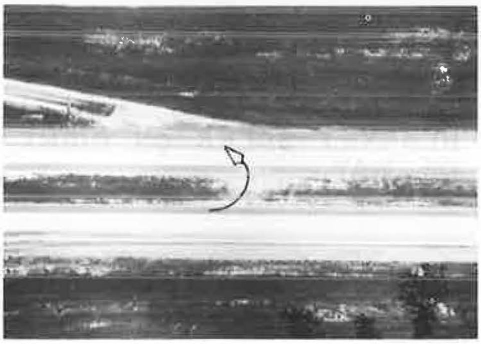
- ENTRANCE WHERE NO ENTRANCE RAMP EXISTS.

B

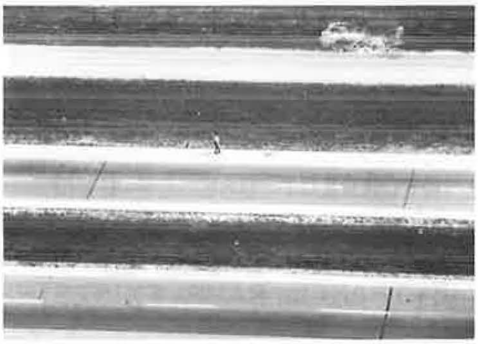
- UNATTENDED VEHICLE ON SHOULDER.

C

- EXIT WHERE NO EXIT RAMP [illegible]



- MEDIAN CROSSING



- HITCH-HIKING

Figure 10. Types of access violations.

and that "very often" was marked for 24 percent of the violation locations, it may be assumed that the true extent of access violations was several times greater than the total number of the types of violation locations reported (986).

The primary cause of access violations was that the violation route was the most convenient route. This generally resulted from one of two conditions: there was no ramp available, or there was no grade separation available.

Figure 13 shows the frequency of the cause of access violations. Since 2 causes could be marked on the questionnaire for each violation, the sum of the percents for all causes was greater than 100.

The greatest cause of violations was that the violation route was the most convenient route. This cause of violation was indicated for over 52 percent of the violation locations.

Figure 14 shows that 35 percent of the violation locations with most convenient route marked as a cause, required no additional distance to go the legal route. (No additional distance to go the legal route means that the violator could have exited back down the freeway, driven the remainder of the distance on the frontage road, and traveled the same distance as was traveled in the route with the violation.)

Seventy percent of the violation locations required an additional distance of one mile or less to go the legal route. This seemed to leave the freeway designer with little opportunity to design freeways to eliminate this cause of violation, since only 30 percent of the violation locations could be eliminated with ramps, interchanges, etc.,

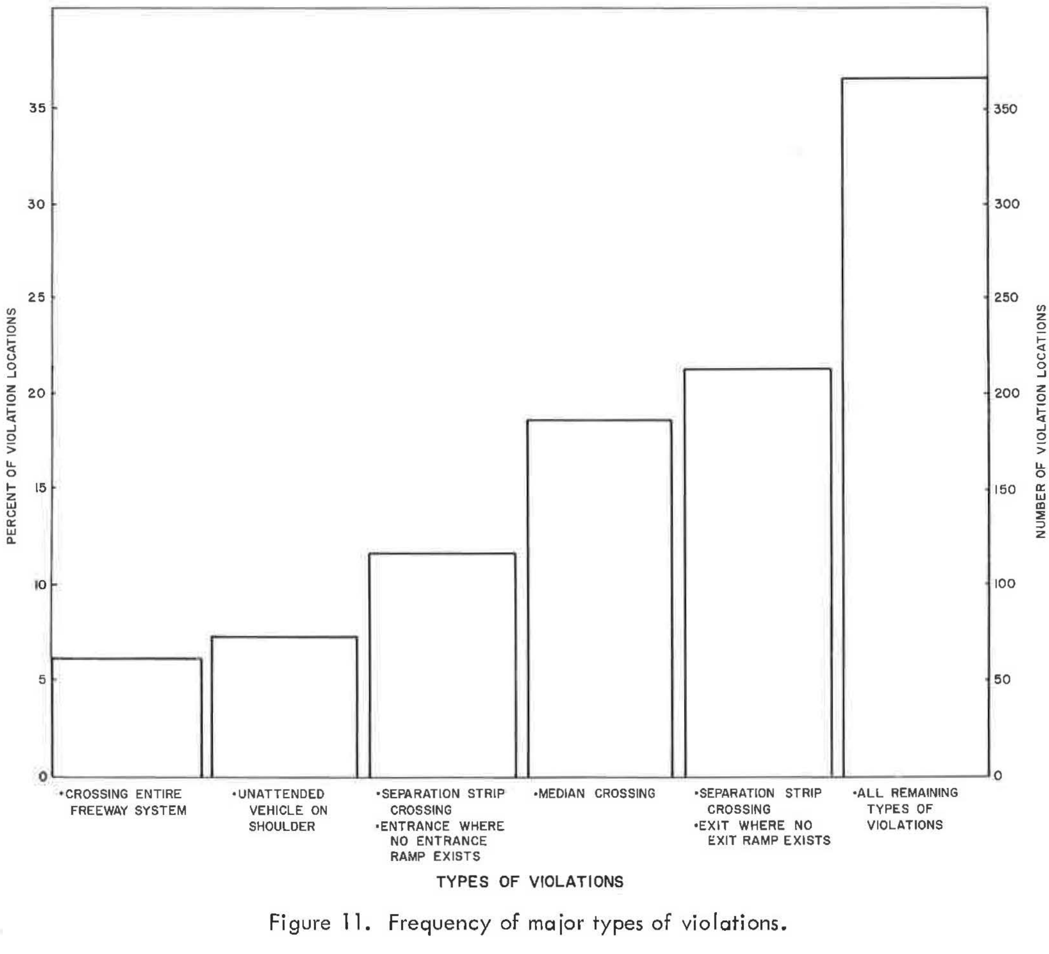


Figure 11. Frequency of major types of violations.

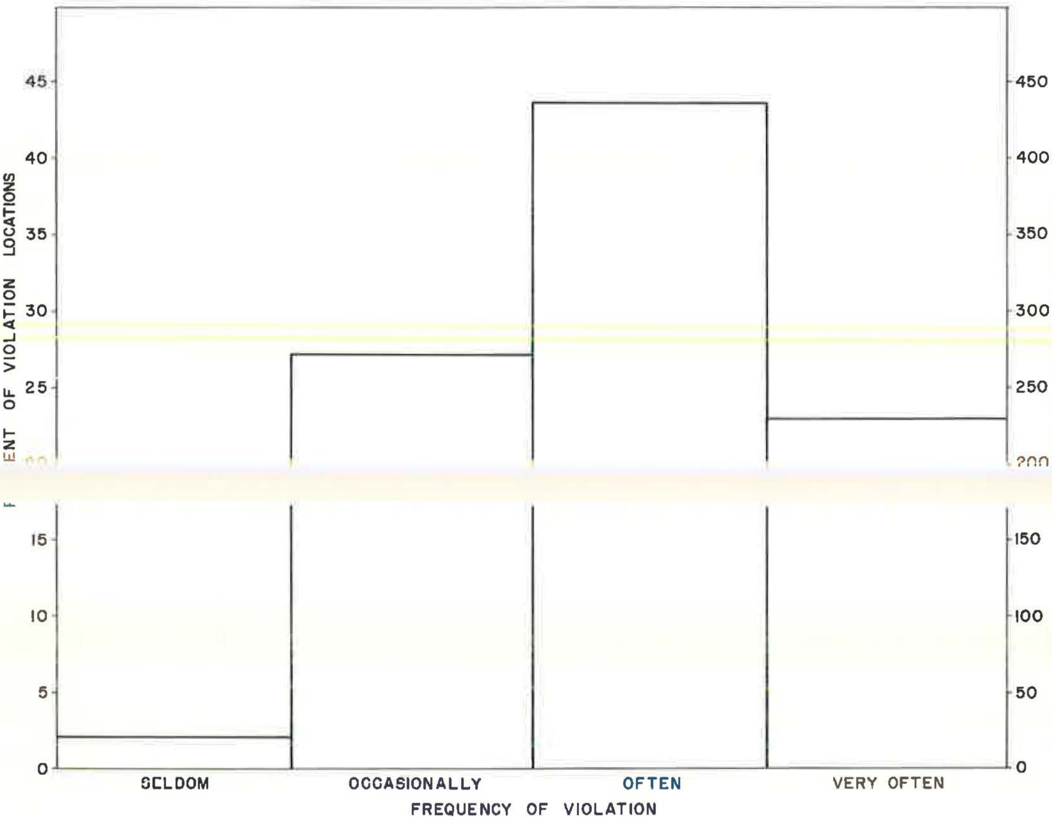


Figure 12. Extent of the frequency of violations.

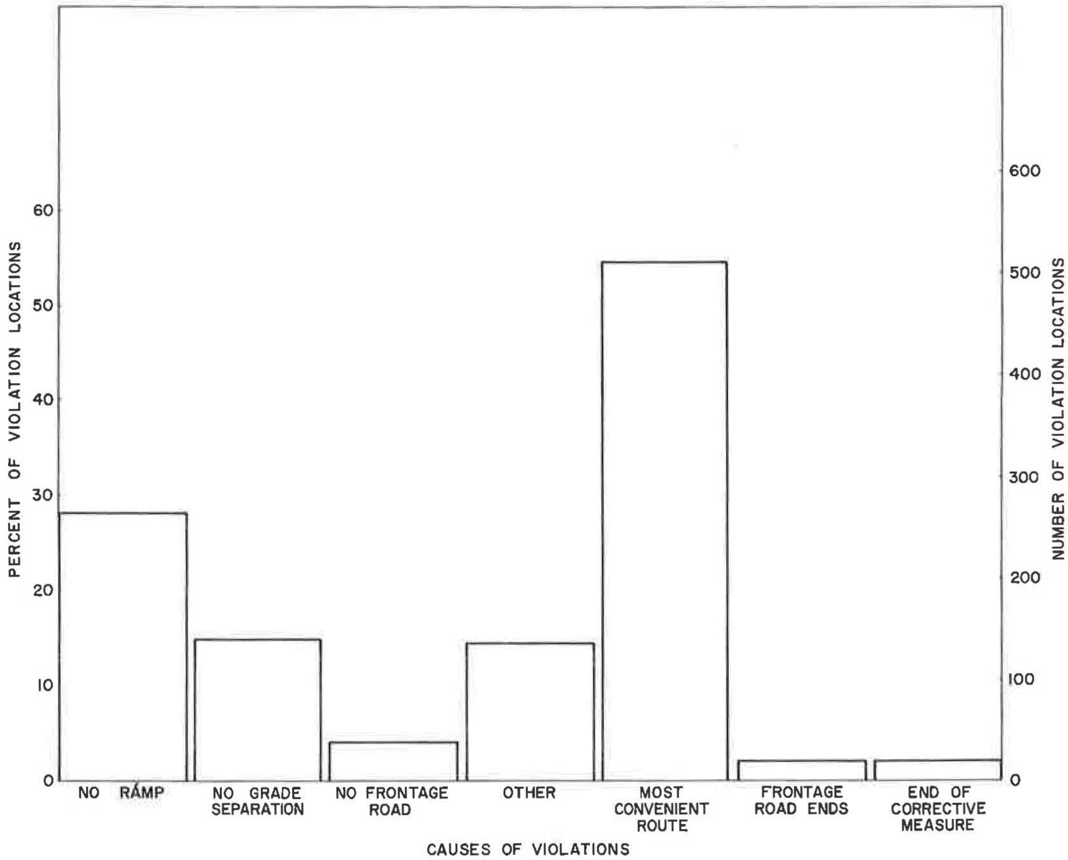


Figure 13. Frequency of causes of violations.

