EVALUATION OF TWO-LANE RAMP MERGES ON URBAN FREEWAYS

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ABSTRACT

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By

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A study of two-lane ramp merges on urban freeways was made in an effort to analyze the design and operational characteristics of this type of merging situation.

Sixteen millimeter color movie films were taken at four locations in the Detroit area. Vehicle path distributions were taken off the film by means of observing vehicles on a projected grid layout.

Given projected traffic volumes, a method of analyzing the capacity or traffic carrying ability of two-lane ramp merges was developed based on the distribution of approaching ramp and mainline traffic.

Since only one lane is added to the freeway beyond a two-lane merge, it is necessary to eliminate one of the ramp lanes. This study indicates that when the inside lane is dropped, traffic makes more efficient use of the total merge area. Further research is needed, however, to verify this conclusion.

Two of the two-lane ramp merges studied have been signed down to one lane because of operational problems created by the design of these merges. The signing proved to be 90% effective; however, the capacity of the merge is very limited.

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CHAPTER I

INTRODUCTION

The purpose of this thesis is to evaluate the capacity, design and operation of two-lane ramp merges on urban freeways. In the Detroit area there are in existence a number of high-speed directional freeway-to-freeway interchanges which contain one or more two-lane entrance ramps. Very high volumes can be carried on two-lane ramps. To handle these high volumes, a freeway lane is generally added to the freeway laneage at the end of the merge. This thesis will include only those two-lane ramp merges where a lane is added to the freeway beyond the merge.

Since very little research has been carried out in this area of freeway operations, detailed analysis of capacity, geometric design and operation is very difficult.

The following problems will be analyzed and discussed in detail:

1. In the early design stages of urban freeways it is necessary to evaluate the ability of the freeway to carry projected traffic volumes at a specified level of service. Any merge point is a potential bottleneck and must therefore be analyzed to determine if the projected total

merging traffic is sufficient to cause severe congestion and possible stoppage of the freeway. Given projected traffic volumes, a method is therefore needed to determine the capacity or traffic carrying ability of two-lane ramp merges.

2. Since only one lane is added to the freeway beyond a two-lane merge, it is necessary to eliminate one of the ramp lanes. There has been considerable controversy for a number of years as to whether the outside or inside lane should be eliminated. An attempt will be made to provide a rationale for determining the proper course of action.

3. Two of the two-lane ramps studied have been signed down to one lane even though a two-lane merge was constructed. Operational problems were experienced after construction due to the approach ramp and freeway geometrics. This section will evaluate the effectiveness of the signing and the effect on the capacity potential of the merge.

In order to analyze the operation of two-lane ramp merges, 16mm color movie films were taken at four locations in the Detroit area. Film analysis of vehicle paths provided the data needed to accomplish the goals of this thesis.

CHAPTER II

REVIEW OF LITERATURE

A complete review of available studies indicates that very little extensive research has been carried out on the subject of two-lane ramp merges.

In 1959 C. J. Keese, C. Pinnell, and W. R. McCasland of the Texas Transportation Institute (6) presented a preliminary report, at the 38th Annual Meeting of the Highway Research Board, on freeway entrance ramp operations which included one two-lane merge. Vehicle paths for 713 vehicles were analyzed indicating that minor use was made of the ramp as a two-lane facility. No other conclusions were documented.

Joseph W. Hess (4) in 1963 published findings of a nationwide study sponsored by the Bureau of Public Roads and the Highway Research Board concerning "Capacities and Characteristics of Ramp Connections." Two-lane ramp operations (both on and off types) comprised 42 of the 219 separate studies submitted. However, the two-lane ramp operations varied so widely in both geometrics and traffic characteristics that no capacity formulas could be determined. Mr. Hess did conclude that an extra downstream freeway lane should be added beyond the merge.

The 1965 Highway Capacity Manual (3) published by the Highway Research Board contains procedures for analyzing all types of highway capacity problems. In the chapter on ramps, two-lane ramp merges are discussed. Four different cases are presented to analyze four different designs.

> Case I--This design requires the addition of a freeway lane and provides the outside ramp lane with direct entry into the added freeway lane. The inside ramp lane must merge into lane 1 of the freeway or into the outside ramp lane. Research results regarding performance are not yet available, estimates therefore are necessary.

The Capacity Manual suggests that for Case I the outside lane carry the bulk of the traffic up to its capacity. The remainder of the traffic will be in the inside ramp lane and will enter lane 1 of the freeway as if it were a single-lane entrance ramp.

> Case II--This design also requires the addition of a freeway lane. In this case, however, the inside ramp lane is led directly into the added freeway lane the outside ramp lane is expected to merge with the inside ramp lane. Again, research results are unavailable. A general computational method for this type of design cannot be suggested, inasmuch as marking practices can affect the paths followed by ramp drivers.

Cases III and IV do not require an added freeway lane and will, therefore, not be discussed. Research is also lacking for both of these cases.

"A Policy on Geometric Design of Rural Highways" (2) published by The American Association of State Highway Officials virtually ignores two-lane ramp merges. They do point out some general capacity limitations for the ramp proper which may or may not be critical, since the merging point with the freeway will generally control the amount of traffic which can be handled on the ramp.

"A Policy on Arterial Highways in Urban Areas" (1), also published by the American Association of State Highway Officials, states:

> In conjunction with entrances bringing 2 lanes of traffic into a highway, the highway beyond the ramp entrance should be at least one lane wider than the highway approaching the entrance.

In a 1968 issue of Traffic Engineering, two-lane entrance ramps are discussed by Ronald C. Pfefer (8) based on conclusions by the 5-F Committee of the Institute of Traffic Engineers. While a considerable amount of data was obtained by the committee, Mr. Pfefer states:

> It should be emphasized, however, that the conclusions are not the result of extensive research and should not be considered as such. The need for basic data to establish specific design criteria is evident. While capacity is the common warrant for a two-lane ramp, there is a lack of available data on which to base capacity analysis.
>
> The current use of three merging lane configurations further complicates the problem. In general, however, capacity must be checked at four points to assure adequate design; on the ramp proper, at the diverge, at the merge and on the freeway beyond the merge.

Mr. Pfefer's article includes a simplified procedure, developed by J. E. Lelsch, for analyzing inner-lane merge designs based on the 1965 Highway Capacity Manual. It assumes, as previously discussed in Case I of the Highway Capacity Manual, that the outside ramp lane operates directly into the added freeway lane and carries the bulk of the ramp traffic. Using the Capacity Manual procedure, the inside ramp lane is analyzed as a normal single-lane ramp merge. The two values are added to compute the total ramp volume. The article goes on to discuss geometric design and operational considerations when determining whether to use the inner lane merge or outer lane merge. Mr. Pfefer further states:

> Highway agencies indicate varying design standards for the merge. In most instances, however, a direct taper of 50:1 or 600 ft. was utilized. Standard ramp width was 24 ft.

The committee concluded that the ramp lane carrying the greater share of the traffic should be carried directly into the added freeway lane. This is especially true if the ramp is short and traffic has very little time to maneuver and change lanes.

CHAPTER III

METHOD OF STUDY

When considering a method of study to evaluate two-lane merges, it is immediately evident that a complex operation is taking place. In comparison with a single-lane merge, where the ramp vehicle is faced with a simple task of finding a large enough gap in lane 1 to fit into, drivers in the ramp lanes of two-lane merges have the option of either proceeding straight ahead into the added lane or merging into freeway traffic. These movements are coupled with that traffic in the outside lane of the freeway which desires to shift into the added lane to gain more freedom of movement or to exit the freeway.

Data Source

In order to document the various movements taking place in the merge, it was felt that a permanent record was needed to insure a complete and accurate data source. It was therefore decided to use 16mm color movie films taken at 8 frames per second. At each of the locations, four reels of film totaling approximately one-half hour in time were taken during the peak hour and four reels during an off peak hour. All filming was done during the summer in good weather, on a normal working day.

