An Investigation of Factors Affecting the Design Location of Freeway Ramps

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•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;

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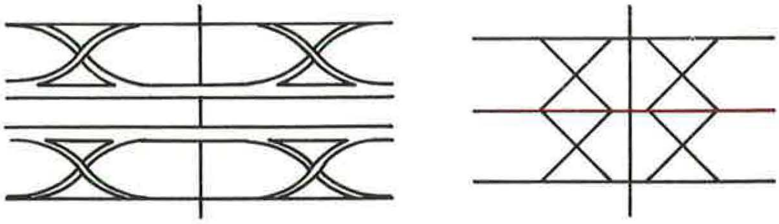

Figure 1. Stacked ramps.

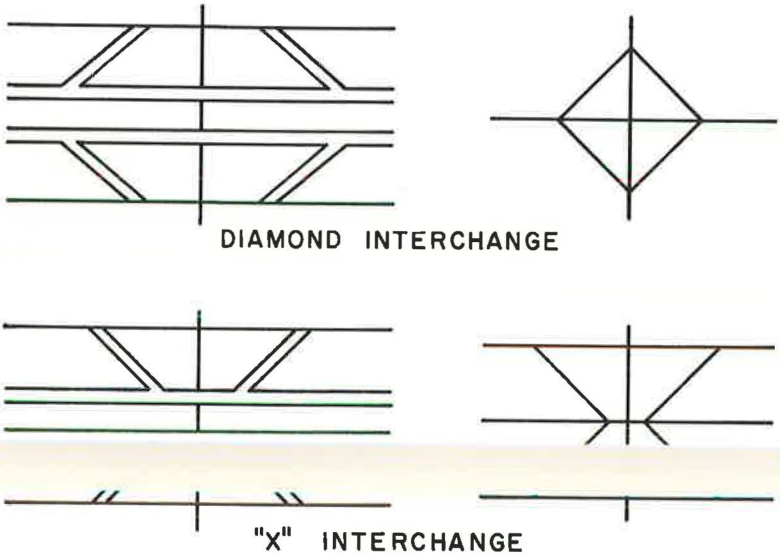

Figure 2. Interchange types.

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.

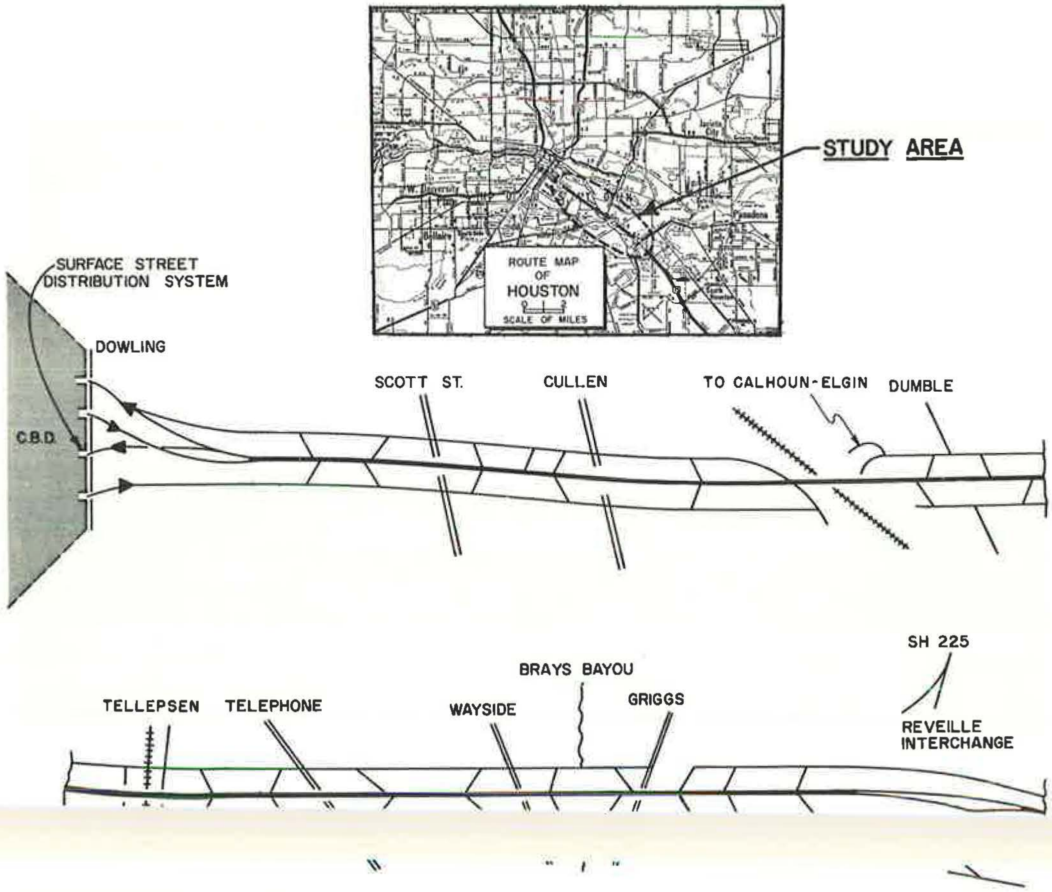

Figure 4. Study area—Gulf Freeway, Houston, Texas.

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

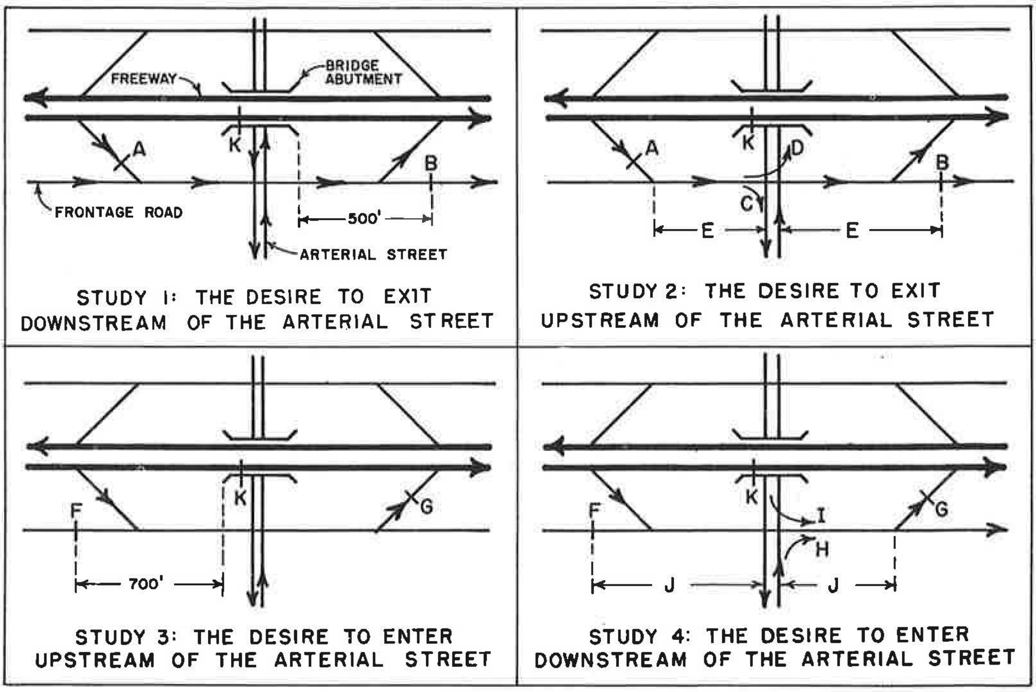

Figure 5. License plate recording points.