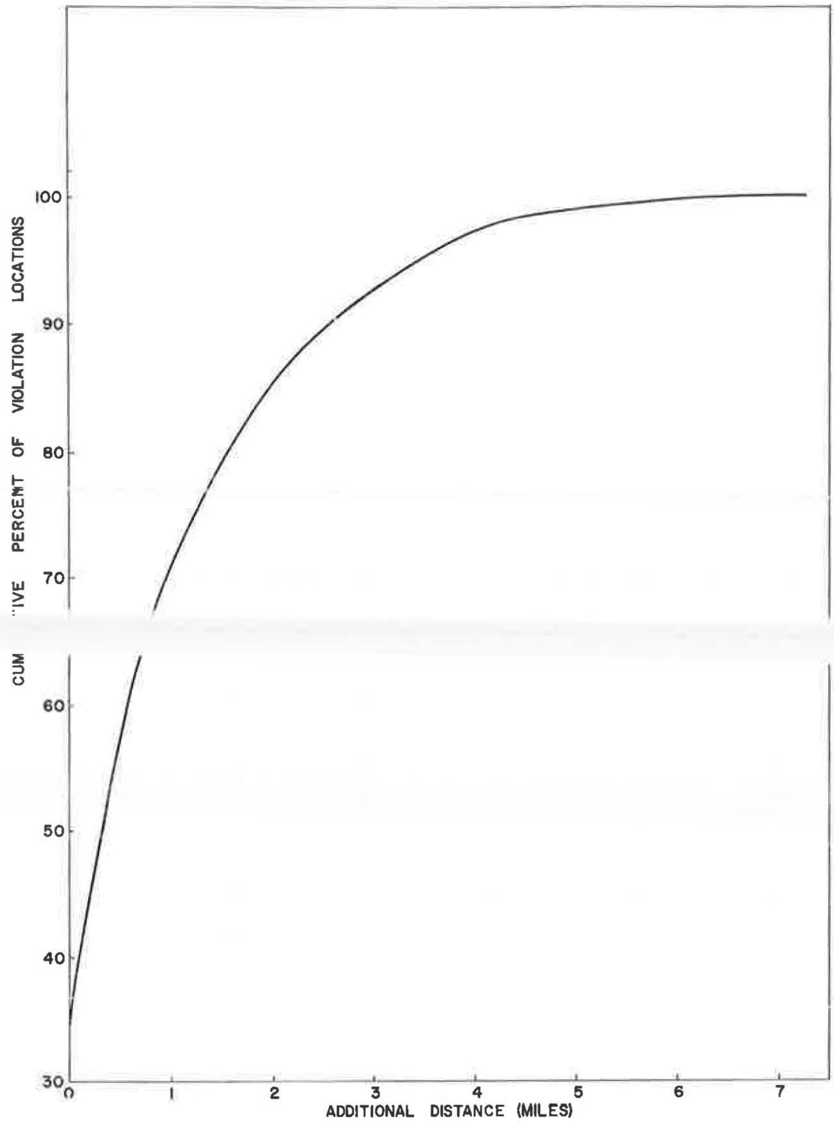


Figure 14. Relationship between additional distance and cause, most convenient route.

spaced at one-mile intervals. At best, the designer would only be able to design corrective measures to enforce the elimination of violation locations for this cause.

Effectiveness of Corrective Measures

The frequency of the use of the different types of corrective measures was determined from the opinions of the field personnel as found in the data collection forms. These frequencies are plotted as bar graphs in Figure 15. The sum of the percent for the bars did not equal 100 percent since up to 3 corrective measures could be marked for one violation.

Determining the effectiveness of these corrective measures was the third objective of this project. Figure 16 shows the percentage that each corrective measure was rated as effective and ineffective. The corrective measure signs were ineffective more often than effective, 78 percent vs 22 percent. Curbs, chain-link fences, and posts with barrier cable showed a very high effectiveness ratio. The sample size for curbs and chain link fences was very small (Fig. 15).

ADDITIONAL CONSIDERATIONS

Types of Access Violators

The access violator was cataloged into 3 types: pedestrian, vehicle, and animal since the type "other" was not marked on any questionnaire. Figure 17 shows the frequency of each of the types of violators with the vehicle accounting for 94 percent. Figure 18 shows that in 10 percent of the violation locations, a specific group was involved. These groups included school children, power companies, telephone companies, roadside advertising companies, and particular business firms. There was a possibility that these violation locations could be eliminated by contacting these groups, pointing out the proper route, noting the severity of the violation, and requesting their help in eliminating the violation. This procedure was effective in eliminating one violation in the Ft. Worth area.

Purposes of Access Violations

Figure 19 shows the major purposes for access violations to be (a) egress from the freeway facility, (b) access to the freeway facility, and (c) a change of direction on the freeway facility. These purposes substantiated the 3 major types of violations which are shown in Figure 11.

Average Daily Traffic

Figure 20 shows that 46 percent of the violation locations occurred on facilities with an average daily traffic from 5,000 to 9,999. The sum of percentages of the bar graphs did not equal 100 percent because 163 (16.5%) of the questionnaires did not furnish a figure for average daily traffic. Only 3 violations were reported with an average daily traffic greater than 30,000.

The 3 major types of violations were correlated with average daily traffic in Figure 21. Approximately 60 percent of the violation locations for each of the major causes took place on a facility with an average daily traffic less than 6,000.

Additional Distance To Go the Legal Route

Figure 22 shows that 35 percent of the violation locations require no additional time to go the legal route. According to this graph, 80 percent of the violation locations occurred when the additional time required to go the legal route was less than 5 minutes.

Severity of Violations

The frequency of the severity of violations is shown in Figure 23. Dangerous was marked for almost 50 percent of the violation locations. An attempt was made to show

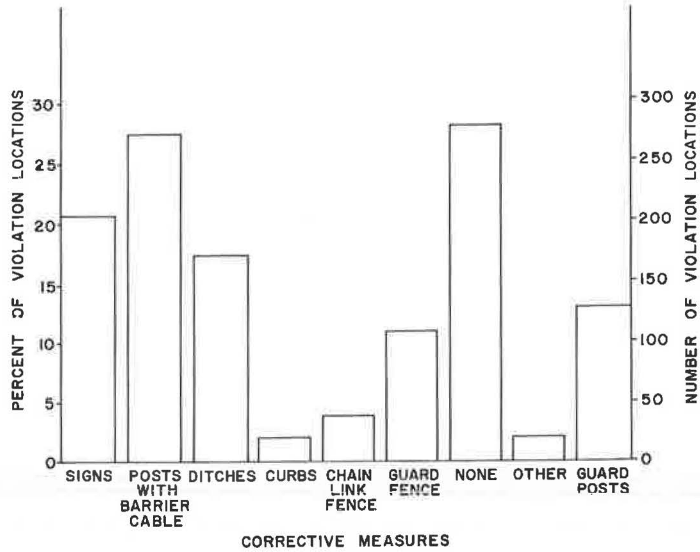


Figure 15. Frequency of corrective measures.

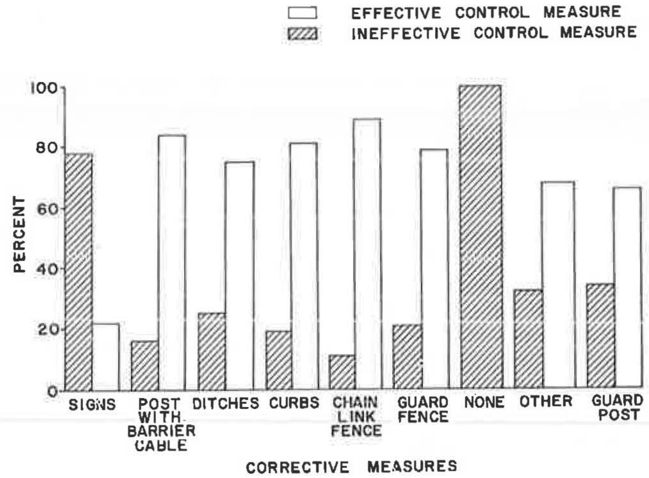


Figure 16. Effectiveness of corrective measures (opinions of field personnel).

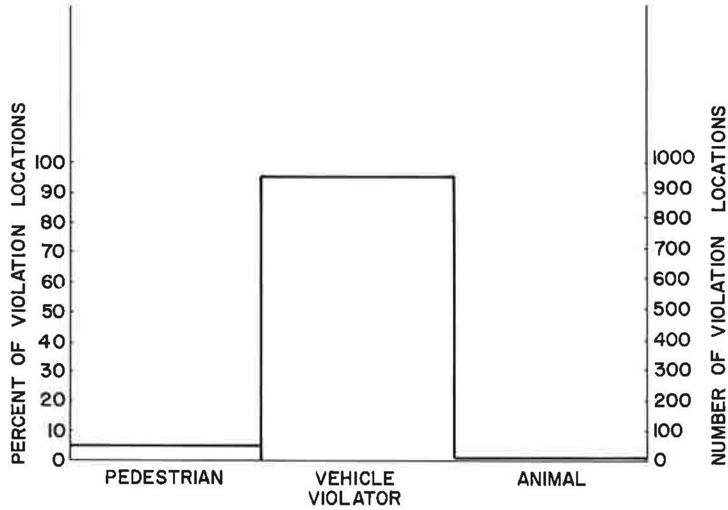


Figure 17. Frequency of violations by violator.

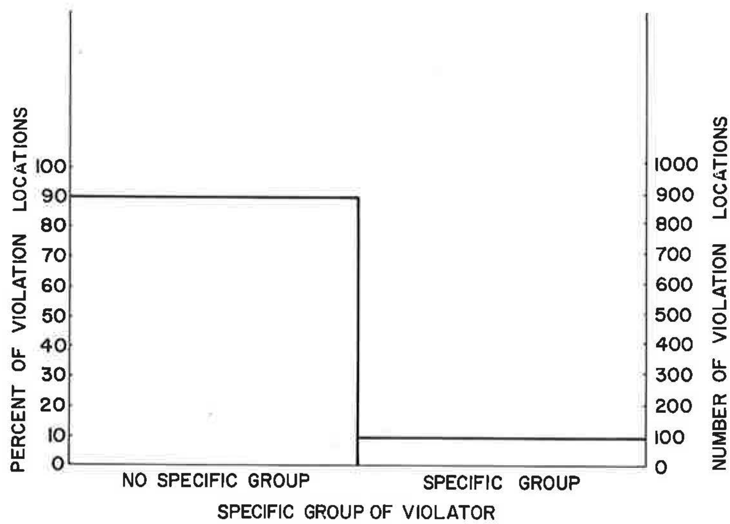


Figure 18. Frequency of violations by a specific group.

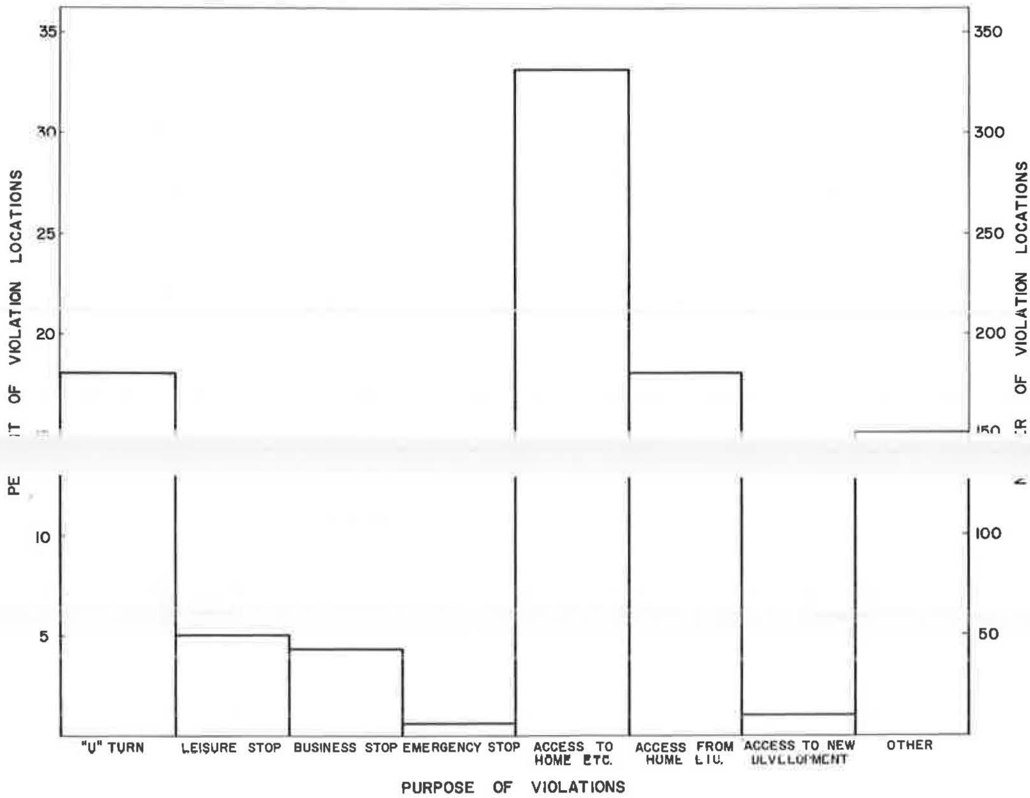


Figure 19. Frequency of purpose of violations.