The camera was set up on the top platform of a maintenance truck which was parked at a location providing good visibility of the merging area. It was generally found that setting up well back in the gore area of a merge and shooting downstream provided the most distortion-free films which were the easiest to analyze. Before filming, each merge was marked with orange cones and orange paint to provide a reference point every 100 feet.

Locations

Figure 1 is a general area map of the Detroit Metropolitan Area and shows the four locations which were utilized in this study. Although there are more than four two-lane merges in the Detroit area, the ones chosen were constructed in recent years and therefore are considered to be representative of newer design practices. Furthermore, economic considerations prevented filming any more than four locations. The locations and pertinent data are shown on Figures 2 through 5. For convenience, they will be referred to in the following manner:

1. US-10 merge
2. M-39 merge
3. I-75 merge
4. I-94 merge

It should be noted that throughout this thesis lane A refers to the outside ramp lane, lane B the ramp lane closest to the freeway, and lane 1 is the outside freeway lane.

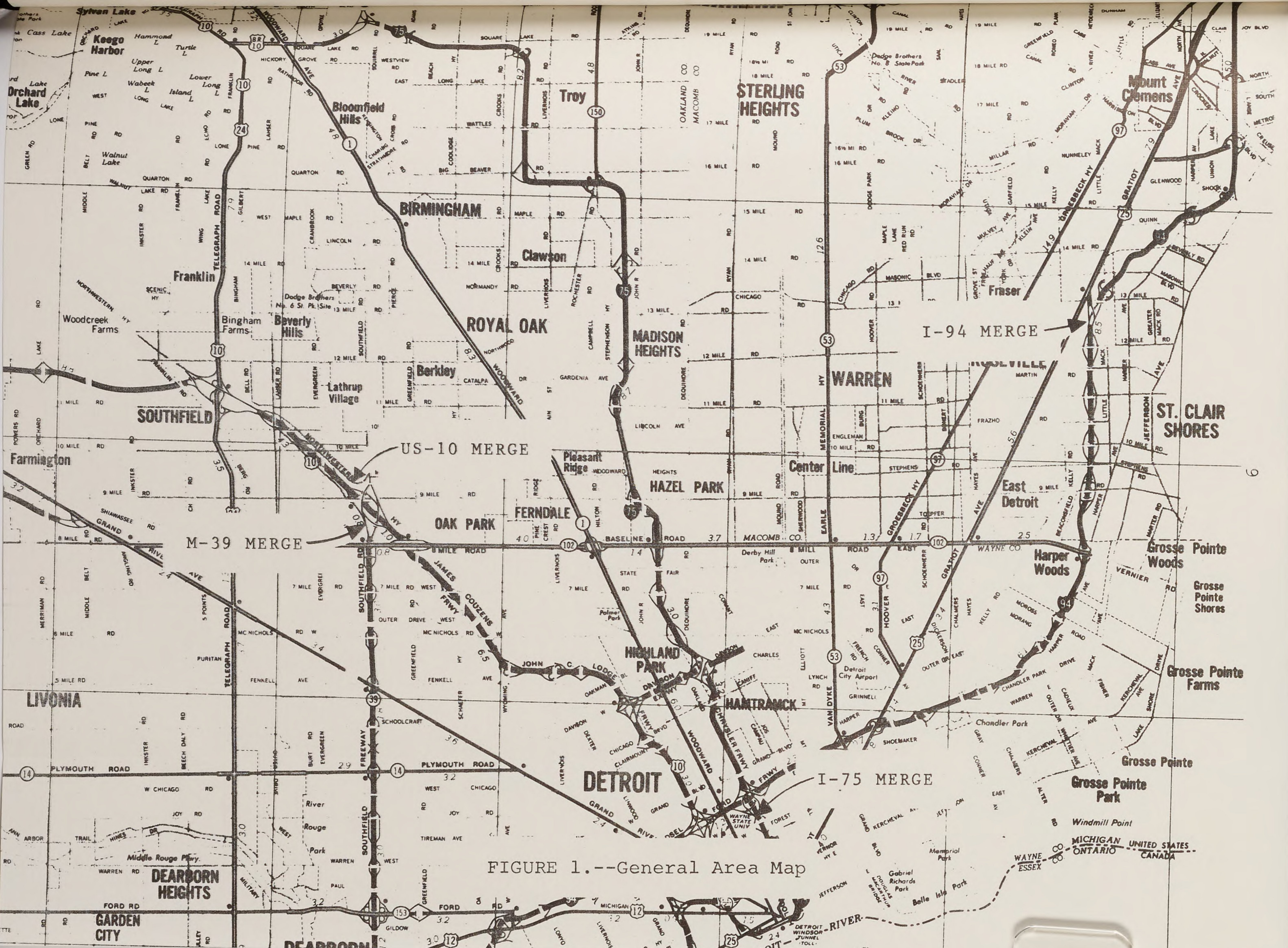
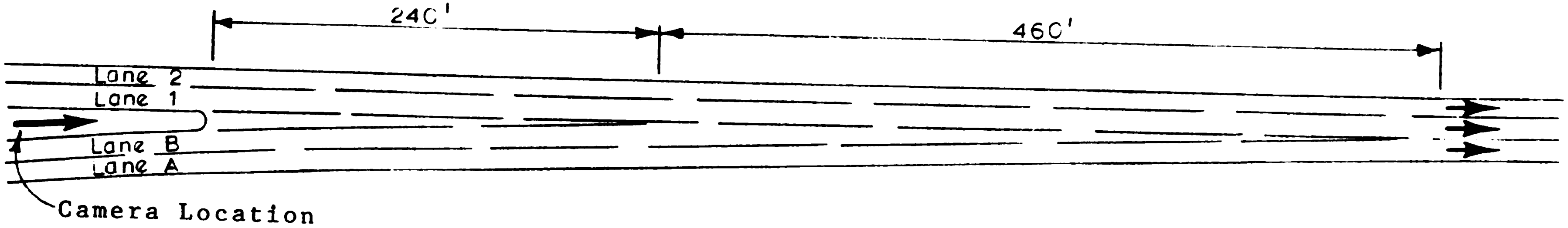


FIGURE 1.--General Area Map



Ramp Lane Width	12'
Freeway Lane Width	12'
Angle of Convergence	2.5°
Ramp Grade at Nose	+1.63%
Freeway Grade at Nose	+1.70%
Freeway Curvature at Nose	Tangent
Ramp Curvature at Nose	4°
Distance to downstream off-ramp	3280'