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.

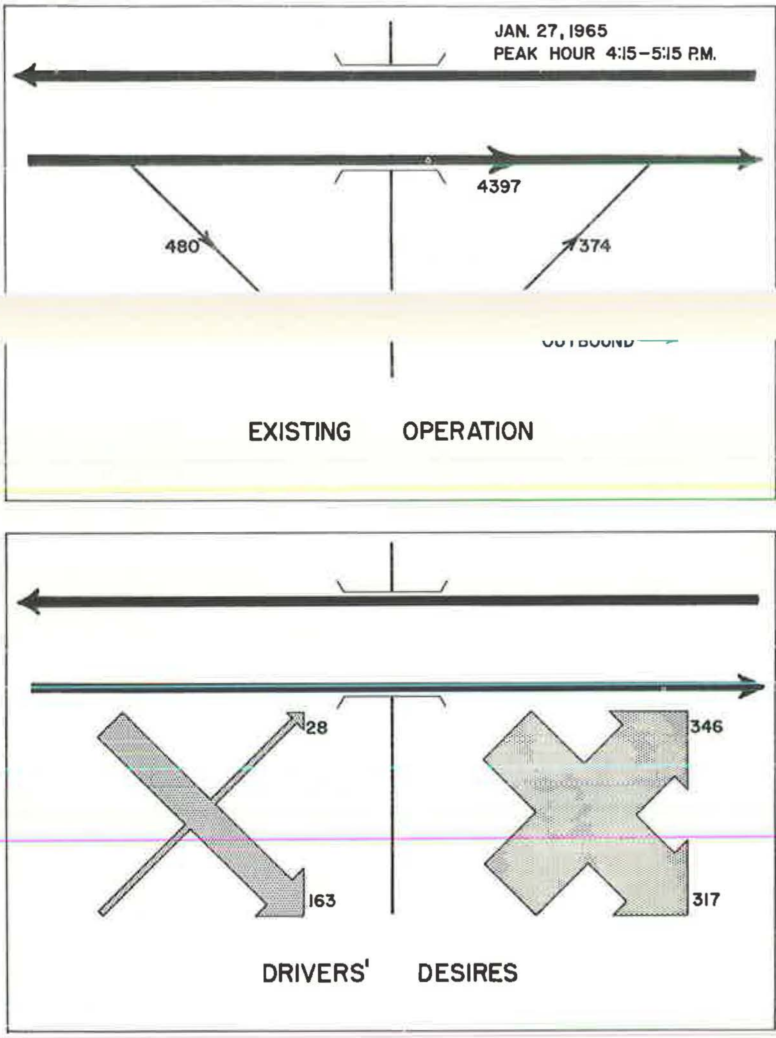

Figure 6. Cullen Interchange outbound.

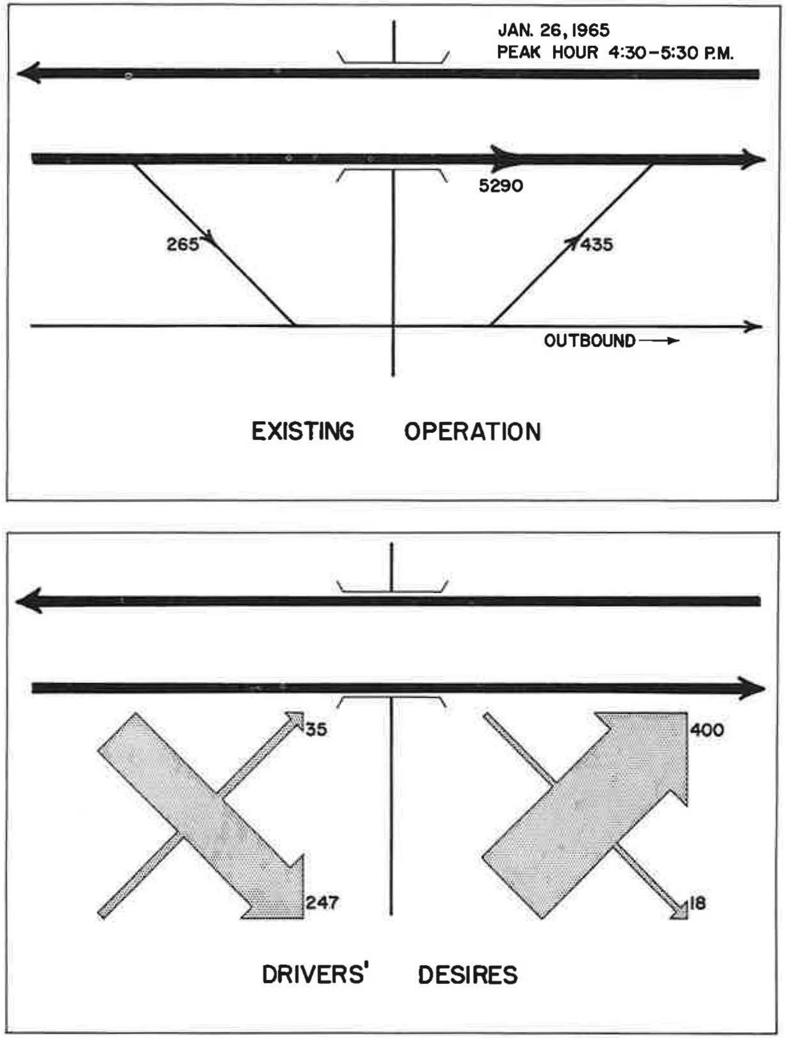

Figure 7. Telephone Interchange outbound.