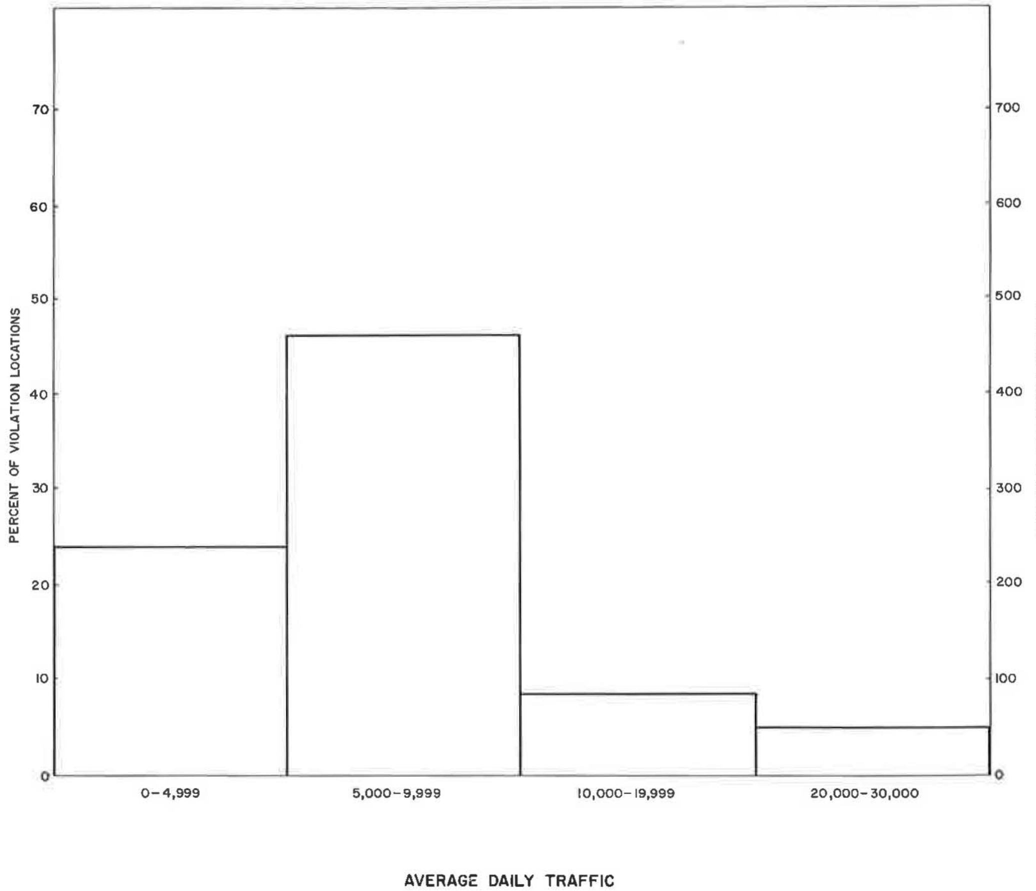


Figure 20. Frequency of average daily traffic.

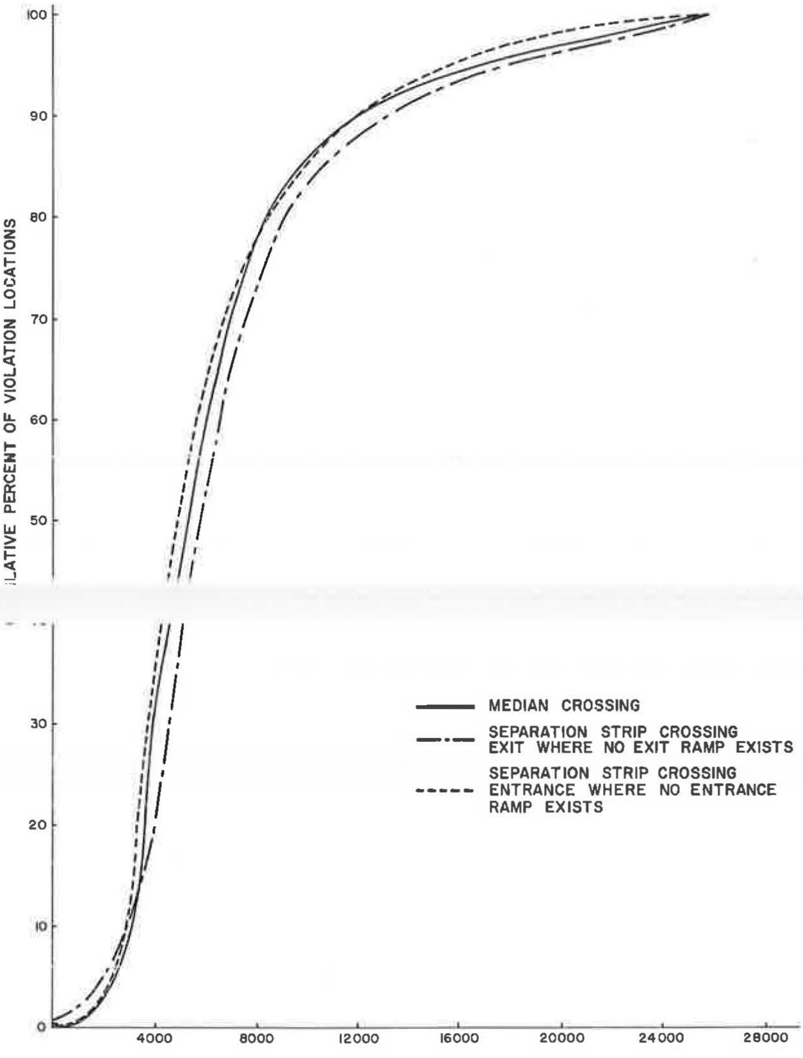


Figure 21. Relationship between average daily traffic and major types of violations.

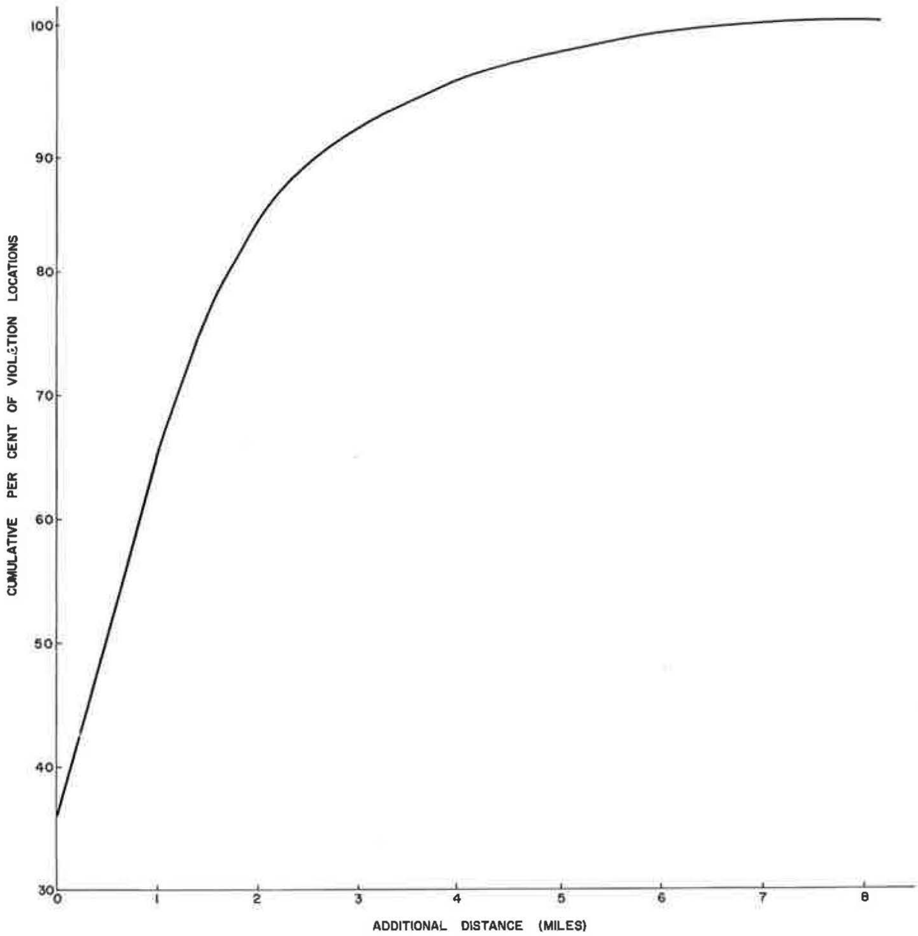


Figure 22. (a) Relationship between additional distance and all violations.

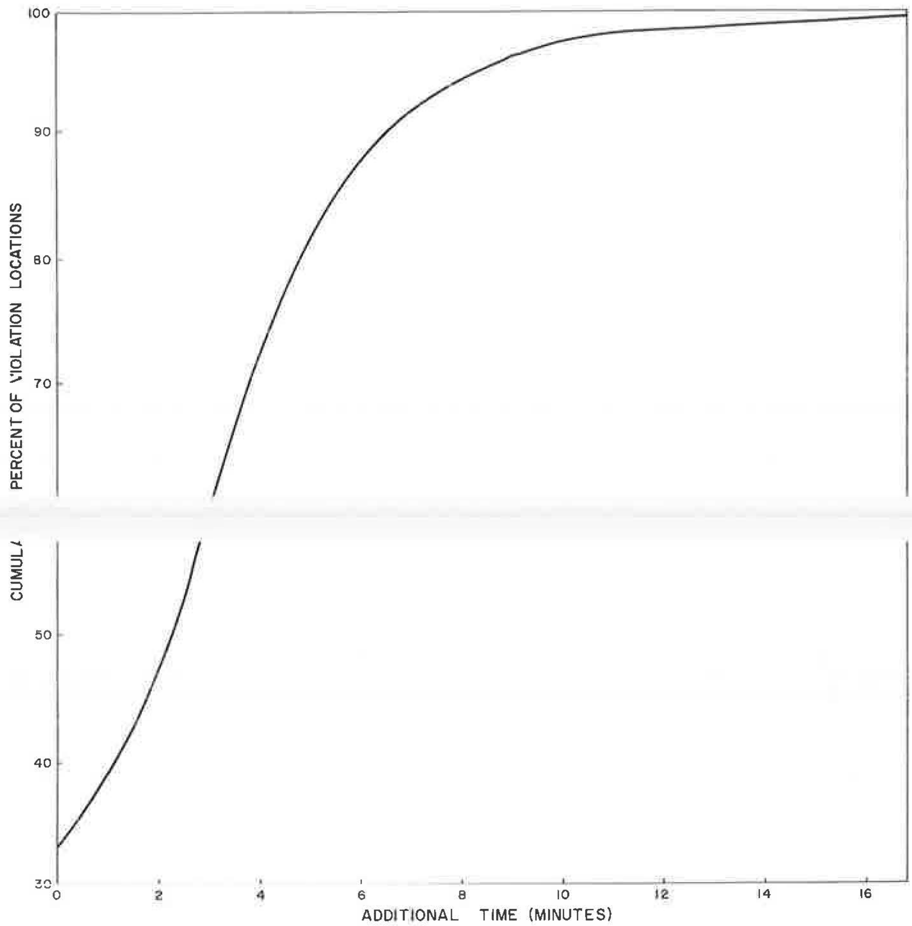


Figure 22. (b) Relationship between additional time and all violations.

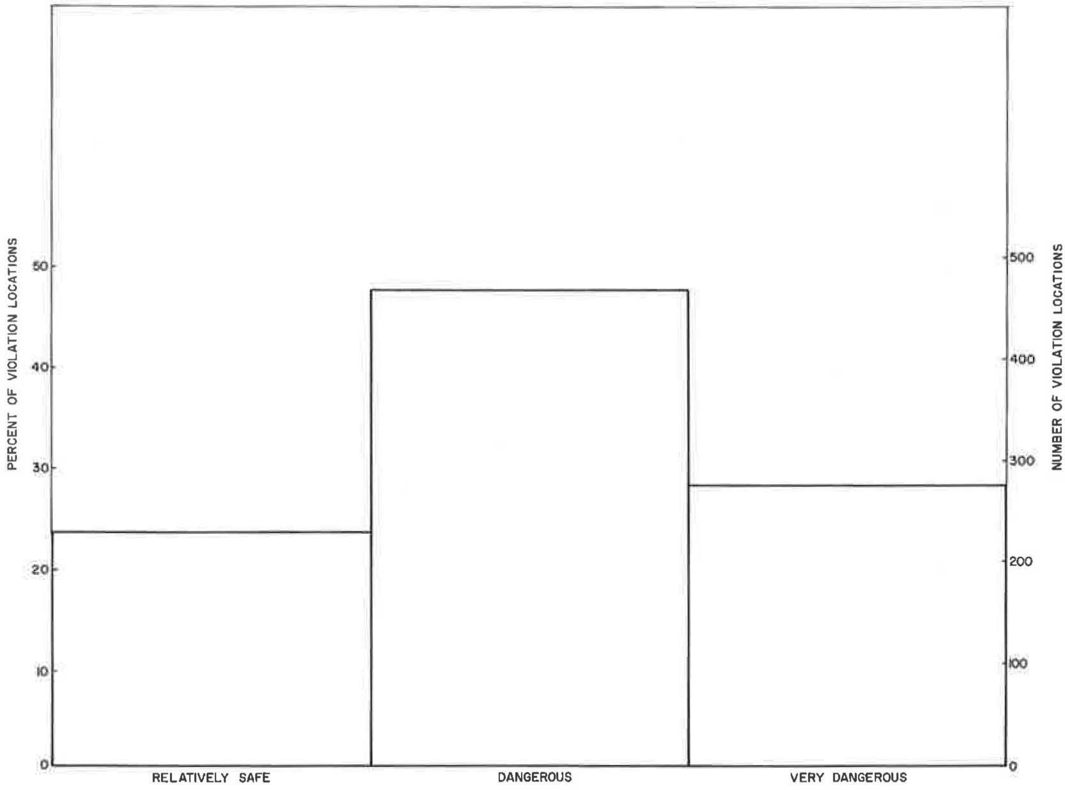


Figure 23. Frequency of the severity of violations.