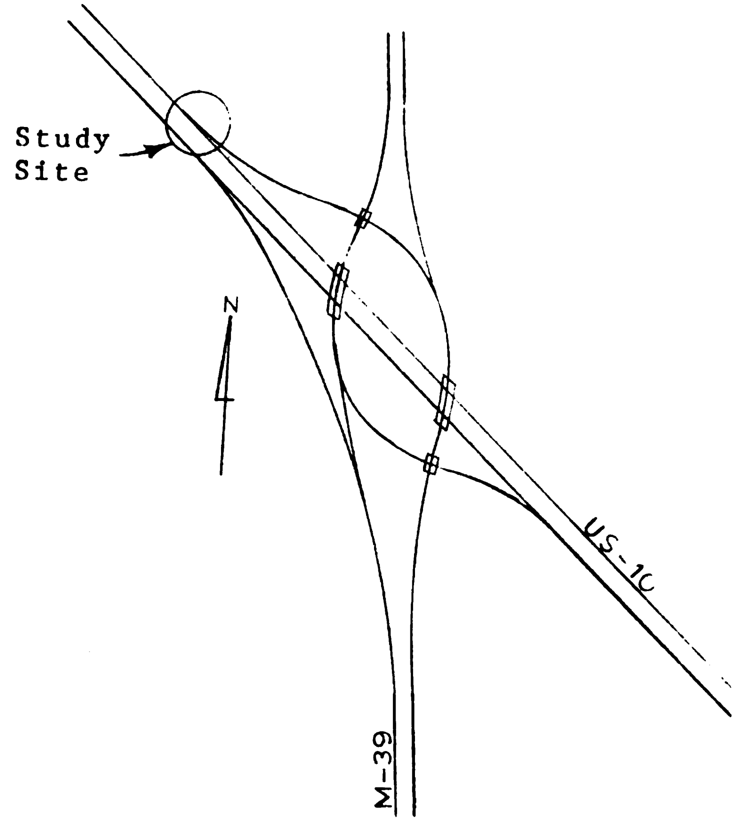
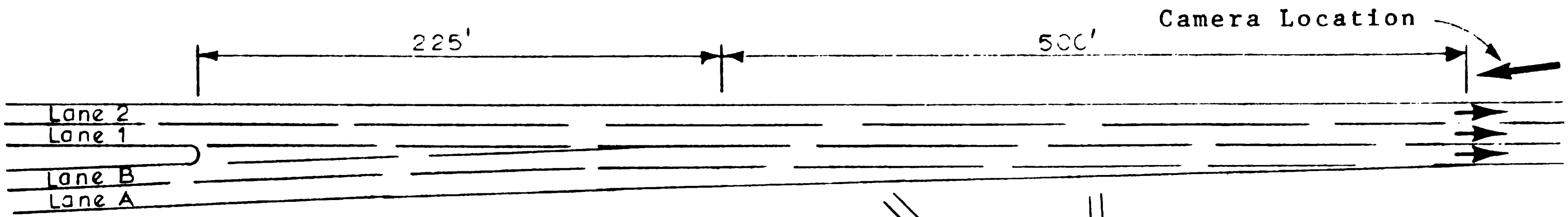


FIGURE 2.--US-10 Merge: Geometric Data.



Ramp Lane Width	12'
Freeway Lane Width	12'
Angle of Convergence	1.5°
Ramp Grade at Nose	-1.72%
Freeway Grade at Nose	-1.82%
Freeway Curvature at Nose	Tangent
Ramp Curvature at Nose	2°
Distance to downstream off-ramp	2662'

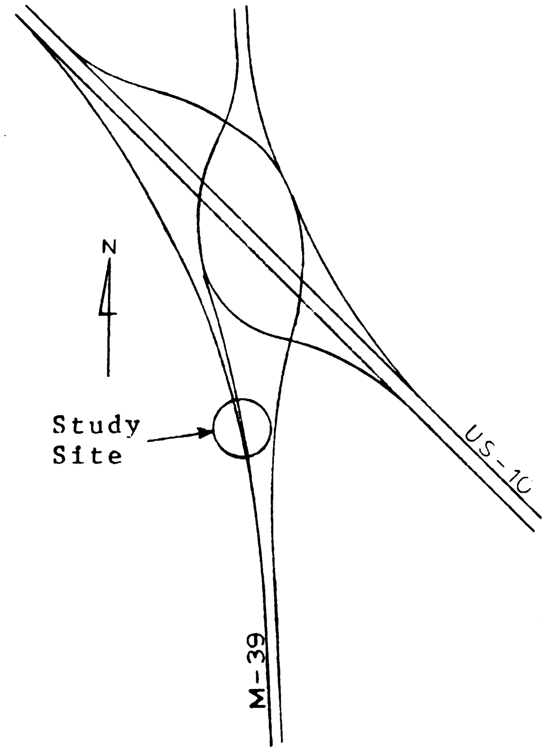
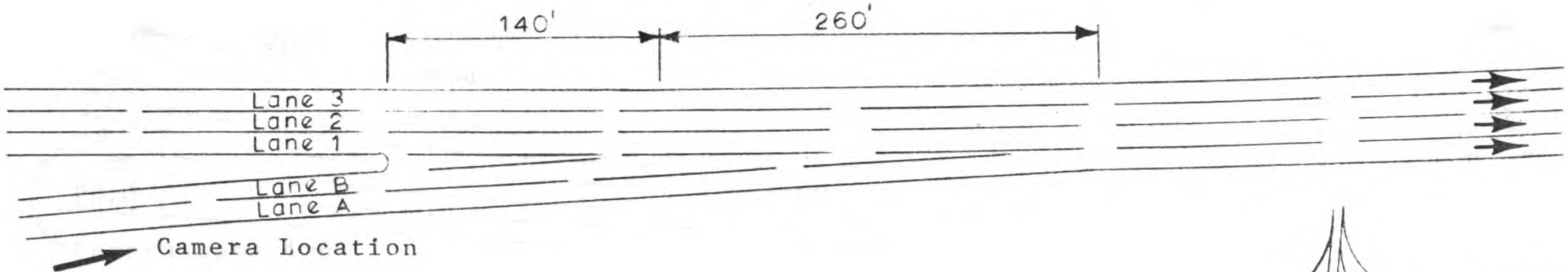
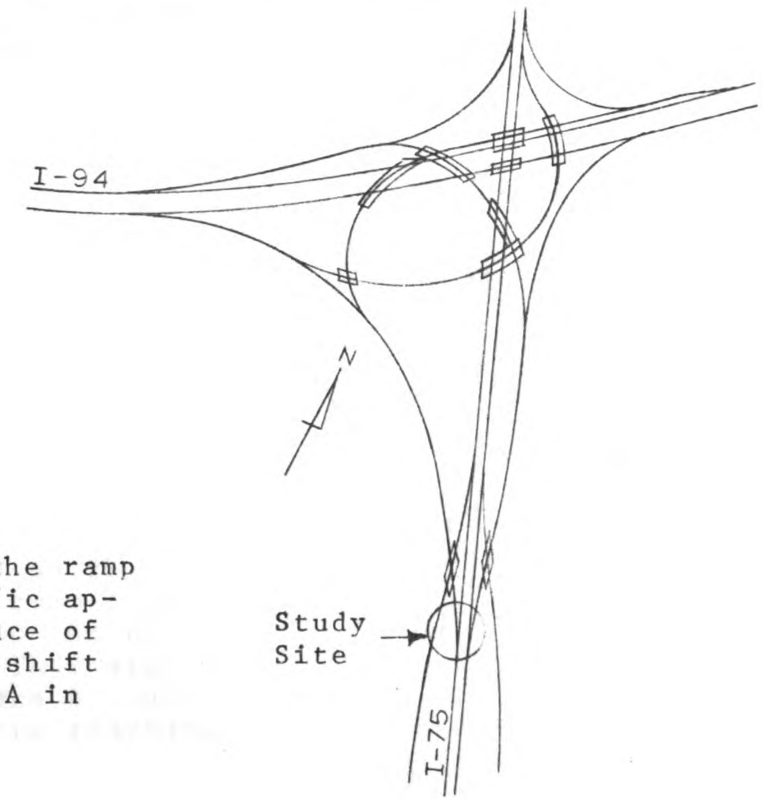


FIGURE 3.--M-39 Merge: Geometric Data.