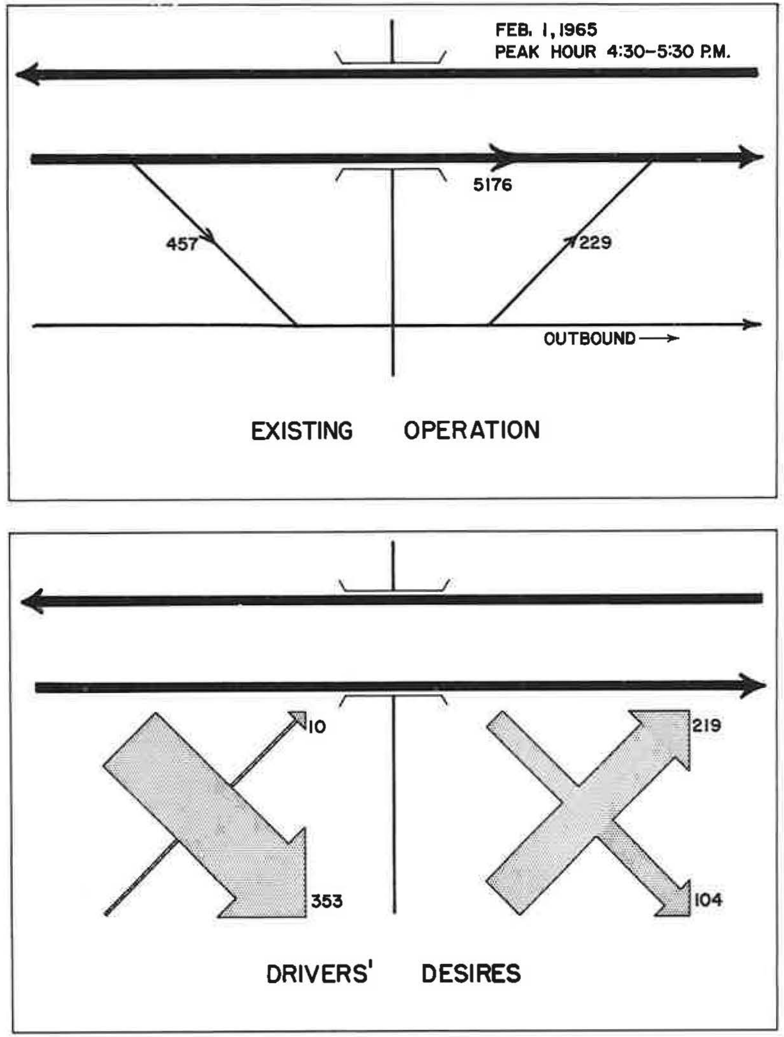

Figure 8. Woodridge Interchange outbound.

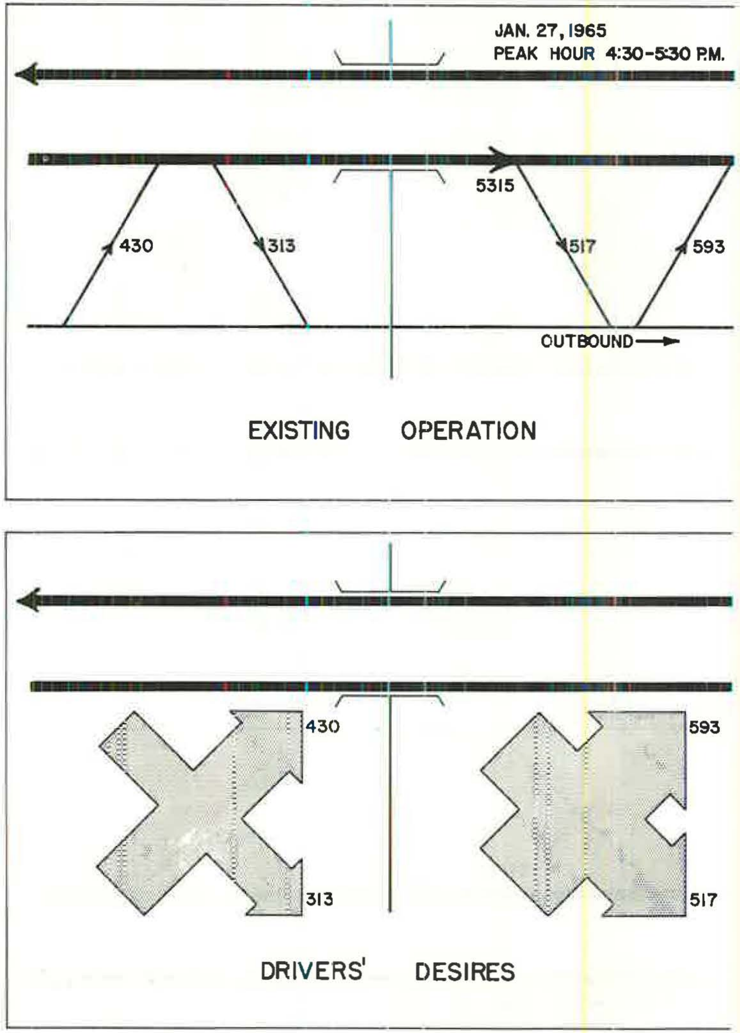

Figure 9. Wayside Interchange outbound.

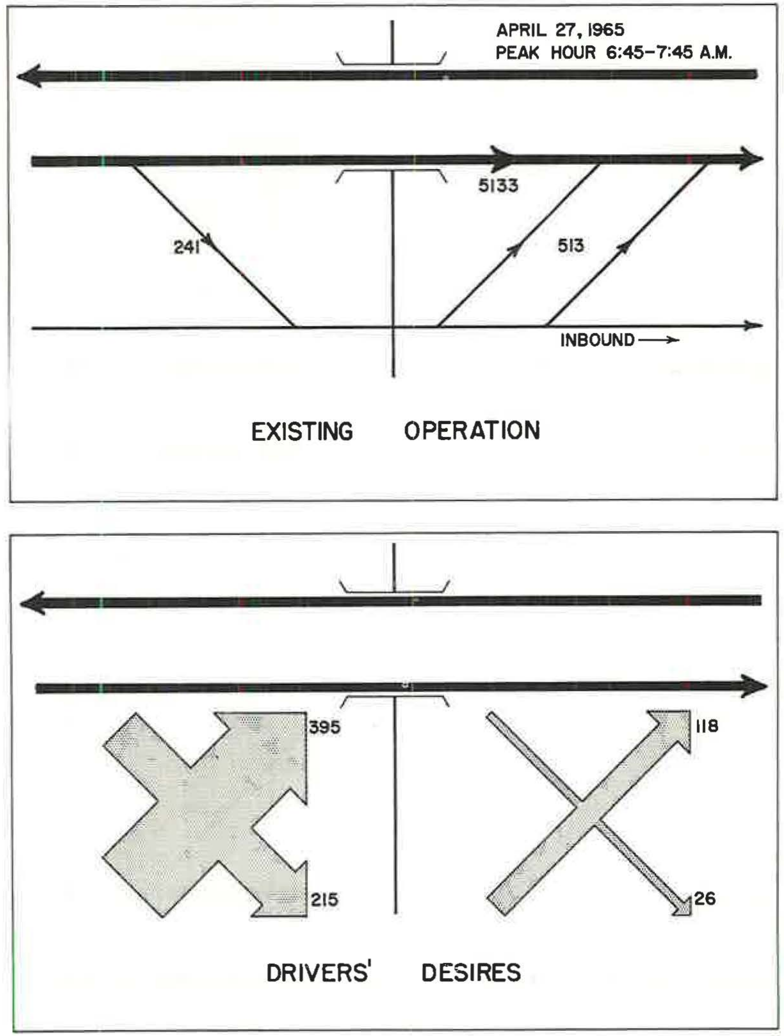

Figure 10. Cullen Interchange inbound.