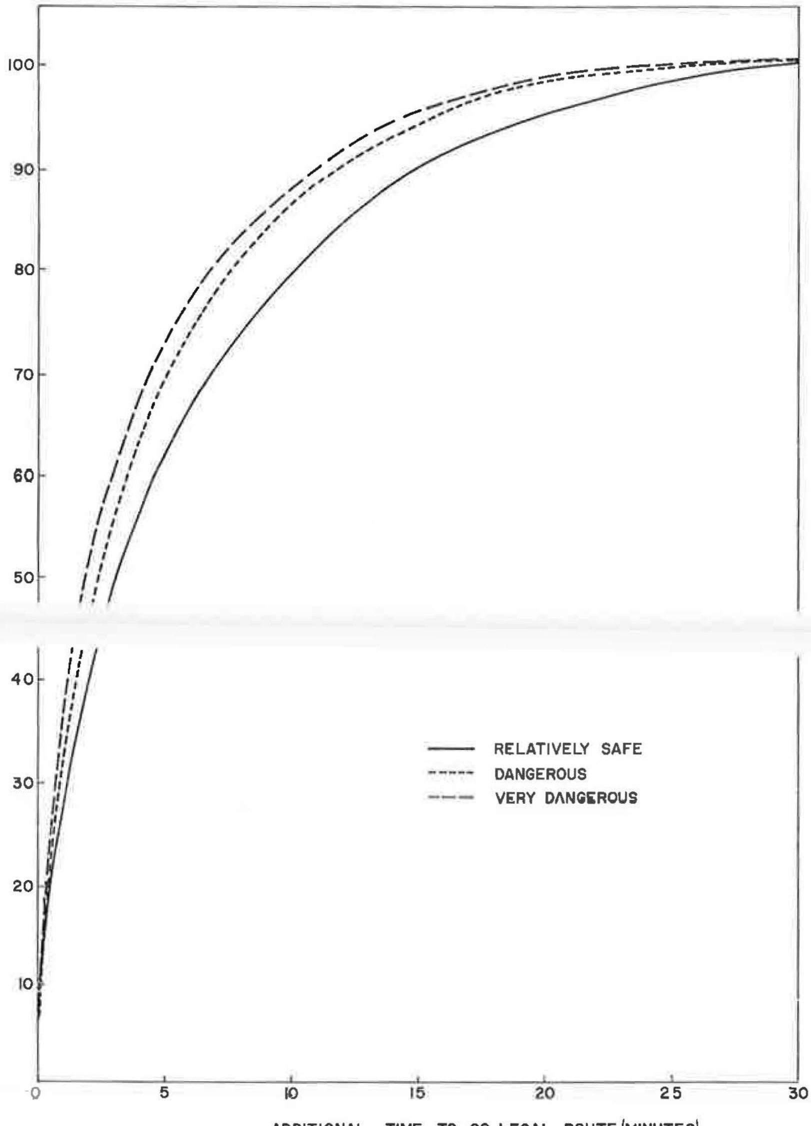


Figure 24. Relationship between additional time and severity of violation.

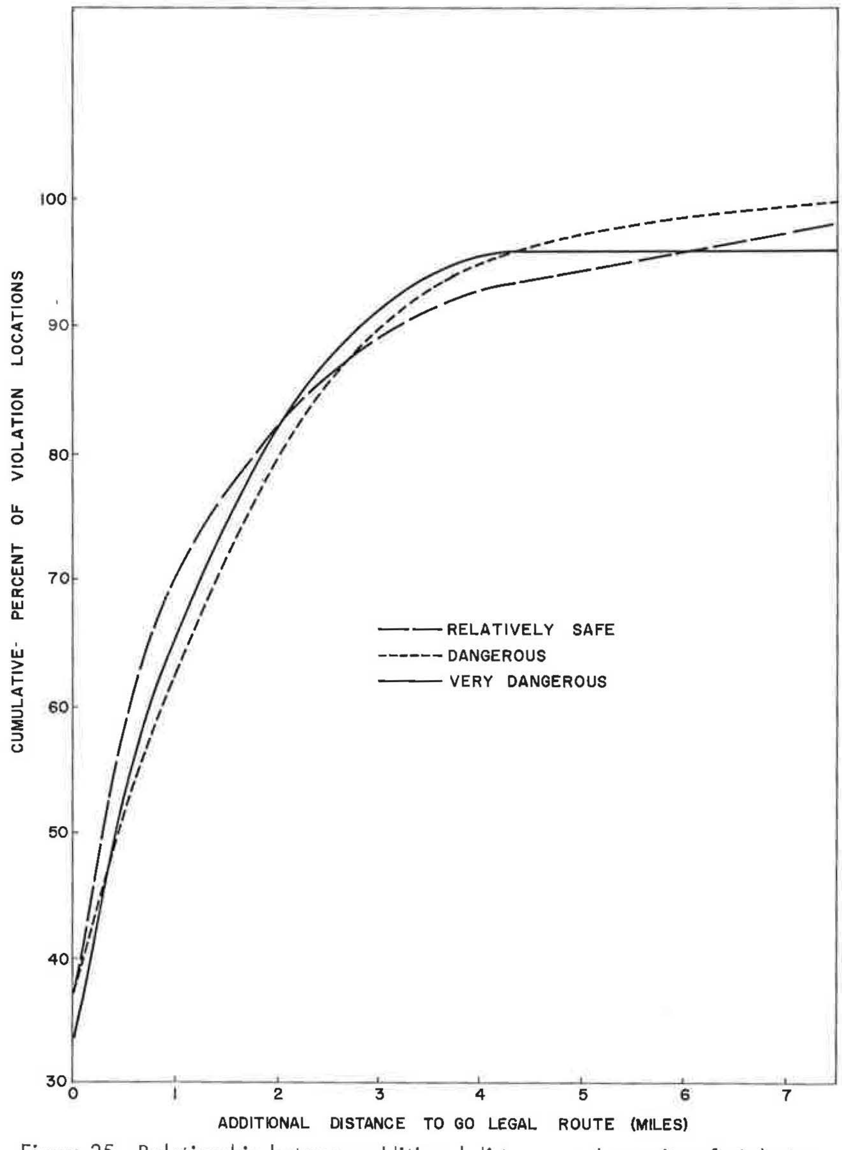


Figure 25. Relationship between additional distance and severity of violation.

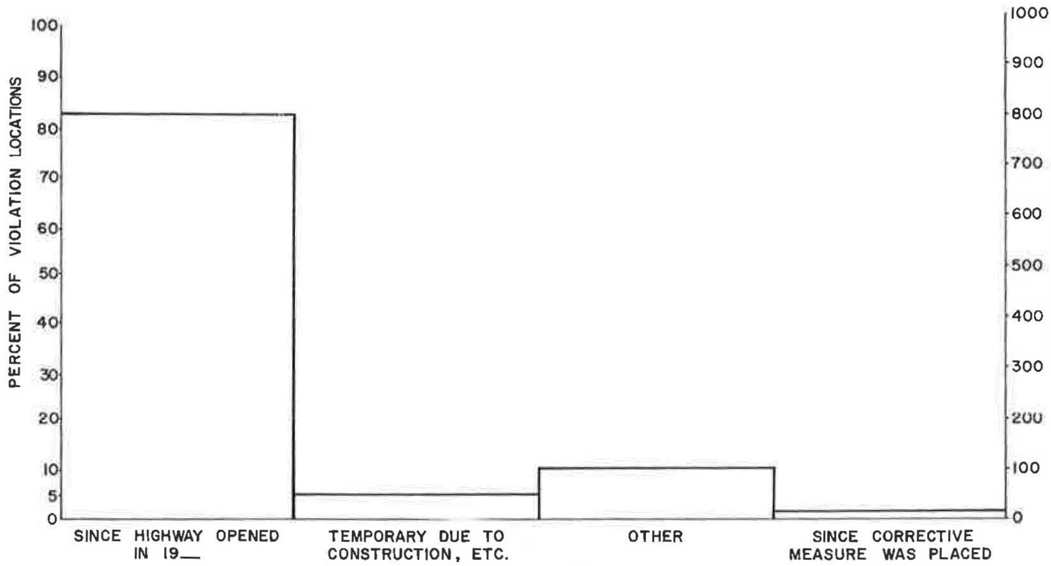


Figure 26. Frequency of presence of violation locations.

that a greater percent of relatively safe violations would occur than dangerous violations for the same additional time to go the legal route. Figure 24 did not substantiate this hypothesis. Therefore, time did not appear to be the basis for determining whether the average driver would violate or go the legal route.

[illegible] that distance was this parameter. Figure 25 [illegible] and very dangerous when the additional distance was between ½ mile and 2 miles. Up to ½ mile, there was practically no difference in the curves, and for distances farther than 2 miles, a greater percent of dangerous and very dangerous violations occurred than relatively safe violations. This comparison was based on the percentages and not on the quantities.

Presence of Violations

Figure 26 illustrates the frequency of the presence of violation locations. The percentages of the 4 bar graphs did not add up to 100 percent since all questionnaires were not completed for this question. Over 80 percent of the violations had existed since the facility opened.

Figure 27 shows the number of violations beginning on new facilities as they were opened to traffic for each of the past 12 years. Figure 28 shows that these numbered over 300. The 300 in existence should have totaled the 800 shown in Figure 26. It did not because the personnel completing the questionnaire failed to fill in the blank when the highway opened, yet realized that the violation location had been used since the highway opened, and marked this box but left the year blank.

Freeway Areas

Figure 29 shows that 75 percent of the violation locations reported occurred on rural freeway facilities and 25 percent on urban freeway facilities. This was anticipated for 3 reasons: (a) the heavy volumes on urban freeways tend to prevent violations; (b) many urban freeways have barrier curbs on frontage roads and a guardrail down the median which prevent some types of violations; and (c) approximately 130 mi of urban freeways were studied in comparison with approximately 640 mi of rural freeways. Yet, there were 1.9 access violation locations per mile of urban freeway while there were 0.85 access violation locations per mile of rural freeway.

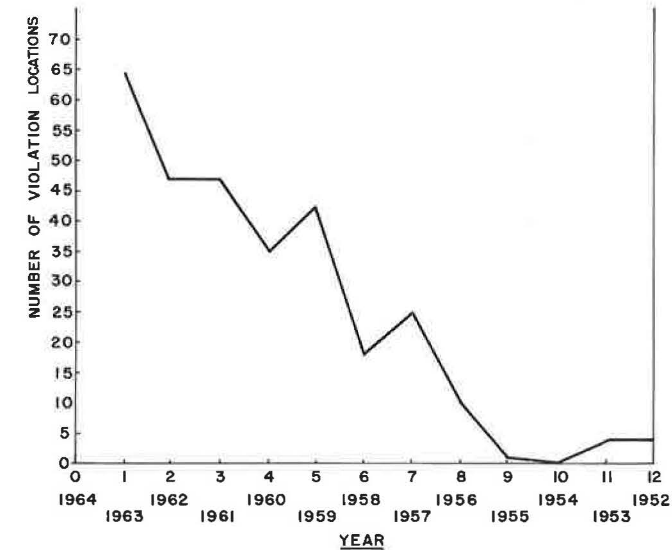


Figure 27. Number of existing violation locations beginning in past years.