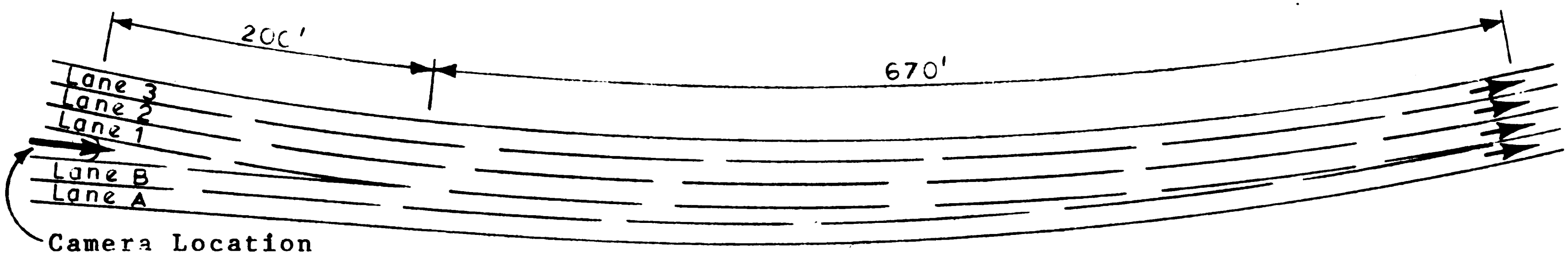


Ramp Lane Width	12'
Freeway Lane Width	12'
Angle of Convergence	3.5°
Ramp Grade at Nose	-0.40%
Freeway Grade at Nose	-1.00%
Freeway Curvature at Nose	Tangent
Ramp Curvature at Nose	5°
Distance to downstream on-ramp	1400'
Distance to downstream off-ramp	3800'

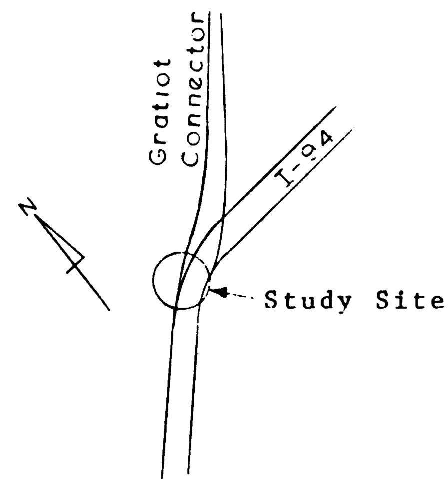


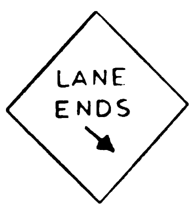
Note: This sign is located on the ramp to the left of ramp traffic approximately 500' in advance of the gore and is meant to shift lane B traffic into lane A in advance of the merge.

FIGURE 4.--I-75 Merge: Geometric Data.



Ramp Lane Width	12'
Freeway Lane Width	12'
Angle of Convergence	2^o
Ramp Grade at Nose	+1.50%
Freeway Grade at Nose	+1.16%
Freeway Curvature at Nose	3^o
Ramp Curvature at Nose	Tangent
Distance to downstream off-ramp	3069'





Note: This sign is located on the ramp to the left of ramp traffic approximately 500' in advance of the gore. This sign is meant to shift lane B traffic into lane A before reaching the gore.

FIGURE 5.--I-94 Merge: Geometric Data.

Film Analysis

The use of films as the data source provides a permanent record which can be used over and over. However, transferring the film data into usable information was both difficult and time consuming.

A Perceptoscope 16mm film projector was used to analyze the film. This projector enables the operator to run the film forward or backward at any speed. It can also be stopped on any frame desired.

Film analysis was accomplished by projecting the picture on a screen at a fixed magnification. A grid layout of the merge similar to the one shown on Figure 6 was then drawn on the screen, matching the 100 ft. field reference marks. Joint lines were also shown on the grid.

Form 1 was used by the observer to record the path of each vehicle as it proceeded through the merge. If a vehicle enters the merge in lane A, the observer records the vehicle's entering lane and each point at which the left rear wheel, if merging left (or the right rear wheel if merging right), crosses a lane line. To illustrate this process a theoretical car path is shown on Figure 6. This path is recorded on Form 1. The car enters the merge in lane A, the left rear wheel crosses the first lane line at point 3 and the second lane line at point 17.

Form 2 was used to total the vehicle paths recorded on Form 1. The segments shown on Form 2 are the same as

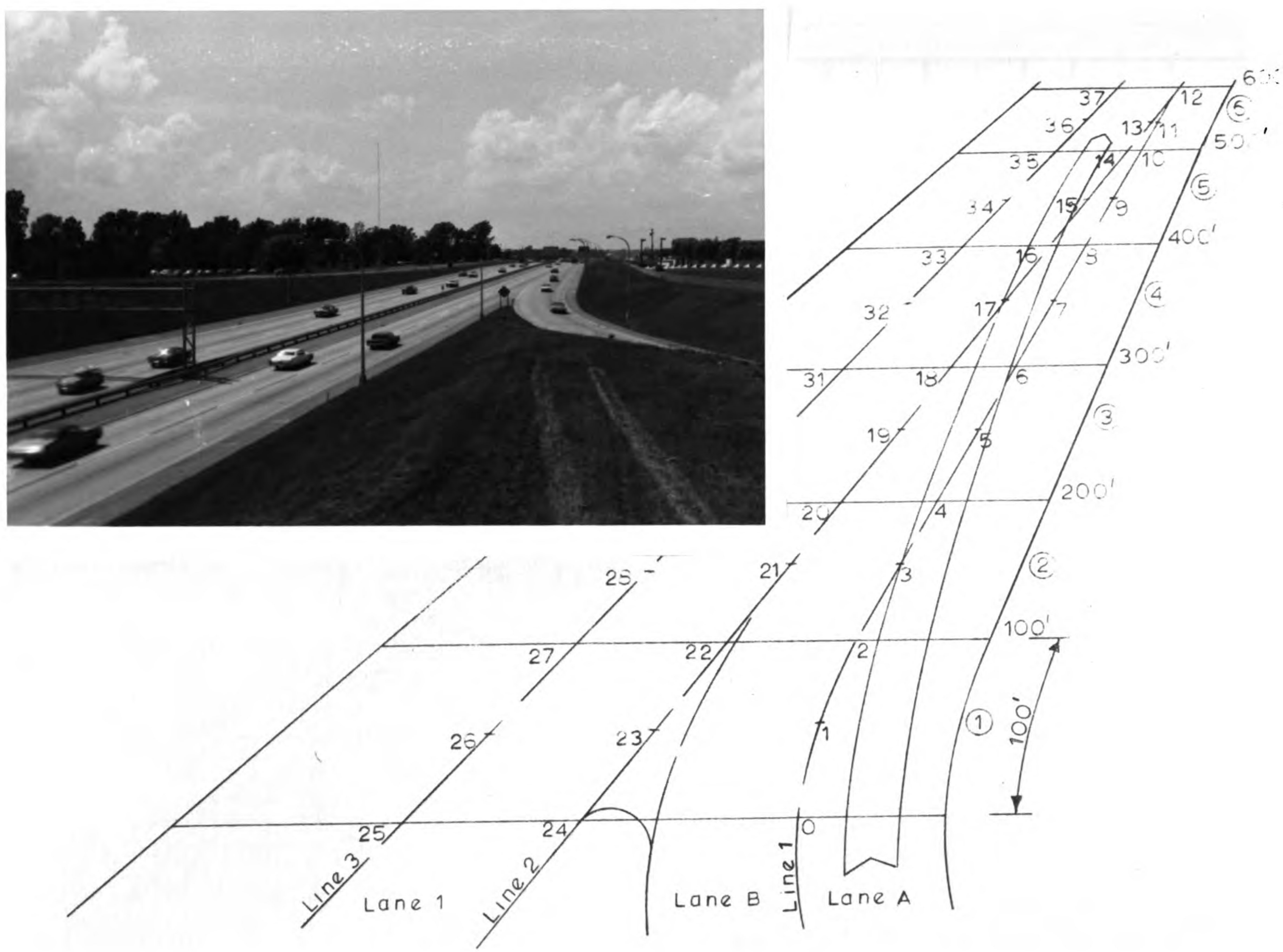


FIGURE 6.--Typical Grid Layout.