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

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

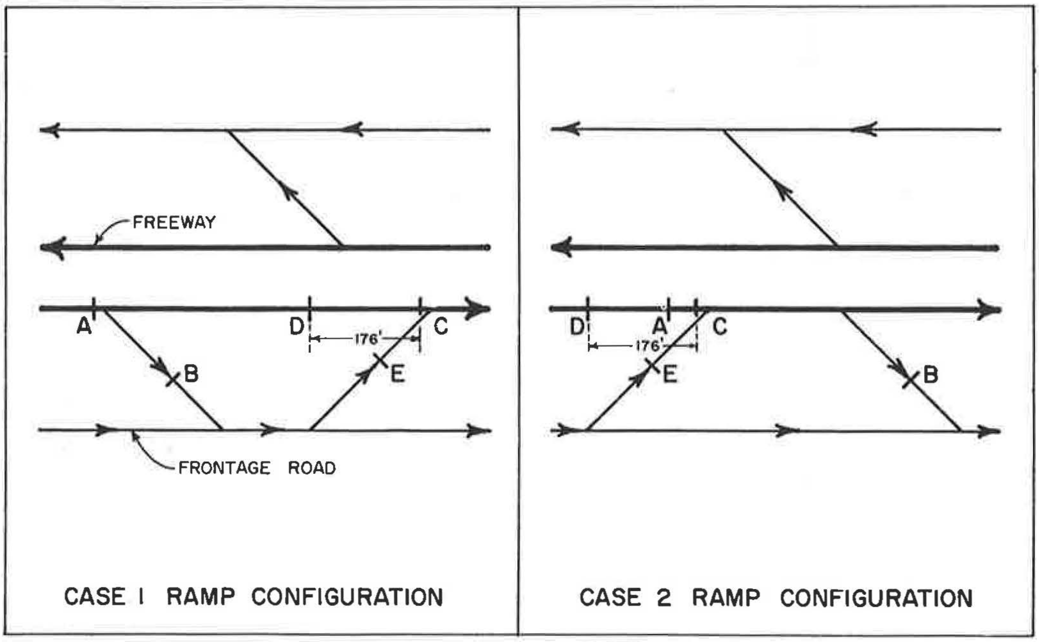

Figure 11. Data collection points.

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.

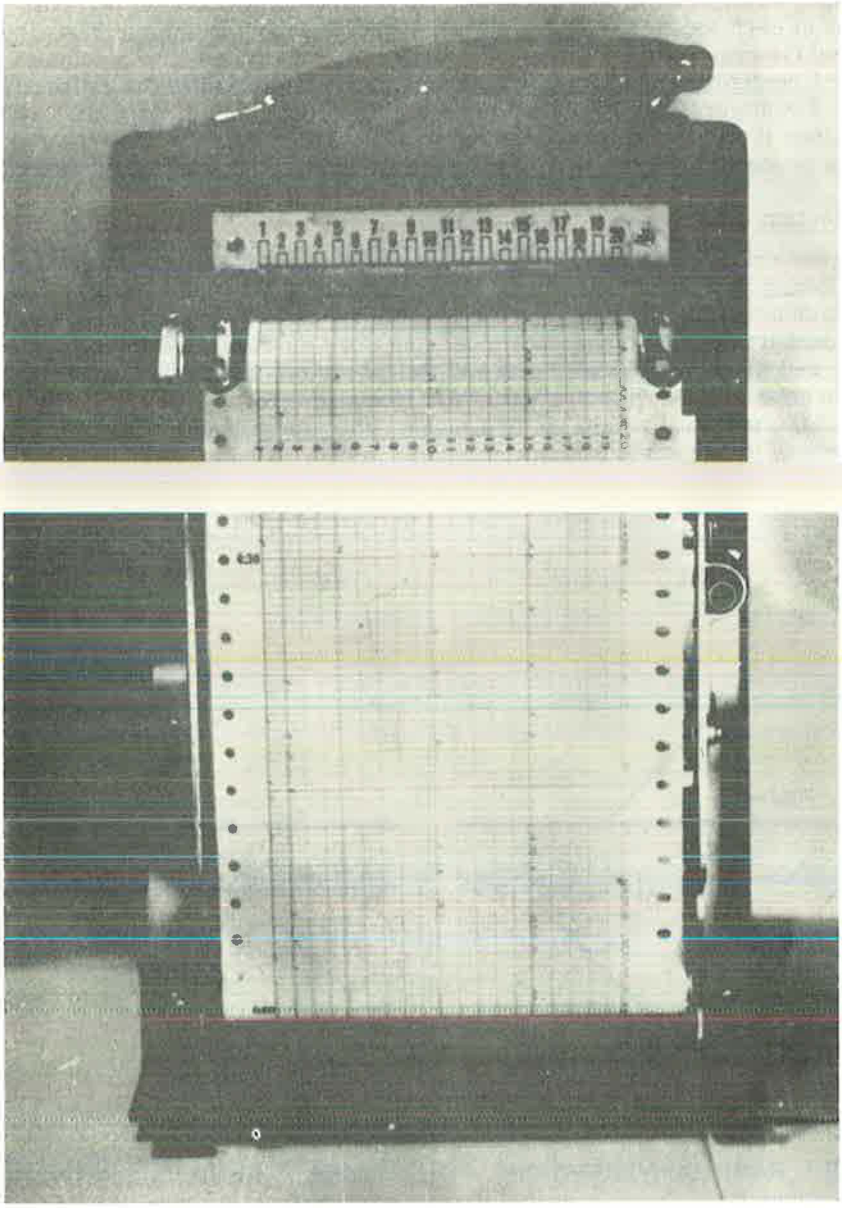

Figure 12. Esterline-Angus 20-pen recorder.

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	2.70[a]	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

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.