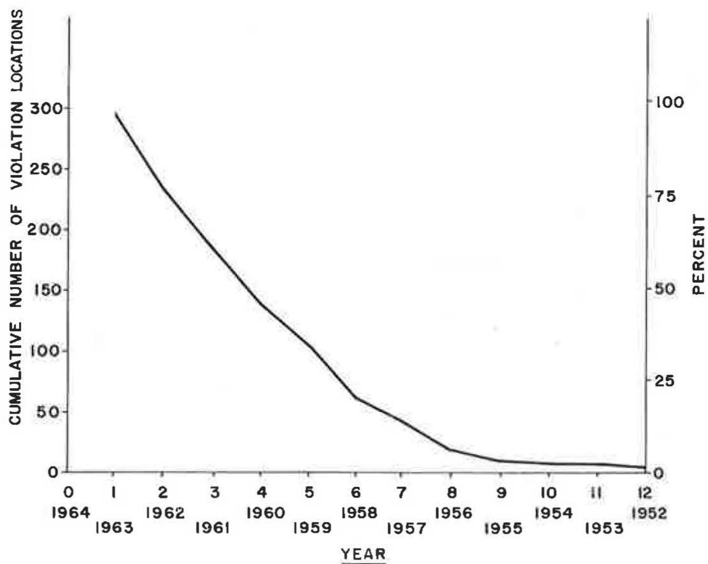
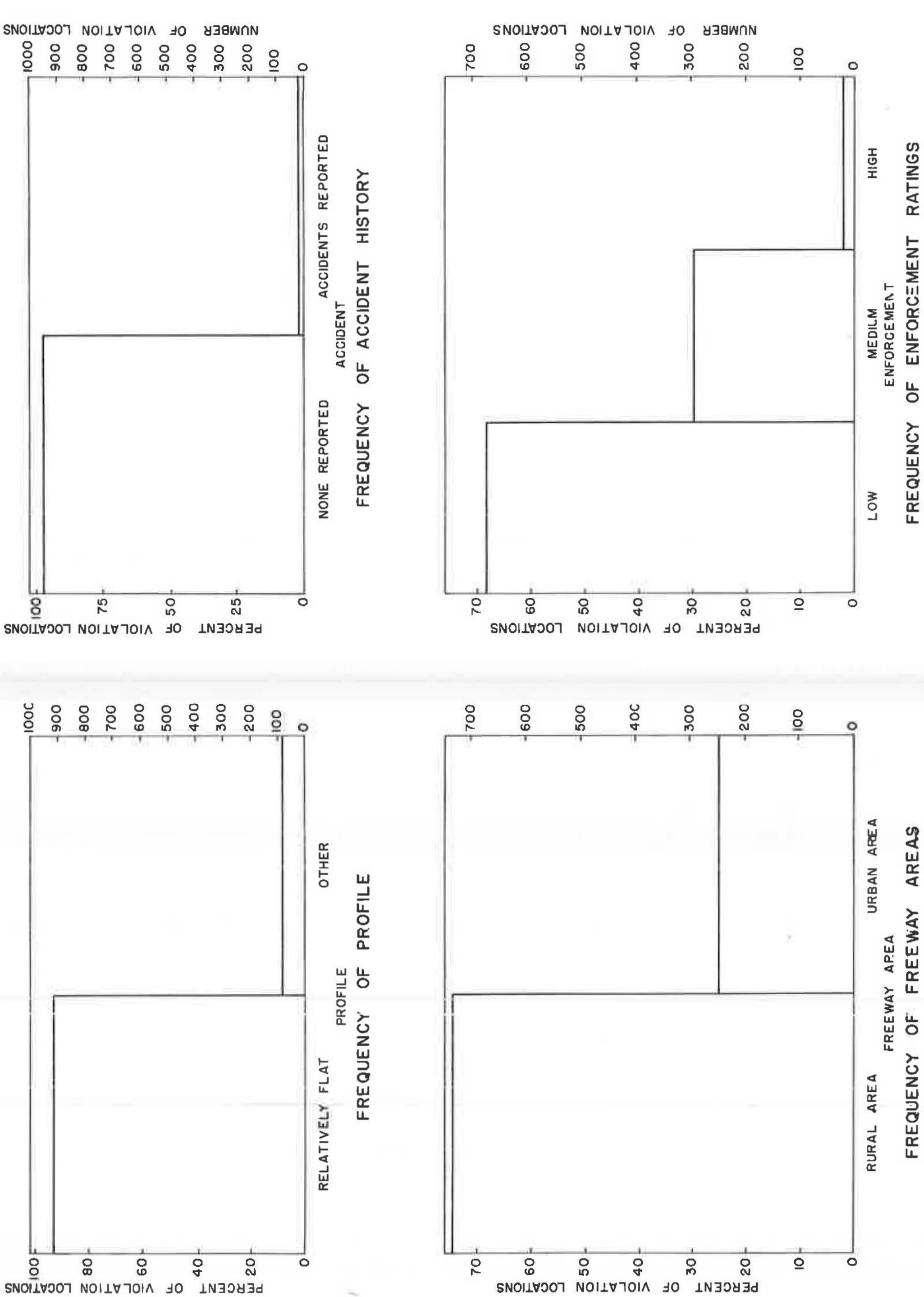


Figure 28. Cumulative number of violation locations.


FREQUENCY OF PROFILE
RELATIVELY FLAT PROFILE
OTHER
PERCENT OF VIOLATION LOCATIONS
NUMBER OF VIOLATION LOCATIONS
FREQUENCY OF ACCIDENT HISTORY
NONE REPORTED
ACCIDENT ACCIDENTS REPORTED
PERCENT OF VIOLATION LOCATIONS
NUMBER OF VIOLATION LOCATIONS
FREQUENCY OF FREEWAY AREAS
RURAL AREA FREEWAY AREA
URBAN AREA
PERCENT OF VIOLATION LOCATIONS
FREQUENCY OF ENFORCEMENT RATINGS
LOW
MEDIUM ENFORCEMENT
HIGH
PERCENT OF VIOLATION LOCATIONS
NUMBER OF VIOLATION LOCATIONS

f e 29.

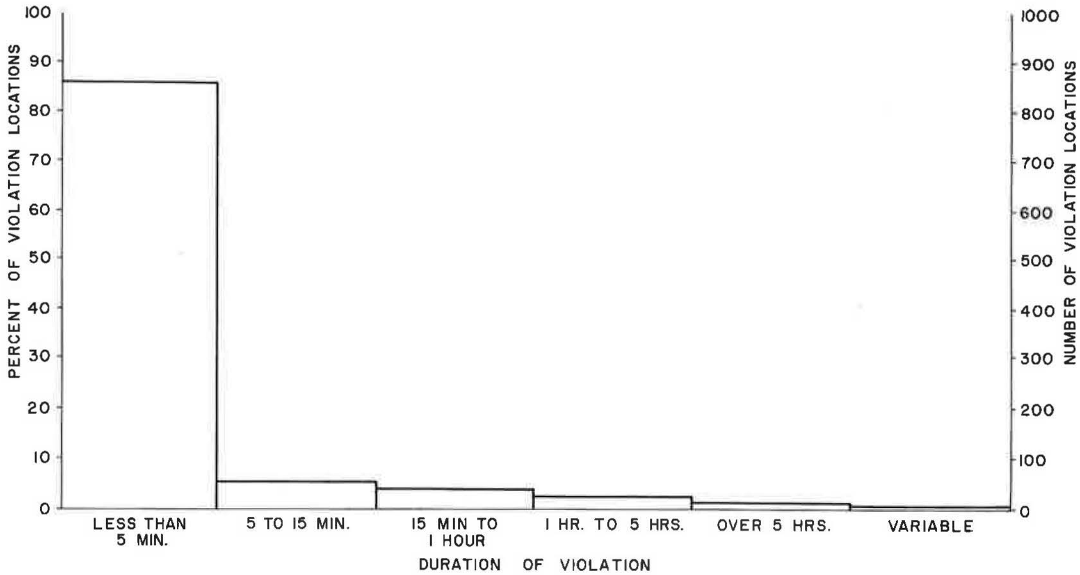


Figure 30. Frequency of duration of violation.

Accident History

Figure 29 shows that a very small number of accidents have been attributed to access violations. However, one fatality resulted from an access violation accident.

Profile

Figure 29 indicates that over 90 percent of the access violation locations occurred on relatively flat terrain.

Enforcement

Figure 29 illustrates the fact that the enforcement level for access violations was generally low. In less than 25 instances was it rated as high.

Duration of Violation

Figure 30 shows that at 86 percent of the violation locations, less than 5 min was required for the execution of the violation maneuver. Since data were not collected using a continuous observation method, a breakdown of less than 5 min was impossible.

CONCLUSIONS

The data were collected on a one-time basis on approximately 770 mi of freeway which included all Interstate highways within the state. The conclusions based on the study performed were:

1. A total of 28 separate types of access violations were observed and defined.
2. A total of 986 access violation locations were observed on approximately 770 mi of Interstate highways, a ratio of 1.3 access violation locations per mile of freeway. Twenty-five percent of these violation locations occurred on the 130 mi of urban freeway studied, a ratio of 1.9 access violation locations per mile of urban freeway. The remainder of the violation locations occurred on 640 mi of rural freeway, a ratio of 0.85 access violation locations per mile of rural freeway.
3. Five types of access violations accounted for 622 or 63.5 percent of the 986 observed access violation locations. These most prevalent types were: (a) separation

strip crossing, exit where no exit ramp exists—204 violations—20.7 percent; (b) median crossing—180 violations—18.2 percent; (c) separation strip crossing, entrance where no entrance ramp exists—112 violations—11.4 percent; (d) unattended vehicle on shoulder—68 violations—7.0 percent; and (e) crossing entire freeway system—58 violations—5.9 percent.

4. The primary cause of access violations was found to be that the violation route was the most convenient. This cause was indicated in over 52 percent of the violations.

5. Prohibitive signs were rated ineffective as corrective measures in 78 percent of the cases.

6. Curbs, chain-link fences, and posts with barrier cable had a very high degree of effectiveness.

7. Access violators were cataloged as: (a) pedestrian, (b) vehicle, and (c) animal. Of these 3, vehicles accounted for 94 percent of the access violators.

8. Approximately 60 percent of the observed violations took place on facilities with an average daily traffic of less than 6,000.

9. The study indicated an extreme desire on the part of the motorist to make direct movements on-to and off-of the freeway.

10. The severity of violations was classed as relatively safe, dangerous, and very dangerous. The persons reporting the data indicated: (a) relatively safe—24 percent of the violation locations; (b) dangerous—49 percent of the violation locations; and (c) very dangerous—27 percent of the violation locations.

SIGNIFICANCE OF RESULTS

In the past, the geometrics of an individual road were developed primarily in relation to safety, right-of-way, physical controls, and economic feasibility. However, because the construction of new facilities vitally affects traffic operations and maintenance procedures, the design of fixed facilities has come to be of special concern to traffic and maintenance personnel. Design engineers have begun, more and more, to consider both traffic and maintenance operations. The principles of location and design must be developed with references to the needs of through and local traffic and

There are many traffic factors which influence geometric design. Vehicles travel the highway under the control of individual operators making it imperative that the abilities and limitations of the driver, vehicle, and the road, both individually and in combination, be considered. This report dramatizes that even the drivers' shortcomings must be taken into consideration. The geometry of the highway facilities must be related to traffic performance and the demands of traffic in order to achieve safe, efficient, and economic traffic operations.

Freeway access violations are the operational effects of certain freeway geometrics, notably ramp spacing, ramp configuration, and the freeway cross-section elements. Traditionally, the chief criteria for freeway connections are the position of major cross-streets and highways, and ramp capacity. The omission of a ramp where traffic desires exit can and does tempt violations. The design of exit ramps, certainly in rural areas, should avoid the appearance of two-way roadways, or any other illusion which might encourage drivers on the crossroad to enter the ramp in the improper direction. It is evident from this study that some travelers on rural frontage roads are not alert to one-way exit ramps. It is not inconceivable that two-way frontage roads will suffer the same fate as the 3-lane, two-way highway of the 1930's. It suffices to say that one-way operation of frontage roads is preferred and should be employed unless there are compelling reasons to the contrary.

The most effective method of reducing access violations at the design level lies in the design of the median and the separation strip. The median serves to delineate the left extremity of the authorized path of vehicle travel, decreasing the amount of inadvertent vehicle encroachment and providing space for vehicles running off the left edge of the pavement to regain control. Landscaping in medians, often used to reduce headlight glare, offers a positive deterrent which traffic will not cross intentionally. The median concept offers flexibility to the planner and the designer. Medians should be wide enough to serve as a recovery area and positive enough to discourage crossing violations without completely isolating the opposing roadways from each other. A median which cannot be crossed or which prevents visibility nullifies the deterrent effect of police patrols.