Appr. Lane	Crossing Points			Appr. Lane	Crossing Points			Appr. Lane	Crossing Points		
A	3	17									

FORM 1.--Record of Vehicle Paths from Film Analysis.

		Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Segment 6	TOTAL
Lane A	Lane Line 1							
	Lane Line 2							
Lane B	Lane Line 1							
	Lane Line 2							
Lane 1	Lane Line 1							
	Lane Line 2							

FORM 2.--Record of All Vehicle Paths per Film.

those shown on Figure 6. Form 2 was completed for each film. Again, using the same theoretical vehicle path, the crossing point at 3 is in segment 2. A mark is therefore recorded under segment 2 as shown. The crossing point at 17 is in segment 4. A mark is recorded under segment 4. The two marks are then connected with a line to denote the path of that particular vehicle. Each vehicle is recorded in this manner. Having determined the number of vehicles in each segment, the percentage distributions were developed as presented in the chapter on Analysis of Results.

A total of 7,742 vehicles were observed in this manner. Tables 1 through 4 indicate the actual number of vehicles analyzed at each location. Since the films vary in length, it was necessary to expand the observed volumes to vehicles per hour as shown. The expanded volumes provide a direct comparison between the peak and off-peak periods. Vehicle speeds and gap acceptance were not analyzed in this study. However, since a permanent film record is in existence, further insight into the operation of the merges could be obtained as part of a future study.

TABLE 1.--US-10 Merge: Time and Volume Data.

	Reel #	Date	Start Time (PM)	Film Time (Min)	Volume on Film Lane A	Volume on Film Lane B	Volume on Film Lane 1	Volume on Film Lane 2	Expanded Volume vph Lane A	Expanded Volume vph Lane B	Expanded Volume vph Lane 1	Expanded Volume vph Lane 2
Peak Hour	1	8-25	4:30	7.85	96	100	205	248	733	764	1566	1895
	2	8-25	4:45	8.00	124	124	212	239	930	930	1590	1793
	3	8-25	5:01	8.14	109	123	231	273	803	907	1703	2012
	4	8-25	5:13	8.08	104	120	195	240	1040	892	1449	1783
	Average								877	873	1577	1871
Off-Peak Hour	1	7-15	2:00	7.84	49	42	101	78	375	321	773	597
	2	7-15	2:15	7.91	61	54	117	105	463	410	888	797
	3	7-15	2:30	8.10	46	44	106	109	341	326	786	808
	4	7-15	2:45	3.49	19	20	66	44	327	344	1135	756
	Average								352	350	896	740

TABLE 2.--M-39 Merge: Time and Volume Data.

	Reel #	Date	Start Time (AM)	Film Time (Min)	Volume on Film Lane A	Volume on Film Lane B	Volume on Film Lane 1	Volume on Film Lane 2	Expanded Volume vph Lane A	Expanded Volume vph Lane B	Expanded Volume vph Lane 1	Expanded Volume vph Lane 2
Peak Hour	1	7-15	7:06	8.02	164	102	104	129	1227	763	778	965
	2	7-15	7:20	7.99	126	99	92	108	946	744	691	811
	3	7-15	7:36	8.04	154	111	78	117	1149	828	582	873
	4	7-15	7:50	8.14	128	80	78	99	944	590	575	730
	Average								1067	731	657	845
Off-Peak Hour	1	7-15	10:00	7.62	57	15	68	67	449	118	535	527
	2	7-15	10:15	8.02	61	8	62	51	456	60	464	382
	3	7-15	10:30	7.94	73	14	54	58	552	106	408	439
	4	7-15	10:45	5.00	37	12	40	37	444	144	480	444
	Average								475	107	472	448

TABLE 3.--I-75 Merge: Time and Volume Data.

	Reel #	Date	Start Time (AM)	Film Time (Min)	Volume on Film					Expanded Volume vph				
					Lane A	Lane B	Lane 1	Lane 2	Lane 3	Lane A	Lane B	Lane 1	Lane 2	Lane 3
Peak Hour	1	9-23	7:15	8.21	218	13	157	219	160	1594	95	1148	1601	1170
	2	9-23	7:30	8.26	215	15	166	235	181	1561	109	1205	1706	1314
	3	9-23	7:45	8.55	250	20	156	229	203	1755	140	1095	1608	1325
	4	9-23	8:00	8.23	247	44	119	207	171	1801	321	868	1509	1247
	Average									1678	166	1079	1606	1289
Off-Peak Hour	1	9-23	10:50	8:34	143	19	50	81	36	1028	137	360	582	259
	2	9-23	11:05	8:25	137	24	61	70	46	996	175	444	509	334
	3	9-23	11:20	8:39	138	38	50	78	34	987	272	358	558	243
	4	9-23	11:35	8.14	163	31	63	99	50	1201	229	464	730	369
	Average									1053	203	407	595	301

TABLE 4.--I-94 Merge: Time and Volume Data.

	Reel #	Date	Start Time (AM)	Film Time (Min)	Volume on Film Lane A	Lane B	Lane 1	Lane 2	Lane 3	Expanded Volume vph Lane A	Lane B	Lane 1	Lane 2	Lane 3
Peak Hour	1	8-25	8:07	7.72	69	9	37	74	66	536	70	287	575	513
	2	8-25	8:21	8.44	77	9	44	72	48	548	64	313	512	341
	3	8-25	8:32	8.19	80	7	28	72	50	586	51	205	528	367
	4	8-25	8:42	8.43	95	9	35	86	57	676	64	249	612	406
	Average									587	62	264	557	407
Off-Peak Hour	1	8-25	10:15	8.36	66	5	36	63	27	474	36	259	452	194
	2	8-25	10:30	8.34	66	6	30	62	36	475	43	216	446	259
	3	8-25	10:45	8.11	60	5	44	57	24	444	41	326	422	178
	4	8-25	11:00	8.21	56	8	39	49	34	409	58	285	358	249
	Average									451	45	272	420	220

CHAPTER IV

ANALYSIS OF RESULTS

Capacity Analysis

The subject of capacity analysis is a complex one. When discussing capacity, confusion exists even among traffic engineers concerning the meaning of the various terms used. This thesis will use the 1965 Highway Capacity Manual (3) as the basis for defining the terms used in this chapter.

The 1965 Highway Capacity Manual defines capacity as "the maximum number of vehicles which have a reasonable expectation of passing over a given section of freeway during a given time period under prevailing roadway and traffic conditions." As a rule of thumb, the capacity of a freeway lane is 2,000 vehicles per hour. Naturally, a freeway does not operate at capacity at all times. The Capacity Manual therefore develops the concept of level of service, which is simply a measure of various operating conditions ranging from very light traffic (Level of Service A) to stop-and-go traffic (Level of Service E). The Capacity Manual describes the various levels of service in the following general terms.

Level of Service A describes a condition of free flow, with low volumes and high speeds. Traffic density is low, with speeds controlled by driver desires, speed limits, and physical roadway conditions. There is little or no restriction in maneuverability due to the presence of other vehicles, and drivers can maintain their desired speeds with little or no delay.

Level of Service B is in the zone of stable flow, with operating speeds beginning to be restricted somewhat by traffic conditions. Drivers still have reasonable freedom to select their speed and lane of operation. Reductions in speed are not unreasonable, with a low probability of traffic flow being restricted. The lower limit (lowest speed, highest volume) of this level of service has been associated with service volumes used in the design of rural highways.

Level of Service C is still in the zone of stable flow, but speeds and maneuverability are more closely controlled by the higher volumes. Most of the drivers are restricted in their freedom to select their own speed, change lanes, or pass. A relatively satisfactory operating speed is still obtained, with service volumes perhaps suitable for urban design practice.