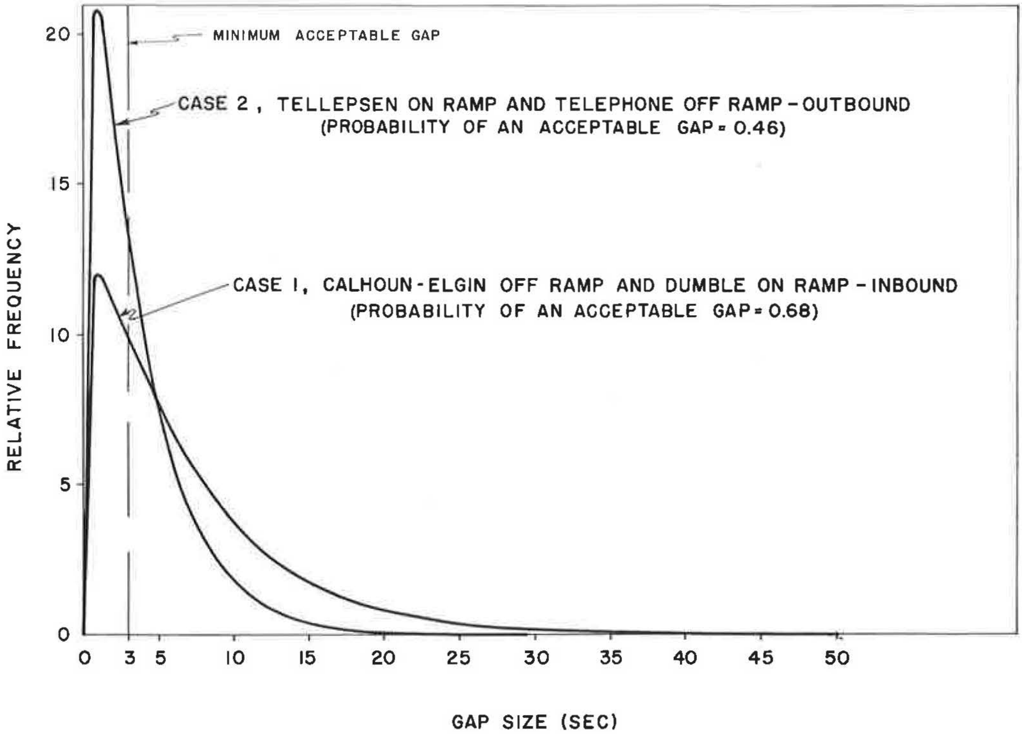

Figure 13. Comparison 1 of acceptable gap probability.

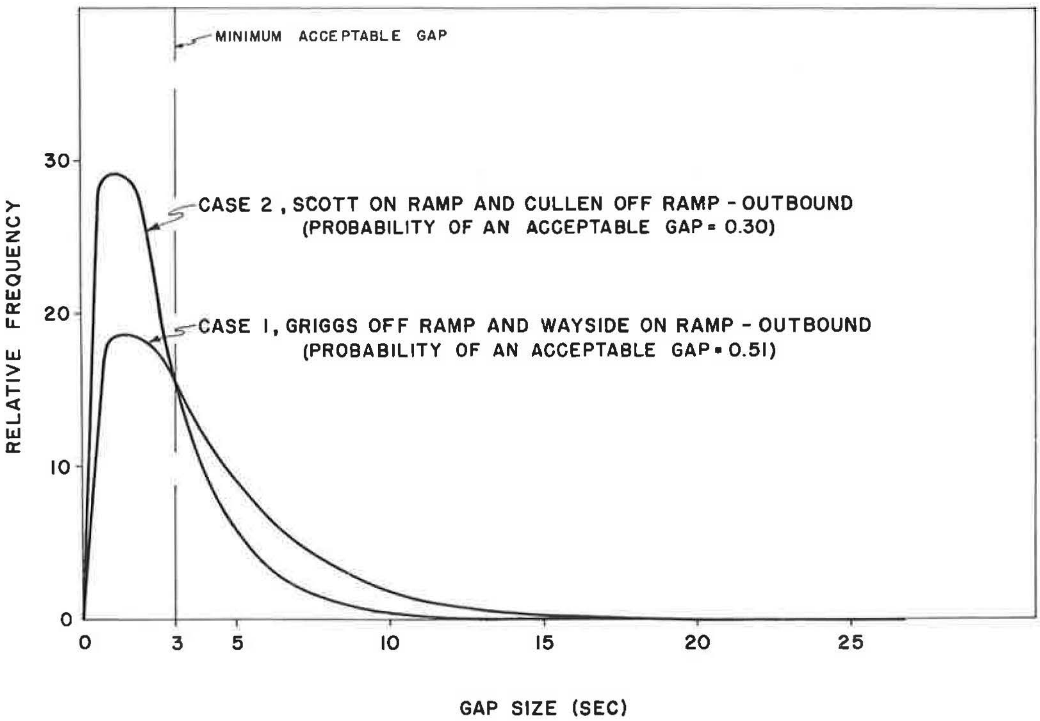

Figure 14. Comparison 2 of acceptable gap probability.

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.

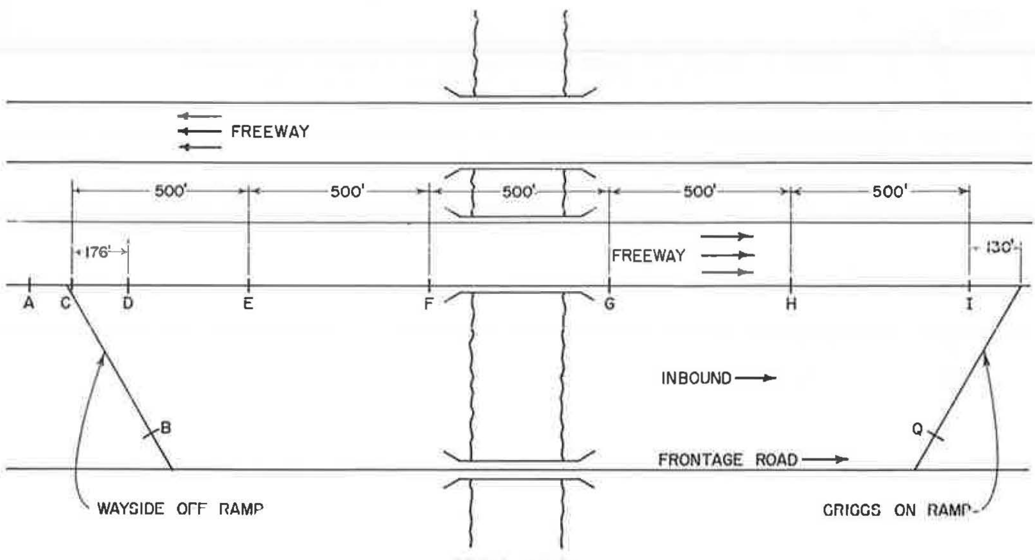

Figure 15. Freeway ramp spacing study location.

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.