Emphasis on safety has led to the incorporation of the separation strip as part of the recovery area available to vehicles out of control. Functionally, however, the separation between a freeway and its frontage road prevents the interference of through movement by local traffic. It should physically discourage crossings from one road to the other except at ramps. Side slopes of 6:1 seem to fulfill both criteria; 8:1 is too easily negotiated, whereas side slopes of 4:1 are too severe for recovery purposes. Pedestrians on freeways are a significant source of access violation. Where special access problems are involved, such as for schools, crossings exclusively for pedestrians may be a suitable solution. Pedestrian overpass sidewalks above the freeway should be enclosed in solid or wire mesh screens to avoid the dropping or throwing of objects from these structures.

After the freeway is built, the problem of coping with access violators is an operational one. Signing and pavement marking are the conventional procedures used to regulate, warn or guide road users. The effectiveness of any sign depends upon its attention, meaning, and respect value. Unfortunately, there is little standardization in signs guarding against access violations.

The recent development of "electronic policeman" (2) in which wrong-way drivers get a positive warning such as a "Stop-Turn Back" illuminated on an exit ramp sign, offers considerable promise. When a car on a secondary road mistakenly enters a freeway exit ramp, 2 wire loops buried in the pavement detect the vehicle and set up a signal that lights the sign. Circuits are arranged to ignore vehicles passing over them in the correct direction. Developments such as these are needed to insure proper operation of off-ramps.

RECOMMENDATIONS

One district, when returning its data collection forms, noted that the frequency of innocent wrong-way violations on exit ramps could not be ascertained by the once-over method of data collection, since no tracks were left and the violation was infrequent. It was recommended that data be collected on a surveillance method to determine the frequency and extent of this type of violation to be significant, studies should be undertaken to determine the best methods to eliminate this type of violation.

The extent and severity of access violations shown in this report suggested additional studies on this subject. These studies should determine:

1. Geometric design changes in freeway facilities which will coincide more closely with drivers' desires. The closer the designer can come to meeting all drivers' desires, the greater the number of violations he will eliminate before they ever occur.
2. The most feasible control measure to be used in eliminating violations now existing on freeway facilities. Factors to be considered should be:

   a. Would any control measure be more of a hazard than the benefit of eliminating the access violations? John W. Hutchinson (3) furnished encorachment rates for the highways he studied. He also notes accident experience with obstacles in the median. Studies of this type must be accomplished to answer this question.
   b. What would be the severity of the accident if the control measure were run into? The most prevalent method presently used in eliminating violations in Texas is wooden posts with barrier cable running between posts. This corrective measure should be tested to determine how the cable reacts when broken during an accident. Does the cable drop to the ground or whip through the air? Possibly another type of post or a smaller wooden post should be used for corrective measures to reduce the severity of any accidents hitting these posts. Actual crash tests on these corrective measures can answer these questions.

c. Would the night visibility of the control measure be adequate? The anticipated problem here is the cable strung between posts. Presently, at some locations, one reflector is attached to the cable centered between the posts. Studies should be conducted to determine if these are adequate to prevent drivers from unknowingly attempting to drive through the cable at night.

d. The need for crossovers for use by police, ambulances, and maintenance vehicles. One method used in Maryland for limiting crossover usage to emergency vehicles is a radio-operated median gate (4). Hutchinson (3) states that agencies requiring emergency access across the median where posts and barrier cables were in place were instructed to carry bolt cutters for the purpose of cutting the barrier cable when necessary. Studies would be undertaken to determine the best method of providing this access.

3. The control measures to be included in the original design and construction to prevent violations that cannot be eliminated through geometric design changes.

REFERENCES

1. Shoulder and Rest Area Procedure Guide. Committee on Shoulders and Medians, HRB Bull. 359, 1962.
2. RCA Develops Device Warning Motorists They're Going Wrong Way. Traffic Engineering, Vol. 35, No. 10, p. 32, July 1965.
3. Hutchinson, John W. The Significance and Nature of Vehicle Encroachment on Medians of Divided Highways. Univ. of Ill., Urbana, Civil Engineering Studies, Highway Engineering Series No. 8, Dec. 1962.
4. Hallstead, William F. Maryland Tries Radio-Operated Median Gates. Public Works, Vol. 94, No. 7, p. 78, July 1963.
5. A Policy on Geometric Designs of Rural Highways. AASHO, 1964.
6. Freeway Operations. The Institute of Traffic Eng.

Non-Intersectional Automobile Fatalities—A Problem in Roadway Design

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•AS has been pointed out in the articles by Stonex (1-5) and others (6-10), proper highway design could save 10,000 lives each year by preventing single car, off-road accidents. In addition, approximately 6,000 lives could be saved annually by preventing head-on collisions. Thus, 16,000 lives could be saved each year by improved highways and adjacent areas. These authors have indicated that trees, ditches, roadside slopes, guardrails of poor design, sign and light posts, bridge abutments, and other features are the problem, in that all too often they are left near the roadway or are placed close to the roadway and left exposed. Stonex (1) has shown that an obstacle-cleared roadside of approximately 33 ft would provide safety for at least 80 percent of the drivers leaving the road, and that a 50-ft obstacle-free roadside area would assure safety for 90 percent.

There are many reasons why a vehicle will leave the roadway or cross into the opposite lane. A few of the causes of accidents are driving too fast for existing conditions, falling asleep, drinking, roadway hazards, mechanical failure of the vehicle, and driver inattentiveness. Seldom can only one of these factors be blamed for the accident, but most all can be categorized as "driver misjudgment."

Since November 1961, we have been studying fatal automobile accidents in and about Washtenaw County, Michigan. The purpose is to determine the causes of death of the occupants—the body areas injured, as well as the structures which were impacted to produce lethal injuries. The police of the area call us to all on-scene fatalities anytime of the day or night. Photographs of the vehicles, roadway, and victims are taken using 35-mm color film. As of January 1, 1965, we investigated 111 accidents in which 146 automobile occupants were killed. No pedestrians, cyclists, car-train, or truck-truck accidents are included in the data.

In studying each case certain conditions become important. The occupant would have lived—if he was not driving too fast, if he had not fallen asleep, if he had not been drinking, if he had worn a seat belt, if the interior of the vehicle had been designed for safety, if the roadway had been better designed, if no roadside obstacles had been present, etc. We contend that there will always be the possibility of an automobile accident when there is a man-machine combination. Thus, in addition to attempting to decrease accidents by driver education, vehicle inspection, etc., the only alternatives are improvement in vehicular design for crash attenuation (especially the interior), and by proper roadway design and clearance of roadside obstacles. If an individual is going to lose control of his vehicle for any reason, the roadway must be designed to prevent cross-median accidents, and obstacles must be removed from the roadside so that serious or fatal injuries will not occur.

In this study 84 percent of the accidents were non-intersectional collisions with the majority (60 percent) being single car, off-road collisions (Table 1). In one of five accidents (18 percent), a vehicle invaded the roadway of another by crossing the centerline or median, or traveled in the wrong lane on an expressway.

Paper sponsored by Committee on Highway Safety and presented at the 45th Annual Meeting.

In 21 cases, more than one roadway hazard could be considered important (Table 2). The most obvious hazard is indicated first; however, other obstacles or design factors played an important part in the fatal accident. For example, a tree-ditch combination

TABLE 1

TYPE OF FATAL ACCIDENT AND OBJECT CAUSING FATALITY

Single Car Collisions	Number
Tree or utility pole[a]	35
Bridge abutment	5
Guardrail or post	4
Earth embankment	4
Rollover (due to):	
Ditch	8
Slope or embankment	6
Lost control on roadway	5
Subtotal	67
Car-to-Car Collisions	
Intersectional	18
Crossed median[b]	10
Crossed centerline	10
Rear end[c]	6
Subtotal	44
Total	111

[b]In two crossed median cases the vehicle was going the wrong way on the expressway.

[c]One case the struck vehicle went through guard posts and down a slope.

TABLE 2

MULTIPLE ROADSIDE OBSTACLES OR DESIGN FACTORS CONTRIBUTING TO FATAL ACCIDENT

Bridge abutment and guardrail	3
Earth embankment and guardrail	1
Three (and):	
Ditch	7
Slope	2
Ditch and slope	1
Guardrail	1
Slope (and):	
Ditch	1
Guardrail	2
Tree	1
End of culvert	1
Rear-end collisions and guard posts and slope	1

Figure 1. Vehicle went off left side of road and struck tree stump 10 ft from edge of road. Driver killed.

indicates that the tree was struck by the vehicle but the ditch kept the car "on track" to the impact point. The rear-end collision was not the real cause of the fatalities in the case in Table 2, but the struck car went through guard posts and down a steep slope causing the ejection of two occupants. Had the vehicle been stopped by a guardrail of proper design these fatalities would not have occurred.

Figure 2. Vehicle left roadway on right side of curve and struck tree (center) 15 ft from edge of road. Driver killed.

Figures 1 to 5 illustrate accidents where the vehicles left the roadway and impacted trees. The trees were less than 15 ft from the roadway. In the case illustrated by Figure 5, the owner of the adjacent property had asked for removal of the tree in the roadway several times before the fatal accident, but the tree remains 3 years afterwards. This tree is now marked for removal, not because of its location, but because of Dutch Elm disease. Trees, or less often utility poles, account for one-third of our fatalities.

Improper ditch design led to at least eight fatal accidents in this study. Figure 6 illustrates a deep ditch adjacent to a curve in the road. The vehicle went off the roadway and traveled in the ditch some 30 ft. It then rocketed out the far side of the ditch and flew 50 ft through the air. The vehicle landed on its front end and, in the upright position, continued for 10 ft striking a tree with the roof, hood, and trunk, killing the driver. Here, two

Figure 3. Off-road tree impact on a curve; tree 15 ft from roadway. Three killed. In this and Figure 2, trees are just beyond apex of curve, the point where a vehicle would strike if it left roadway.

Figure 4. Photograph of car-tree impact.

roadside hazards are apparent—the deep ditch along the roadway, and the tree located on the convex side of the curve, 31 ft from the edge of the road.

In Figures 7 and 8, impacts of vehicles against earthen embankments adjacent to the roadway caused the fatalities. An example of two poorly designed roadside factors playing a part in the fatal collision course of the vehicle is shown in Figures 9 and 10, where there is a narrow drainage ditch adjacent to the embankment. The vehicle left the roadway and the right wheels caught in the ditch. The slope then increases markedly, preventing the vehicle from returning to the roadway. The vehicle struck the trees and the driver was killed.

Approximately 9 percent of our cases are cross-median accidents. In Figure 11 (showing widely separated expressway lanes) the vehicle was traveling from right to left, failed to navigate the curve, crossed the median and struck another car. Tire tracks are evident in the median.

Figure 5. A tree in the road—the scene of accident shown in Figure 4. In Figure 20, this case was plotted at zero feet from road edge.

Narrow medians contributing to fatal accidents are shown in Figures 12 to 15. The vehicle in Figure 13 crossed the median shown in Figure 12, and was struck by a semi-trailer. Figure 14 shows the tracks of a vehicle which crossed a median. The right front tire hit the bottom of the drainage ditch in the center of the median; the vehicle rolled over and struck an oncoming car. This was a roof-to-roof contact (Fig. 15). Figure 16 illustrates a case where a flat-bed trailer broke loose, crossed the median, and struck a car head-on, killing the passenger.

Figure 6. Deep ditch adjacent to road. Vehicle traversed ditch, flew 50 ft and impacted the roof, hood, and trunk against tree located 31 ft from roadside.

Improper guardrail design was a factor in a single car accident (Fig. 17). The vehicle traveled across the narrow median (9 ft), recrossed the median and struck the guardrail head-on. The guardrail deformed in such a manner that it caused the vehicle to roll over, and the driver was killed. In another accident a vehicle struck the guardrail with the right front bumper, which caught on the posts causing the vehicle to spin; the left rear door of the car then hit the end of a bridge rail (Fig. 18).

There are also several cases where exit ramps of expressways have been used to enter the wrong lane of traffic. In one, the driver became confused at a triple-overpass complex, went the wrong way on the expressway, and struck another car (Fig. 19). Two passengers were killed. At a court trial, the judge indicated that the defendant could not be charged with criminal negligence because there were no road signs giving notice that the one-way expressway was ahead.

Figure 7. Vehicle "missed the curve" knocking down several guard posts and hit earth embankment in the distance. Proper guardrail design would have prevented a straight-on course to embankment which should have been removed. Impact point location 20 ft from road edge.