Level of Service D approaches unstable flow, with tolerable operating speeds being maintained though considerably affected by changes in operating conditions. Fluctuations in volume and temporary restrictions to flow may cause substantial drops in operating speeds. Drivers have little freedom to maneuver, and comfort and convenience are low, but conditions can be tolerated for short periods of time.

Level of Service E cannot be described by speed alone, but represents operations at even lower operating speeds than in Level D, with volumes at or near the capacity of the highway. At capacity, speeds are typically, but not always, in the neighborhood of 30 mph. Flow is unstable, and there may be stoppages of momentary duration.

For purposes of this discussion, the maximum volumes which can be handled at each level of service are as follows:

Level of Service	Vehicles per hour per lane (5% trucks)
A	1,000
B	1,200
C	1,400
D	1,600
E	1,800

Level of Service D is generally used when designing freeways for the Detroit area.

The ability of a freeway to handle the design level of service is dependent on the merge and diverge points. For example, at the merge of a single-lane ramp and lane 1 of the freeway, if the total volume of the two exceeds the design volume, then the potential exists for a bottleneck. Either the ramp traffic will back up, or freeway traffic will have to shift out of lane 1 assuming there is room in adjacent lanes. For Level of Service D, at the merge of a single-lane ramp and lane 1, 1800 vph is considered to be the capacity at that point. Any volume exceeding 1800 vph will cause the merge to break down. Moskowitz and Neman (7) have described the capacity of a merge in the following manner.

> Merging operation will be smooth as long as total ramp and adjacent lane rate-of-flow does not exceed 1800 vph provided that the entrance ramp terminal is long enough and has a gradual taper. Maximum combined flow-rates for a merge of a particular ramp and adjacent freeway lane have been observed as high as 2000 and 2200 vph. However, it

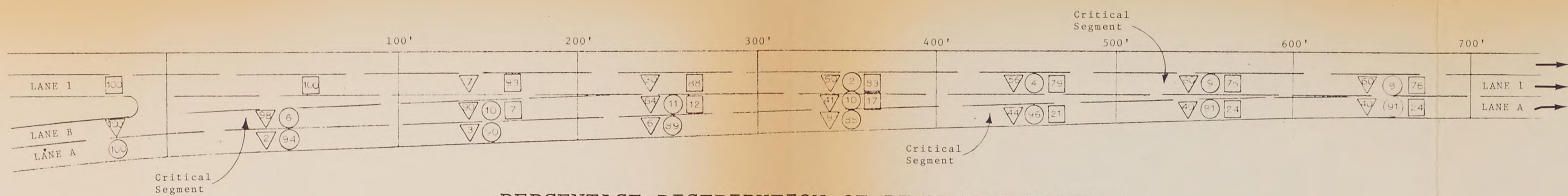
> is not recommended that this figure be anticipated in design procedures, since there are certain conditions of geometric design and traffic characteristics (which are difficult to predict or evaluate) that can prevent its attainment. 1800 vph is a dependable figure and can be counted on under all circumstances, with normal truck percentages and grades of less than 3%.

Therefore, 1800 vph is the key to analyzing any merge operation and will also be the basis for analyzing the capacity of two-lane merges. Two-lane ramp merges involve the merging and crossing of traffic in lanes A, B and 1. For purposes of analysis the lanes A, B and 1 were divided into 100 foot segments. At any particular segment along the merge, a portion or percentage of the traffic from each approach lane will be in that segment. If this traffic totals more than 1800 vph then that segment will experience congestion and possible stop-and-go operation, which in turn effects the rest of the merge. It was therefore necessary to determine how the ramp and lane 1 traffic distributed itself when passing through the merge area. This was accomplished by compiling by percentages the vehicle paths taken off the films. For each 100 foot segment the percentage distribution of traffic originating from lanes A, B and 1 was compiled. It was determined that the peak hour traffic should be used as the basis for the capacity analysis, as this would be representative of typical everyday urban driving conditions.

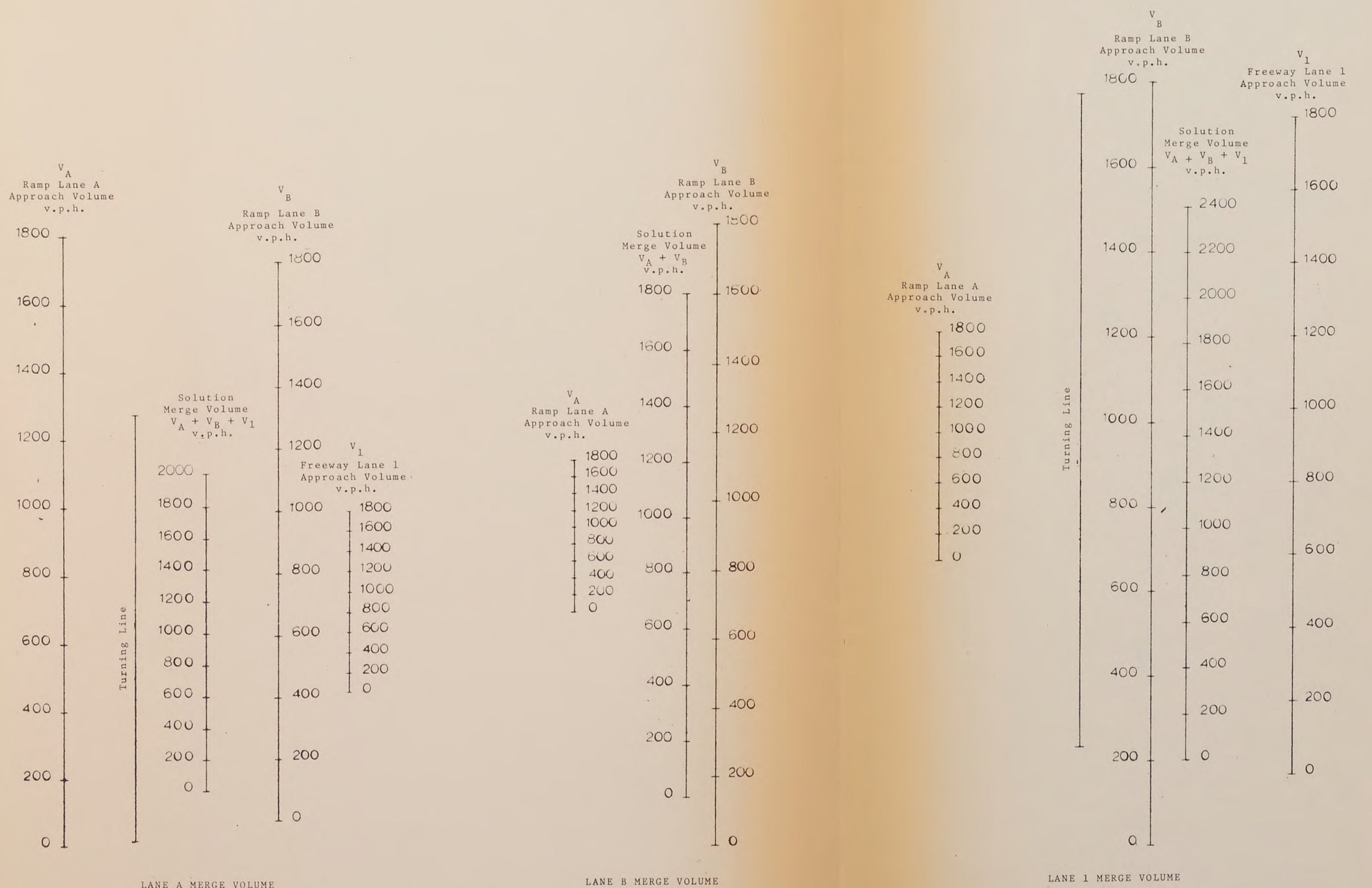
The results of the film analysis of peak hour traffic for the US-10 merge and the M-39 merge are illustrated in Figures 7 and 8 respectively. Figure 9 shows the off-peak distribution for the two merges. The design of the US-10 merge provides for the elimination of lane B and is referred to as an inside merge. The M-39 merge provides for the elimination of lane A and is referred to as an outside merge. Both will be analyzed for comparison of their traffic carrying ability.