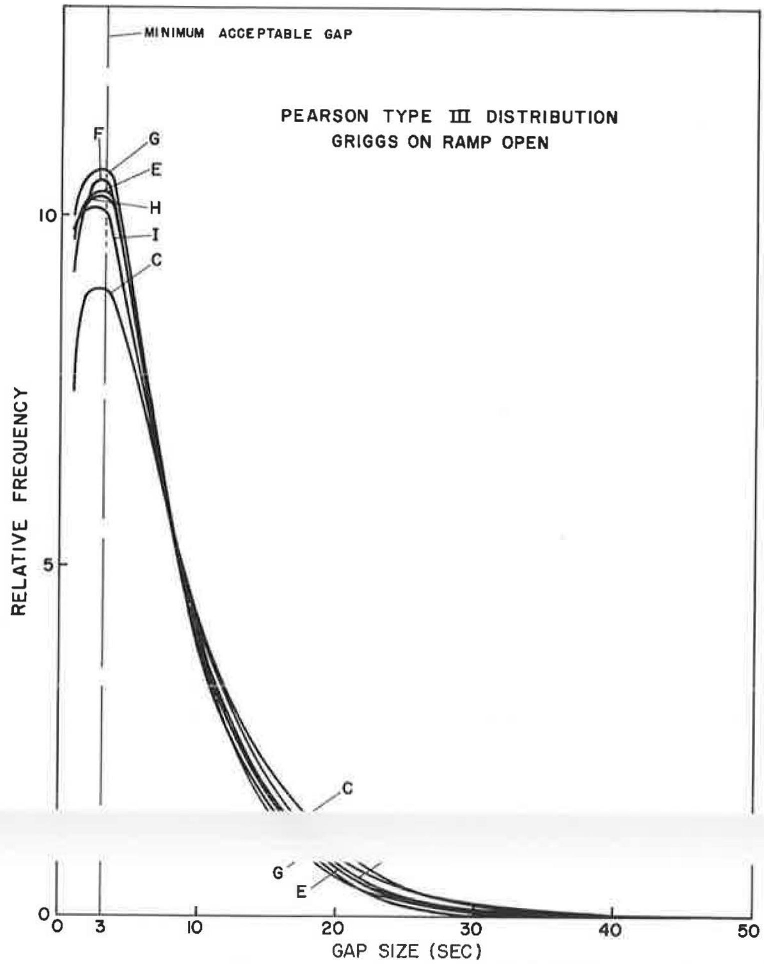

Figure 16. Gap distributions, Brays Bayou, 1:30-3:00 PM, Jan. 25.

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

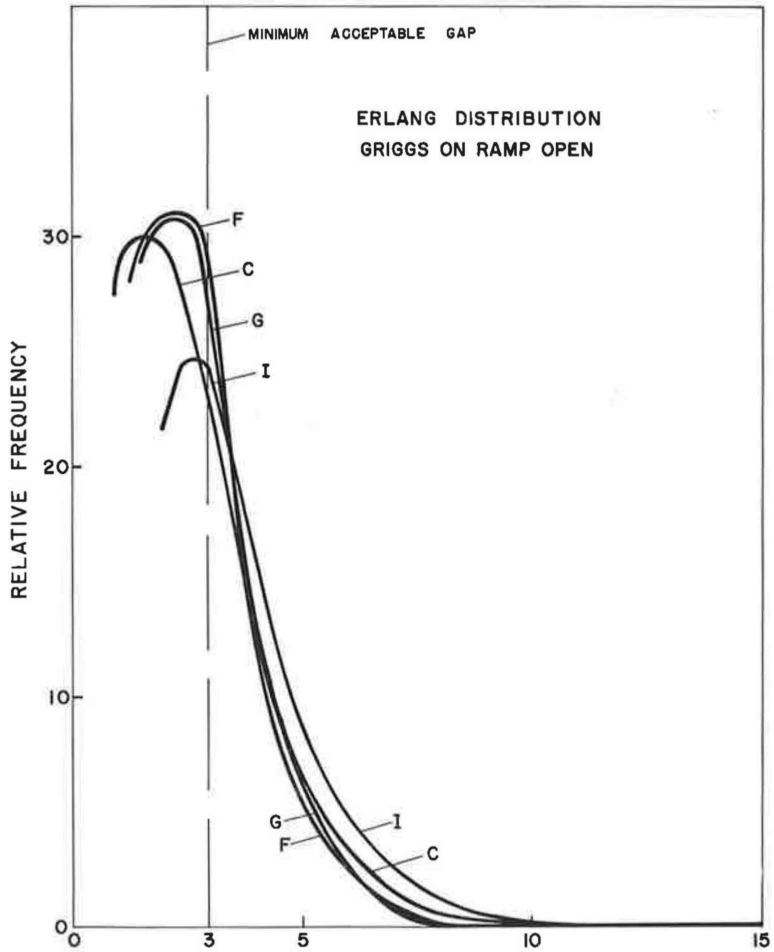

Figure 17. Gap distributions, Brays Bayou, 7:20-7:25 AM, Feb. 18.

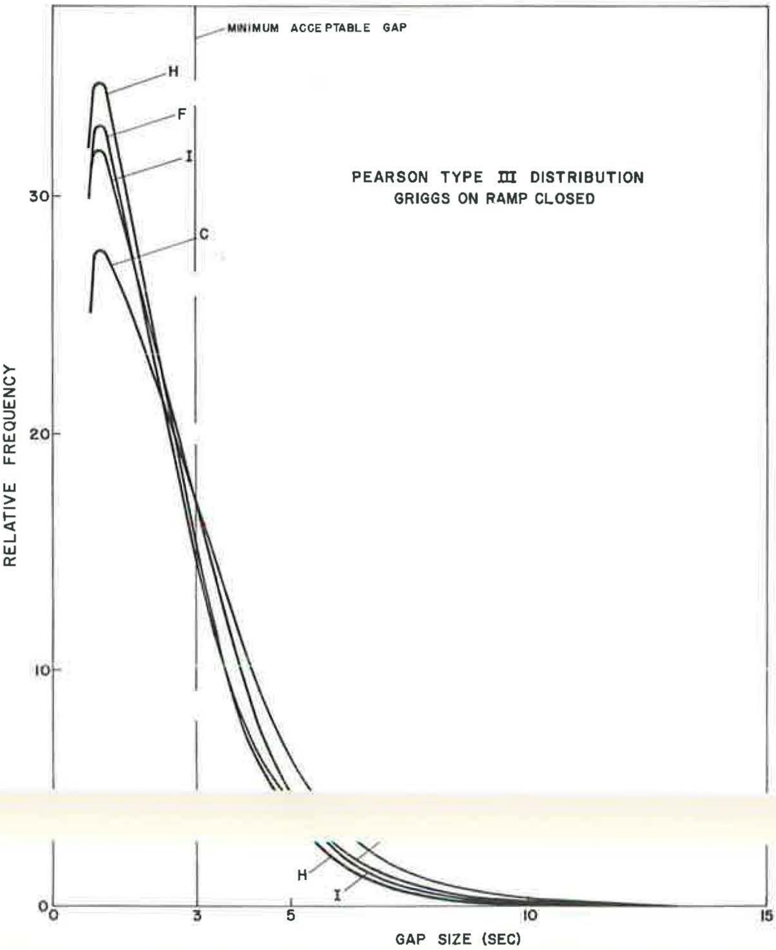

Figure 18. Gap distributions, Brays Bayou, 7:05-7:10 AM, Feb. 18.