Figure 8. Attempt to return to roadway by braking turned vehicle into embankment on opposite side of road. Driver killed—embankment 10 ft from roadway.

Figure 9. Entrapment of vehicle by drainage ditch next to embankment led it to distant trees. Ditch 10 ft from roadside.

Figure 10. Steep slope prevented return of car to roadway before impact. Trees are 25 ft from road edge.

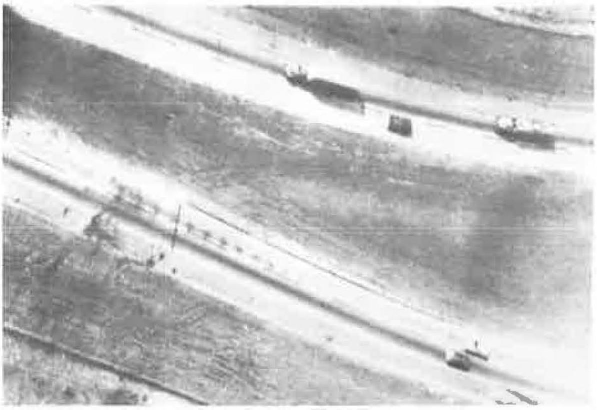

Figure 11. Aerial view of wide median at a curve in an expressway. Vehicle crossed median as indicated by marks in turf. Three killed. Path of vehicle across median was approximately 95 ft.

Figure 12. Narrow (28 ft) median at point of crossover. Car-truck collision, one killed.

Figure 13. Vehicle involved in cross-median accident shown in Figure 12.

Figure 14. Narrow (26 ft) median at point of crossover. Vehicle hit drainage ditch in center of median, rolled over striking another car.

Figure 15. Roof-to-roof impact due to deep drainage ditch in center of median shown in Figure 14. Four killed.

Figure 16. Narrow (27 ft) median allowing flat bed trailer to cross and strike a car.

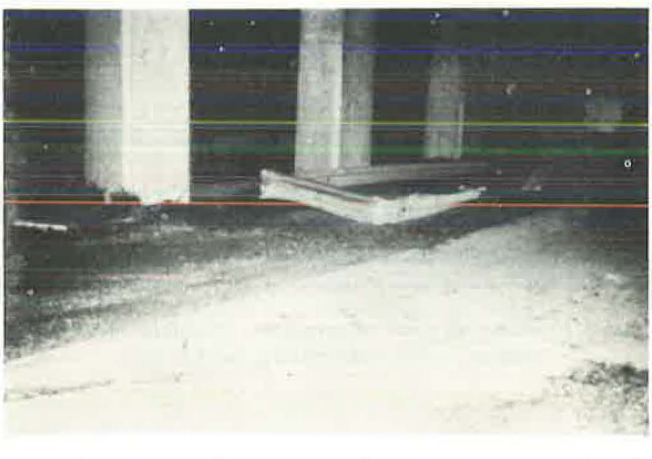

Figure 17. Vehicle crossed narrow median, attempted to return to proper lane by recrossing median and struck guardrail head-on, causing a rollover. Driver killed.

Figure 18. Vehicle "snagged" by guardrail, spinning it into bridge railing. Shown is the point of impact, left rear door. One killed.

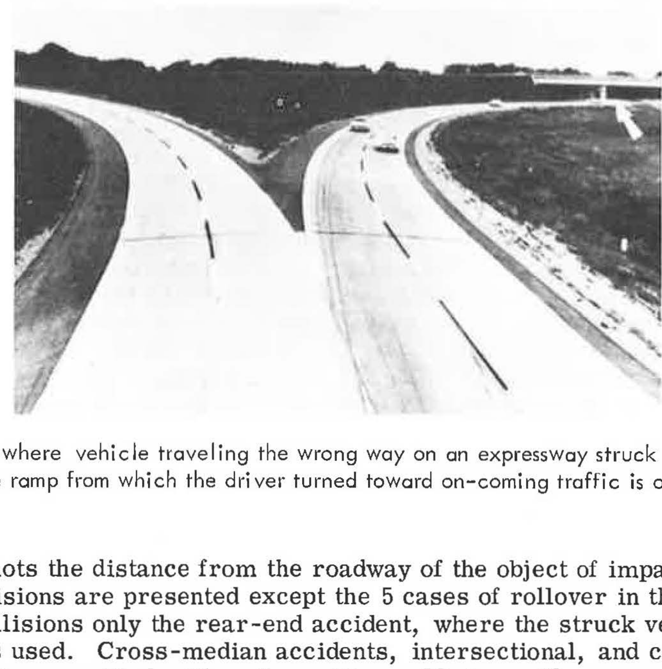

Figure 19. Area where vehicle traveling the wrong way on an expressway struck another car (arrow). Entrance ramp from which the driver turned toward on-coming traffic is on the left.

Figure 20 plots the distance from the roadway of the object of impact. All data of single car collisions are presented except the 5 cases of rollover in the roadway. Of the car-car collisions only the rear-end accident, where the struck vehicle went through guard posts, is used. Cross-median accidents, intersectional, and cross-centerline accidents have been omitted. Therefore, Figure 20 shows 63 cases where the roadway or roadside design, obstacles, etc., are considered one of the leading factors in the fatal accident. All of the hazards are no more than 32 ft from the edge of the road.

Obviously, one need not travel too far off the roadway to strike an obstacle. If, as has been shown (1), the roadside is cleared of obstacles for 33 ft from the edge of the road, probably 80 percent of the accidents would not have occurred. In this distance the driver could have regained control of the vehicle.

Analysis of a small sampling of the Cornell ACIR data by Stonex indicates that 80 percent of the vehicles struck an object within 12 ft of the roadway. Our data indicate

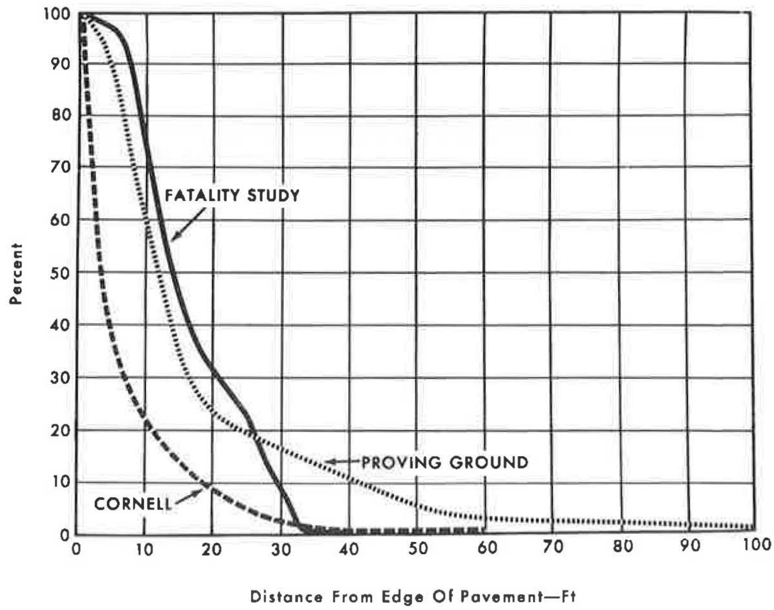

Figure 20. Distribution of impacted roadside obstacles vs distance from edge of roadway (63 cases).

that of 63 cases, 80 percent struck an object within 27 ft of the road's edge. Although the General Motors Proving Ground data indicate that control of 80 percent of the vehicles can be regained within 33 ft of the pavement's edge, this is true only if proper roadside clearance has been carried out. In our cases, ditches, embankments and

courses have been presented. If these were to be removed by slight modification and maintenance of the roadside, it would be a minimum and practical first step toward the total elimination of roadside obstacles.

Hutchinson (8), studying a 40-ft wide median, has shown that only 20 percent of the vehicles entering the median traveled more than 33 ft. He believes that the slope characteristics play an important role in the distance traveled. In addition to the two accidents where a vehicle was traveling the wrong-way on the expressway, we have found that most of eight cross-median accidents were due to the narrow width of the medians. Median widths in five cases were 26 ft, 28 ft, 32 ft, 40 ft, and 95 ft. In three cases they were 27 ft.

Can roadways be designed with adequate clearance and characteristics to prevent fatal accidents? Recently a new bypass has been opened in the Ann Arbor area which exemplifies some excellent roadside design features. Figure 21 shows an entrance ramp to the expressway, with an obstacle-clear, off-road area, and a smooth, gentle slope. In sharp contrast is Figure 22, where a steep slope is protected by only the guard posts and cables. At expressway speeds, this type of guardrail could be penetrated and a vehicle could roll down the slope. Figure 23 shows another example of good roadside design, but the lone large tree in the distance is a definite hazard.

Clear roadside areas are often hazardous because of open drainage ditches or earthen mounds close to the roadside (Figs. 24, 25). Open ditches should be closed over or constructed farther from the edge of the road; earthen embankments should also be eliminated.

Figure 26 shows a partially protected man-made obstacle. The relatively flat off-road area is hazardous because the support post of the sign is unprotected. Extension of the guardrail around the front of the sign would offer protection.

Figure 21. Expressway entrance ramp. Note obstacle clear, off-road area.

Figure 22. Steep slope and inadequate guardrail adjacent to expressway.

Figure 23. Clear area off roadway except for large tree.

Figure 24. Deep drainage ditch near expressway.

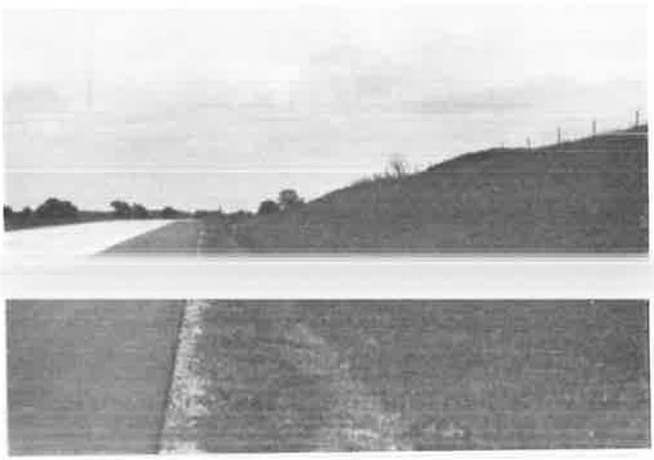

Figure 25. Earthen embankment approximately 25 ft from edge of road.

Figure 26. Expressway sign with support completely unprotected by guardrail.

Figure 27. Unprotected bridge abutment—a definite hazard on expressways.

Bridge pillars (Fig. 27) or the ends of bridges should be protected by energy-dissipating and vehicle-decelerating guardrails of adequate design.

Expressway overpasses are often inadequate from the standpoint of safety. Figure 28 shows a sidewalk adjacent to a bridge rail. This type of design is very inadequate in respect to crash attenuation. Parenthetically, it is illegal to walk on expressways in most states so that sidewalks are not needed. Better bridge rail configurations have been designed and crash tested in California, and more recently, by the General Motors Corporation (Fig. 29). The GM bridge rail design will not snag the vehicle and allows controlled return to the roadway (10).

The importance of roadside design cannot be overemphasized for its life saving benefits and injury reducing potential. We have discussed how roadside clearance can

Figure 28. Typical bridge rail design with sidewalk.

Figure 29. Prototype bridge rail designed to allow controlled return of vehicle to roadway (courtesy

be instituted. More thought must be given to the concept of roadside clearence as a definitive step toward reducing the number of highway fatalities and injuries.

REFERENCES

1. Stonex, K. A. The Single-Car Accident Problem. Soc. Auto. Eng., Paper 811A, Automotive Engineering Congress and Exposition, Detroit, Mich., Jan. 13-17, 1964.
2. Stonex, K. A. Relation Between the Automobile and the Highway. HRB Proc., Vol. 33, pp. 91-103, 1954.
3. Stonex, K. A. Vehicle Aspects of the Highway Safety Problem. Traffic Safety Res. Rev., Vol. 6, No. 2, pp. 15-24, 1962.
4. Stonex, K. A. Roadside Design for Safety. HRB Proc., Vol. 39, pp. 120-156, 1960.
5. Stonex, K. A. Relation of Cross-Section Design and Highway Safety. Traffic Safety Res. Rev., Vol. 7, pp. 3-14, 1963.
6. Lundstrom, L. C. Safety Aspects of Vehicle-Road Relationships. Soc. Auto. Eng., Paper 66B, Summer Meeting, Atlantic City, New Jersey, June 1958.
7. Lundstrom, L. C., and Skeels, P. C. Full Scale Appraisals of Guardrail Installations by Car Impact Tests. HRB Proc., Vol. 38, pp. 353-355, 1959.
8. Hutchinson, J. W. The Significance and Nature of Vehicle Encroachment on Medians of Divided Highways. Civil Engineering Studies, Highway Engineering Series No. 8, Univ. of Illinois, Urbana, Dec. 1962.