In developing this capacity analysis method, it was felt that one segment in each lane would exceed 1800 vph before the other segments in that lane, regardless of the distribution of traffic entering the merge. It was therefore necessary to check all the lane-segments for all possible combinations of entering volumes to determine the critical segments. This critical segment could then be used for determining the capacity of each lane. The critical segments found in the analysis of M-39 and US-10 are indicated on Figures 7 and 8. It is now necessary to check only three segments for the US-10 merge and two segments for the M-39 merge to determine whether 1800 vph will be exceeded in the merge area.

For the US-10 merge, the nomographs shown on Figure 7 provide a simple method of determining whether the merge will exceed 1800 vph. The nomographs were developed using the percentages found at the critical segment of each lane, based on a method found in Hoelscher and Springer's "Engineering Drawing and Geometry." For example, the nomograph for the lane A merge volume is based on the percentages found in the critical



PERCENTAGE DISTRIBUTION OF PEAK HOUR TRAFFIC



Solution Procedure:

To determine the lane A merge volume

1. Draw a line from V_A to V_B. intersecting the turning line.
2. Draw a line from step 1 intersection of the turning line to V_1.
3. The intersection point of the line from step 2 with the solution line is the total merge volume.

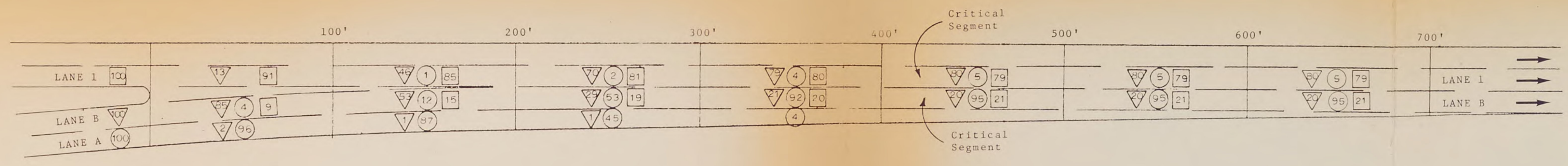
To determine the lane B merge volume

1. Draw a line from V_A to V_B. intersecting the solution line
2. The intersection point of the line from step 1 with the solution line is the total merge volume.

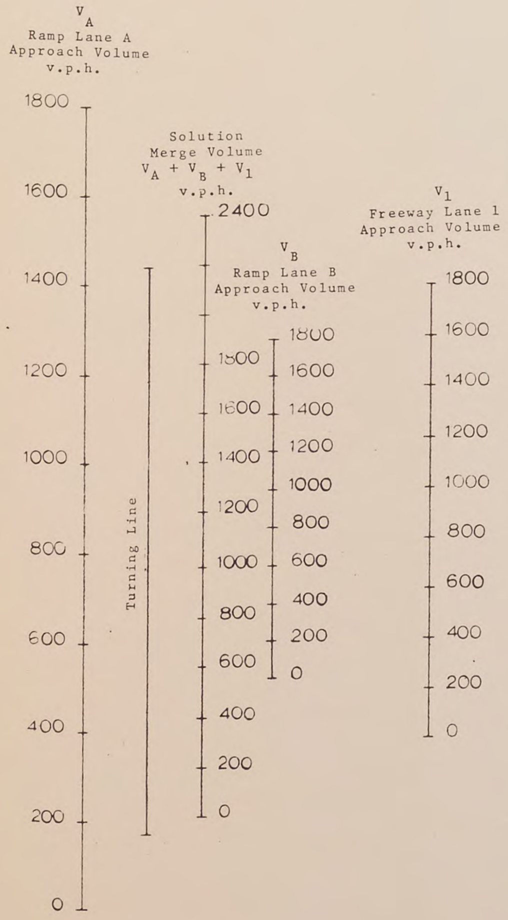
To determine the lane 1 merge volume

1. Repeat the procedure for determining the lane A merge volume.

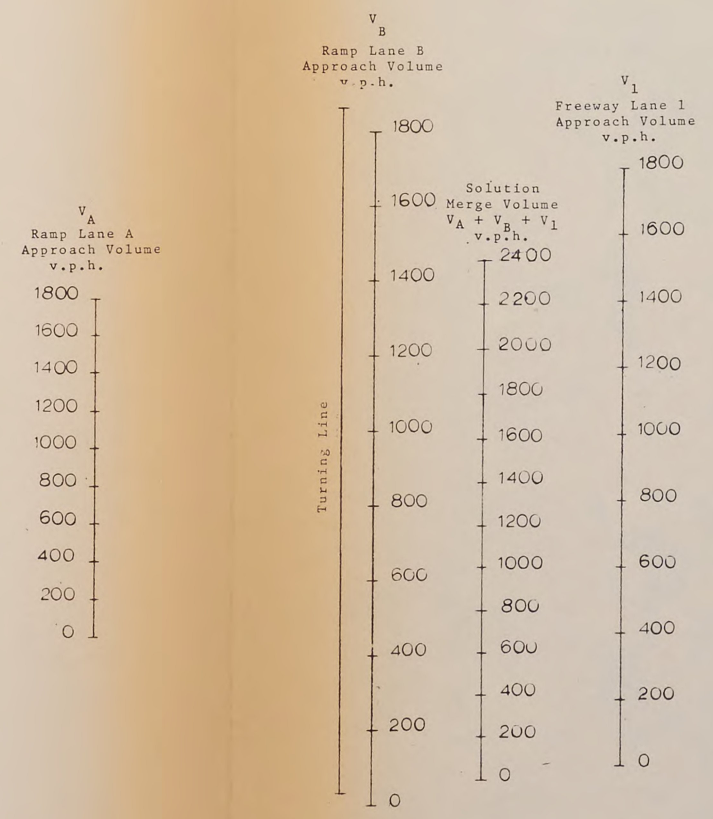
FIGURE 7.--US-10 Merge: Percentage Distribution of Peak Hour Traffic and Nomographs.



PERCENTAGE DISTRIBUTION OF PEAK HOUR TRAFFIC



LANE B MERGE VOLUME



LANE 1 MERGE VOLUME

Solution Procedure:

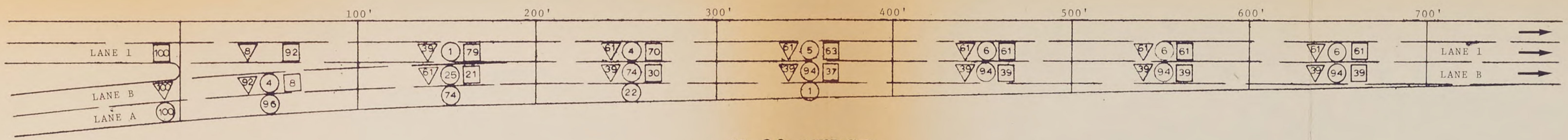
To determine the lane B merge volume

1. Draw a line from V_A to V_B intersecting the turning line.
2. Draw a line from step 1 intersection of the turning line to V_1.
3. The intersection point of the line from step 2 with the solution line is the total merge volume.