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

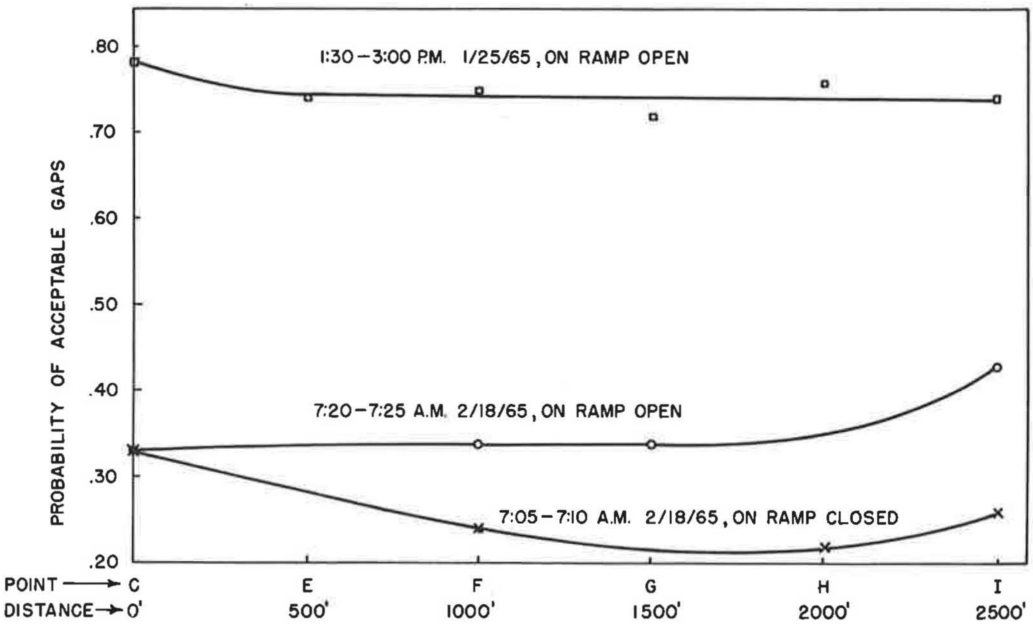

Figure 19. Effect of distance between ramps on acceptable gap probability.

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

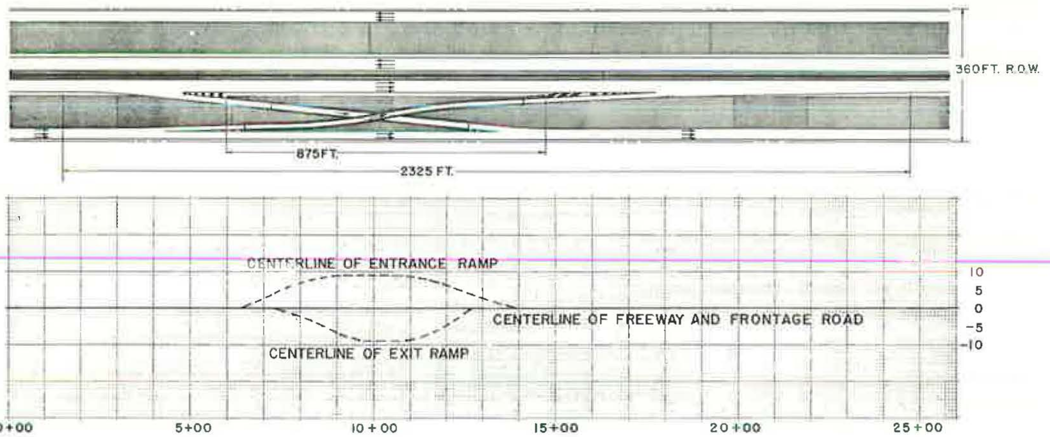

Figure 20. Plan profile of stacked ramps.

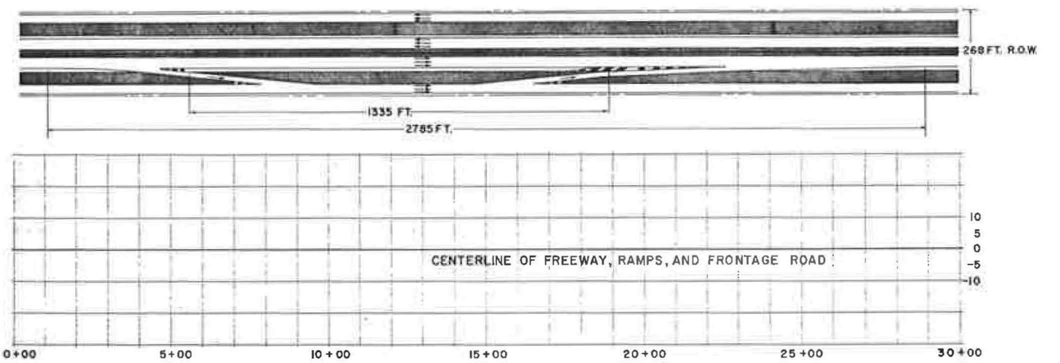

Figure 21. Plan profile at an off-ramp upstream of an on-ramp.

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.

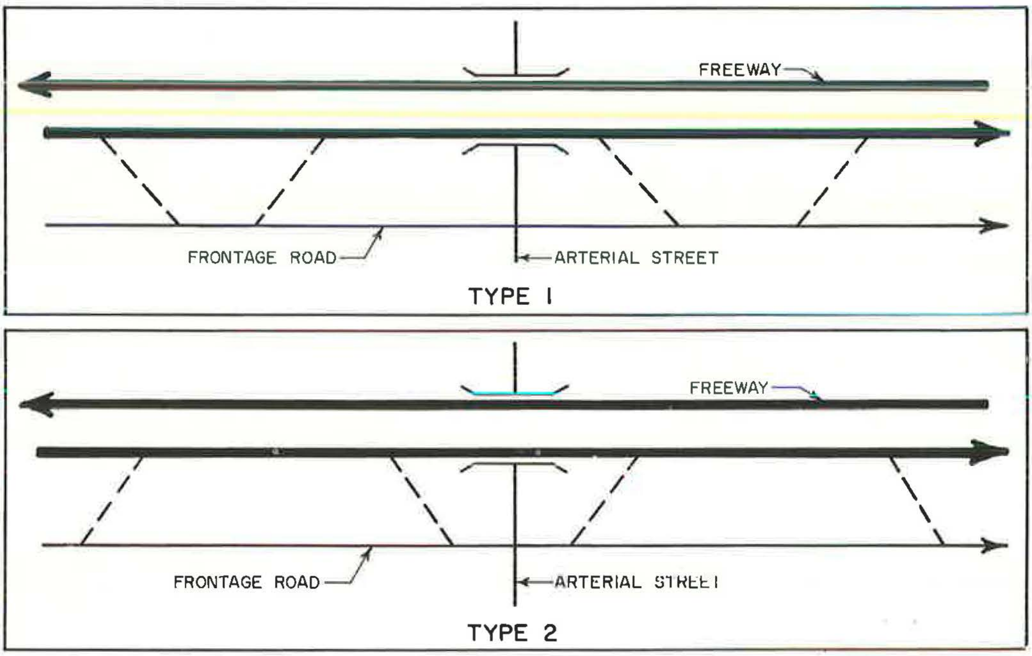

Figure 22. Types of interchange layouts.