9. Beaton, J. L., and Field, R. N. Dynamic Full-Scale Tests of Median Barriers. HRB Bull. 266, pp. 78-125, 1960.
10. Cichowski, W. G., Skeels, P. C., and Hawkins, W. R. Appraisal of Guardrail Installations by Car Impact and Laboratory Tests. HRB Proc., Vol. 40, pp. 137-149, 1961.

Final Report on Long-Term Load Tests on Cylindrical Concrete Foundations for Overhead Sign Supports

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•CYLINDRICAL concrete foundations of an economical design for mounting pole-supported traffic control devices were constructed and tested in 1957 in order to determine the overturning resistance of such foundations. Tests were conducted at three locations which were selected to provide a range from very good to very poor soil conditions. Short-term load tests were conducted to measure the strength characteristics under transient loading conditions such as wind loads on large sign areas. The same foundations were then left under fixed, constant loads to determine their creep characteristics under long-term loading conditions. This report presents the results of measurements of rotational creep movement over a 7-yr period and completes the record of this research project.

The foundations were excavated with a 30-in. power auger; this was a very fast and economical method and it eliminated any need for concrete forms and backfilling. The fact that concrete was placed against undisturbed soil indicated that good resistance to overturning loads could be expected. A complete description of these foundations and soils tested is found in Highway Research Board Bulletin 247 (1960).

[illegible] of anchor-mounted steel poles, with a pre-[illegible] foundations was measured in two ways. One method consisted of transit readings on a scale mounted at the top of the pole and the other consisted of use of a clinometer on the top of the concrete foundation itself.

Measurements were made at irregular intervals for the first two years of loading after which no additional measurements were made until the final one in July 1965. The shape of the curves produced followed the anticipated pattern of relatively rapid movement initially and a steadily decreasing rate.

A record of the observations made throughout the test period is given in Tables 1 through 6. In general the two methods of measurement of the rotation agree very well for the organic soil, fairly well for the granular soil, and not very well for the plastic. No reason is known for the discrepancies encountered but the angles measured were very small. The data are shown graphically in Figure 1 in which the two independent measurements are first averaged and then smooth curves were drawn through the plotted points.

Although moments up to 195,000 ft-lb were applied to the foundations, angular movements of less than ½ deg were observed in the two soils (plastic and granular) considered to be good foundation material. In the very poor (organic) soil, the short foundation deflected almost 3 deg. This soil was tested only to obtain information, and not with the expectation that it would ever be considered suitable for supporting foundations.

Paper sponsored by Committee on Guardrails and Guide Posts and presented at the 45th Annual Meeting.

TABLE 1

MT. GILEAD

Date	Elapsed Time (days)	Rotation of Foundation (radians)		
		Clinometer on Top of Foundation	Transit Reading at Top of Pole	Average
Nov. 14, 1957	0	0.0000	0.0000	0.0000
Dec. 18, 1957	34	–	0.0009	0.0009
Jan. 28, 1958	75	0.0006	0.0021	0.0013
Mar. 7, 1958	113	0.0021	0.0026	0.0023
Apr. 23, 1958	160	0.0016	0.0034	0.0025
May 26, 1958	193	0.0011	0.0034	0.0022
July 22, 1958	250	0.0010	0.0038	0.0024
Mar. 4, 1959	475	0.0016	0.0043	0.0030
Apr. 28, 1959	530	0.0011	0.0043	0.0027
July 15, 1959	608	0.0010	0.0040	0.0025
Dec. 22, 1959	768	0.0008	0.0048	0.0028
July 8, 1965	2,793	0.0027	0.0122	0.0075

Soil: A-6-a (plastic)
Depth of foundation: 8.2 ft
Horizontal load: 6,000 lb.
Height of load: 24.4 ft
Moment at groundline: 146,000 ft-lb

TABLE 2

MT. GILEAD

Date	Elapsed Time (days)	Rotation of Foundation (radians)		
		Clinometer on Top of Foundation	Transit Reading at Top of Pole	Average
Nov. 14, 1957	0	0.0000	0.0000	0.0000
Dec. 18, 1957	34	–	0.0011	0.0011
Jan. 28, 1958	75	0.0011	0.0018	0.0015
Mar. 7, 1958	113	0.0012	0.0020	0.0016
Apr. 23, 1958	160	0.0011	0.0030	0.0022
May 26, 1958	193	0.0016	0.0032	0.0024
July 22, 1958	250	0.0012	0.0039	0.0026
Mar. 4, 1959	475	0.0019	0.0059	0.0039
Apr. 28, 1959	530	0.0011	0.0060	0.0036
July 15, 1959	608	0.0019	0.0048	0.0034
Dec. 22, 1959	768	0.0021	0.0060	0.0042
July 8, 1965	2,793	0.0074	0.0085	0.0080

Soil: A-6-a (plastic)
Depth of foundation: 12.0 ft
Horizontal load: 8,000 lb
Height of load: 24.4 ft
Moment at groundline: 195,000 ft-lb

TABLE 3

HOLMESVILLE

Date	Elapsed Time (days)	Rotation of Foundation (radians)		
		Clinometer on Top of Foundation	Transit Reading at Top of Pole	Average
Dec. 16, 1957	0	0.0000	0.0000	0.0000
Jan. 28, 1958	43	0.0017	0.0003	0 0010
Mar. 6, 1958	80	0.0030	0.0029	0.0030
Apr. 23, 1958	128	0.0034	0.0041	0.0038
May 27, 1958	162	0.0034	0.0056	0.0045
July 22, 1958	218	0.0036	0.0047	0.0044
Mar. 4, 1959	443	0.0048	0.0054	0.0051
Apr. 28, 1959	498	0.0052	0.0052	0.0052
July 16, 1959	577	0.0046	0.0065	0.0055
Dec. 22, 1959	736	0.0051	0.0074	0.0063
July 8, 1965	2,761	0.0083	0.0083	0.0083

Soil: A-2-6 (granular)
Depth of foundation: 8.0 ft
Horizontal load: 6,000 lb
Height of load: 24.4 ft
Moment at groundline: 146,000 ft-lb

HOLMESVILLE

Date	Elapsed Time (days)	Rotation of Foundation (radians)		
		Clinometer on Top of Foundation	Transit Reading at Top of Pole	Average
Dec. 16, 1957	0	0.0000	0.0000	0.0000
Jan. 28, 1958	43	0.0014	0.0009	0.0011
Mar. 6, 1958	80	Neg.	0.0008	0.0008
Apr. 23, 1958	128	0.0010	0.0019	0.0015
May 27, 1958	162	0.0005	0.0022	0.0014
July 22, 1958	218	0.0013	0.0022	0.0018
Mar. 4, 1959	443	0.0015	0.0023	0.0019
Apr. 28, 1959	493	0.0006	0.0025	0.0016
July 16, 1959	577	0.0013	0.0031	0.0022
Dec. 22, 1959	736	0.0011	0.0037	0.0024
July 8, 1965	2,761	0.0042	0.0025	0.0034

Soil: A-1-b (granular)
Depth of foundation: 12.3 ft
Horizontal lodd: 8,000 lb
Height of load: 24.3 ft
Moment at groundline: 194,000 ft-lb

TABLE 5

MANSFIELD

Date	Elapsed Time (days)	Rotation of Foundation (radians)		
		Clinometer on Top of Foundation	Transit Reading at Top of Pole	Average
Oct. 9, 1957	0	0.0000	0.0000	0.0000
Oct. 14, 1957	5	–	0.0016	0.0016
Jan. 28, 1958	111	0.0030	0.0062	0.0046
Mar. 6, 1958	148	0.0148	0.0176	0.0162
Apr. 23, 1958	196	0.0165	0.0212	0.0188
May 27, 1958	230	0.0171	0.0208	0.0189
July 22, 1958	286	0.0188	0.0228	0.0208
July 15, 1959	644	0.0278	0.0334	0.0306
Dec. 22, 1959	804	0.0308	0.0362	0.0335
July 8, 1965	2,829	0.0471	0.057	0.052

Soil: A-7-6 (organic)
Depth of foundation: 7.9 ft
Horizontal load: 1,000 lb
Height of load: 24.1 ft
Moment at groundline: 24,100 ft-lb

TABLE 6

MANSFIELD

Date	Elapsed Time (days)	Rotation of Foundation (radians)		
		Clinometer on Top of Foundation	Transit Reading at Top of Pole	Average
Dec. 17, 1957	0	0.0000	0.0000	0.0000
Jan. 28, 1958	42	0.0049	0.0060	0.0054
Mar. 6, 1958	79	0.0086	0.0108	0.0097
Apr. 23, 1958	127	0.0100	0.0112	0.0106
May 27, 1958	161	0.0102	0.0110	0.0106
July 22, 1958	217	0.0111	0.0118	0.0115
July 15, 1959	575	0.0148	0.0171	0.0160
Dec. 22, 1959	735	0.0162	0.0186	0.0174
July 8, 1965	2,760	0.0241	0.025	0.024

Soil: A-7-5 (organic)
Depth of foundation: 12.0 ft
Horizontal load: 3,000 lb
Height of load: 24.2 ft
Moment at groundline: 72,600 ft-lb

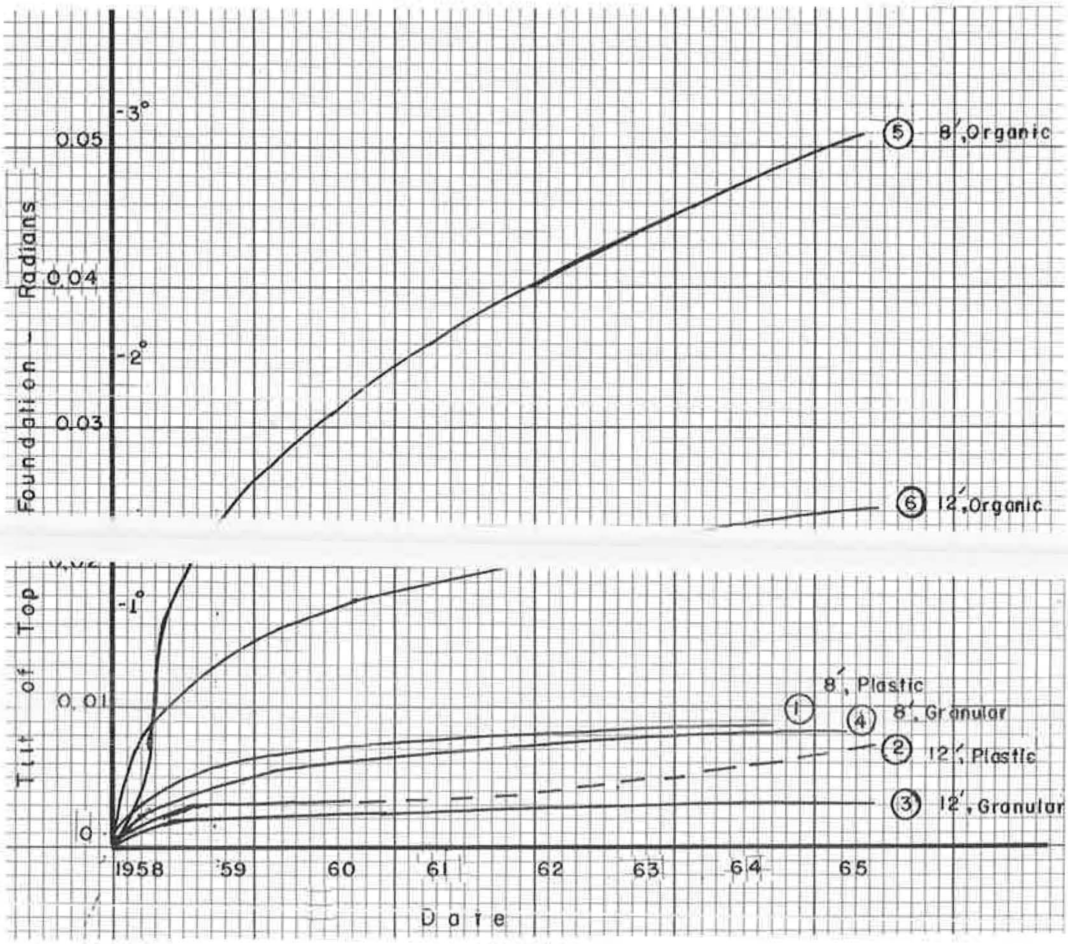

Figure 1. Results of long-term tests.