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

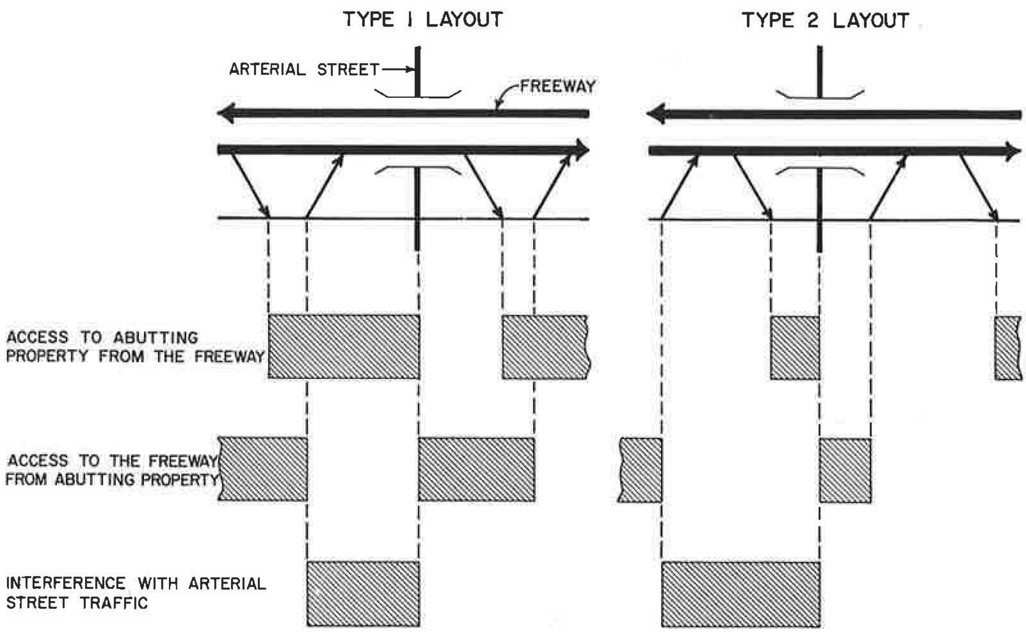

Figure 23. Some effects of interchange layouts.

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.

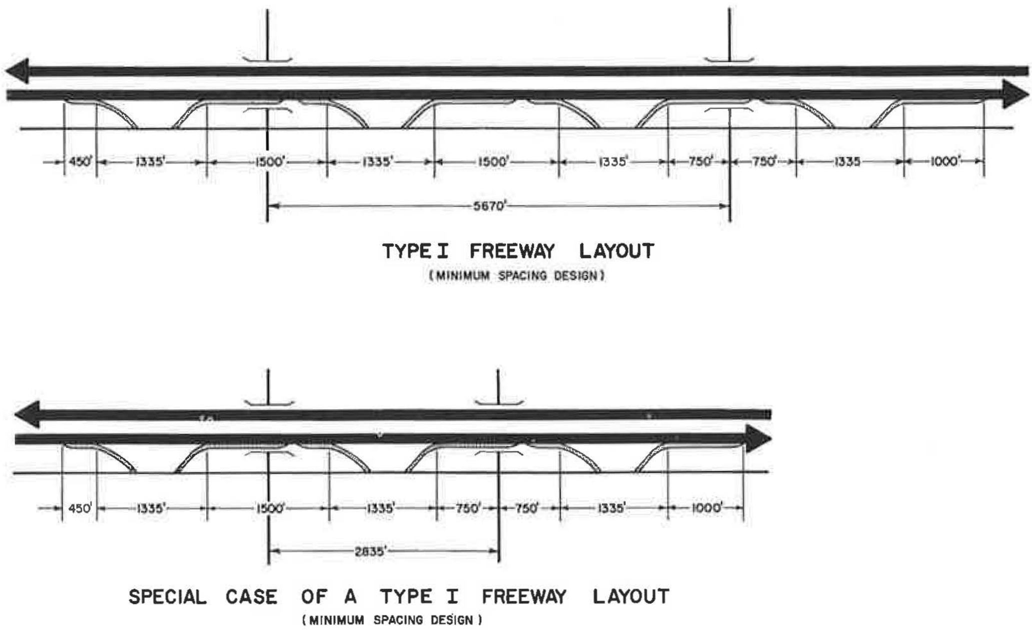

Figure 24. Type I freeway layouts.

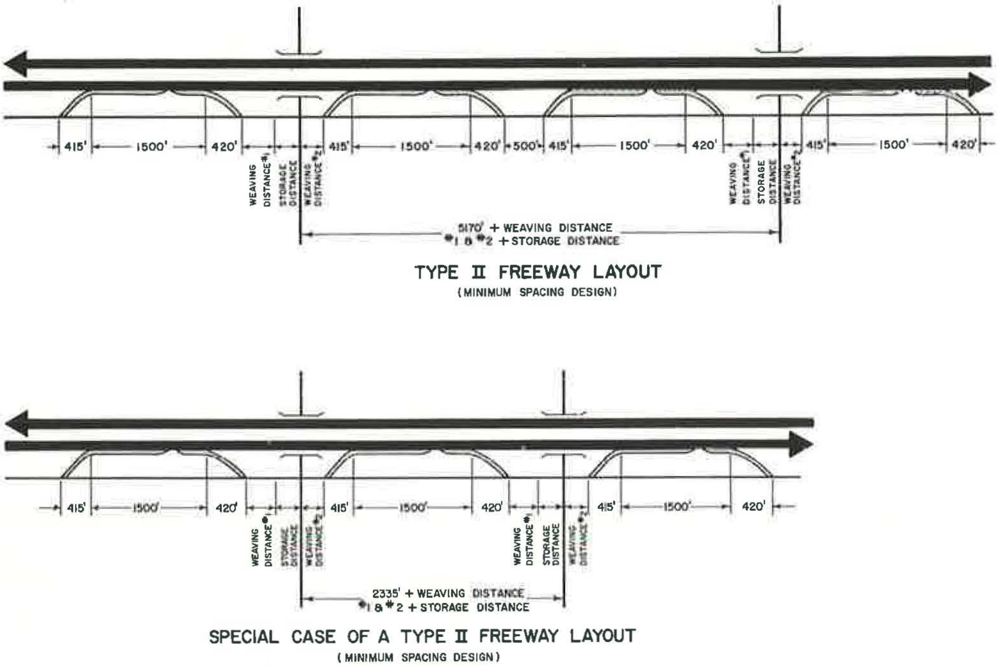

Figure 25. Type II freeway layouts.

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

The Type II freeway layout and its special case [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.

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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

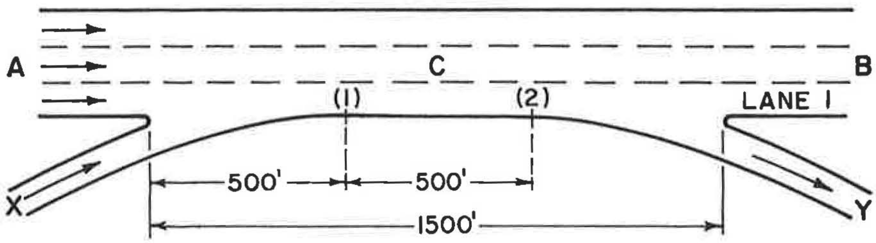
TABLE 14

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.