To determine the lane 1 merge volume

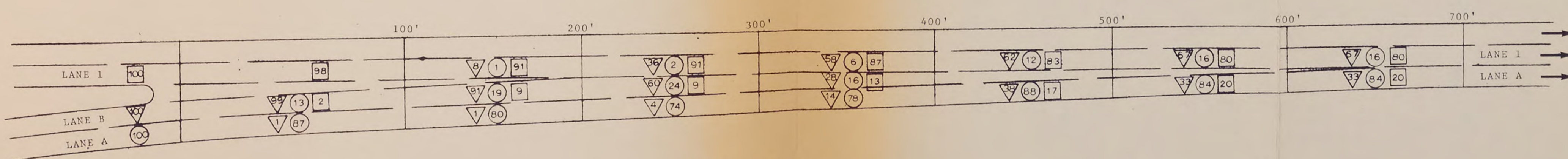
1. Repeat steps 1 through 3.

FIGURE 8.--M-39 Merge: Percentage Distribution of Peak Hour Traffic and Nomographs.



M-39 MERGE

PERCENTAGE DISTRIBUTION OF OFF PEAK HOUR TRAFFIC



US-10 MERGE

PERCENTAGE DISTRIBUTION OF OFF PEAK HOUR TRAFFIC

FIGURE 9.--US-10 Merge and M-39 Merge: Percentage Distribution of Off-Peak Hour Traffic.

segment for lane A. The procedure for using the nomographs is also shown on Figure 7. The following example using Figure 7 illustrates this procedure.

Given: Total Ramp Volume--1800 vph.
Freeway Volume--3500 vph.
Two-lane freeway (one direction)

Determine if lane A, B or 1 will exceed 1800 vph in the merge.

Step 1: Determine the distribution of approaching ramp and mainline traffic. Assume that lane A carries 80% and lane B 20% of the ramp traffic. At the beginning of the merge each lane will therefore carry the following volumes:

Lane A = 1440 vph.
Lane B = 360 vph.
Lane 1 = 1400 vph. (from Table 8.3, 1965 Highway Capacity Manual)

Step 2: Using the nomographs on Figure 7, determine the merge volume for each lane. The results are as follows:

Lane A = 1830 vph.
Lane B = 440 vph.
Lane 1 = 1440 vph.

While lane A exceeds 1800 vph, lane B and lane 1 are well under capacity.

The reader will note that the lane A and lane B approach volume percentages were assumed as 80% for lane A and 20% for lane B. Present research has not developed a method for predicting these percentages. To point out the critical nature of this assumption, the percentages will be reversed.

Given: Ramp volume--1800 vph.
Freeway volume--3500 vph.

Step 1: Determine the distribution of approaching ramp and mainline traffic. Assume 20% in lane A and 80% in lane B. At the approach to the merge each lane will therefore carry the following volumes:

Lane A = 360 vph.
Lane B = 1440 vph.
Lane 1 = 1400 vph.

Step 2: Using the nomographs on Figure 7, determine the merge volumes for each lane. The results are as follows:

Lane A = 1730 vph.
Lane B = 1430 vph.
Lane 1 = 2000 vph.

By reversing the percentages in lanes A and B, lane A is no longer over 1800 vph, however, lane 1 is now well over 1800 vph.

The nomographs for the M-39 merge shown on Figure 8 were developed in the same manner as the US-10 nomographs. The same example will again be used. Lane A does not have a nomograph as the percentages distribution in any of its segments never exceeds 100%.

Given: Ramp volume--1800 vph.
Freeway volume--3500 vph.

Step 1: Determine the distribution of approaching ramp and mainline traffic. Assume 80% in lane A and 20% in lane B. At the beginning of the merge each lane will carry the following volumes:

Lane A = 1440 vph.
Lane B = 360 vph.
Lane 1 = 1400 vph.

Step 2: Using the nomographs on Figure 8, determine the merge volumes for each lane. The results are as follows:

Lane A = not critical.
Lane B = 1720 vph.
Lane 1 = 1460 vph.

In this case none of the lanes exceeds 1800 vph and the ramp will handle the traffic without congestion. However if the percentages are again reversed, lane 1 will carry 2200 vph which is well over capacity.

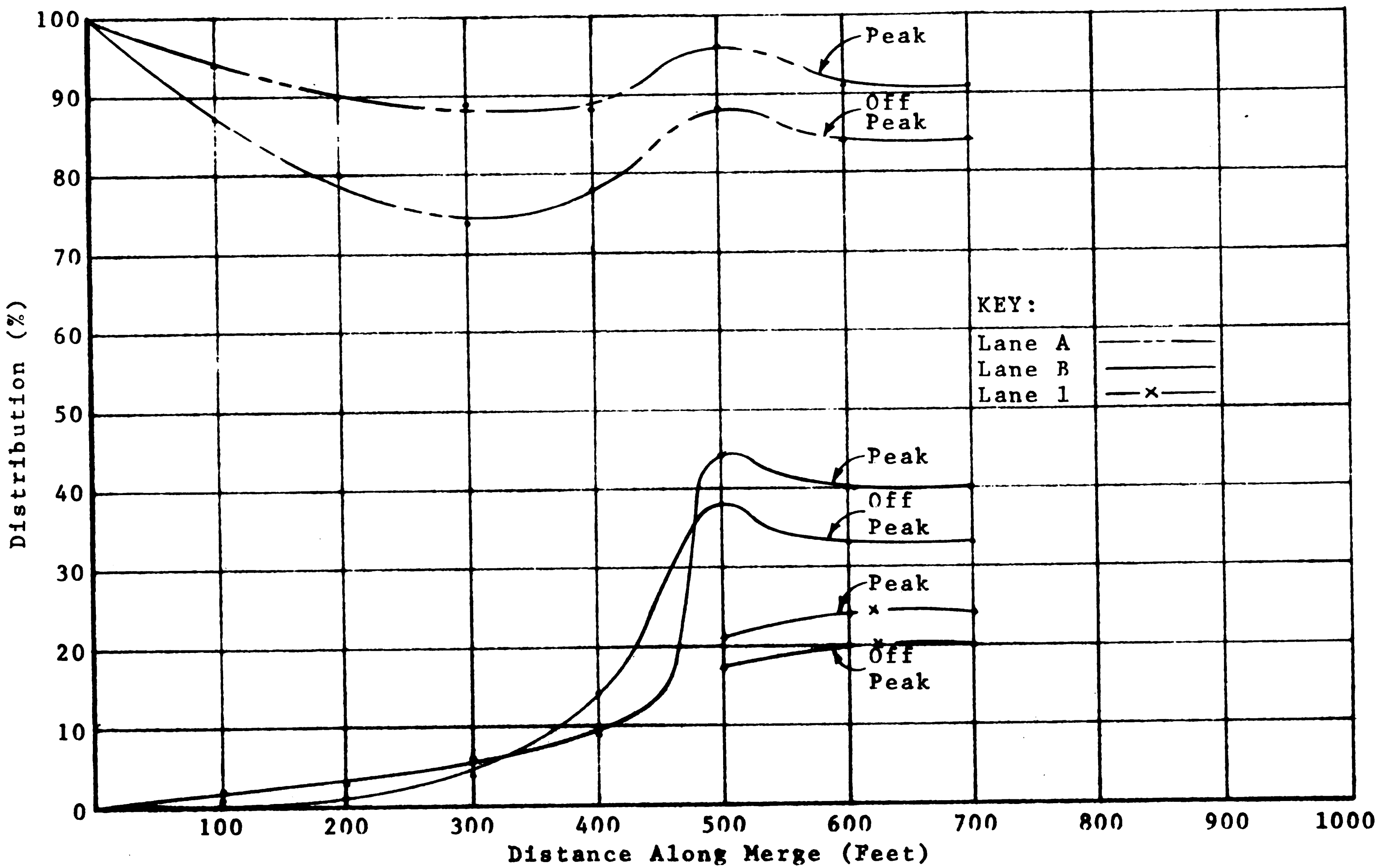
It should be noted that the I-75 and the I-94 merges were not utilized in this portion of the study. Because of geometric characteristics which affected the operation of the merge, they were signed down to one lane and were therefore not representative of typical two-lane merges.

Merge Design

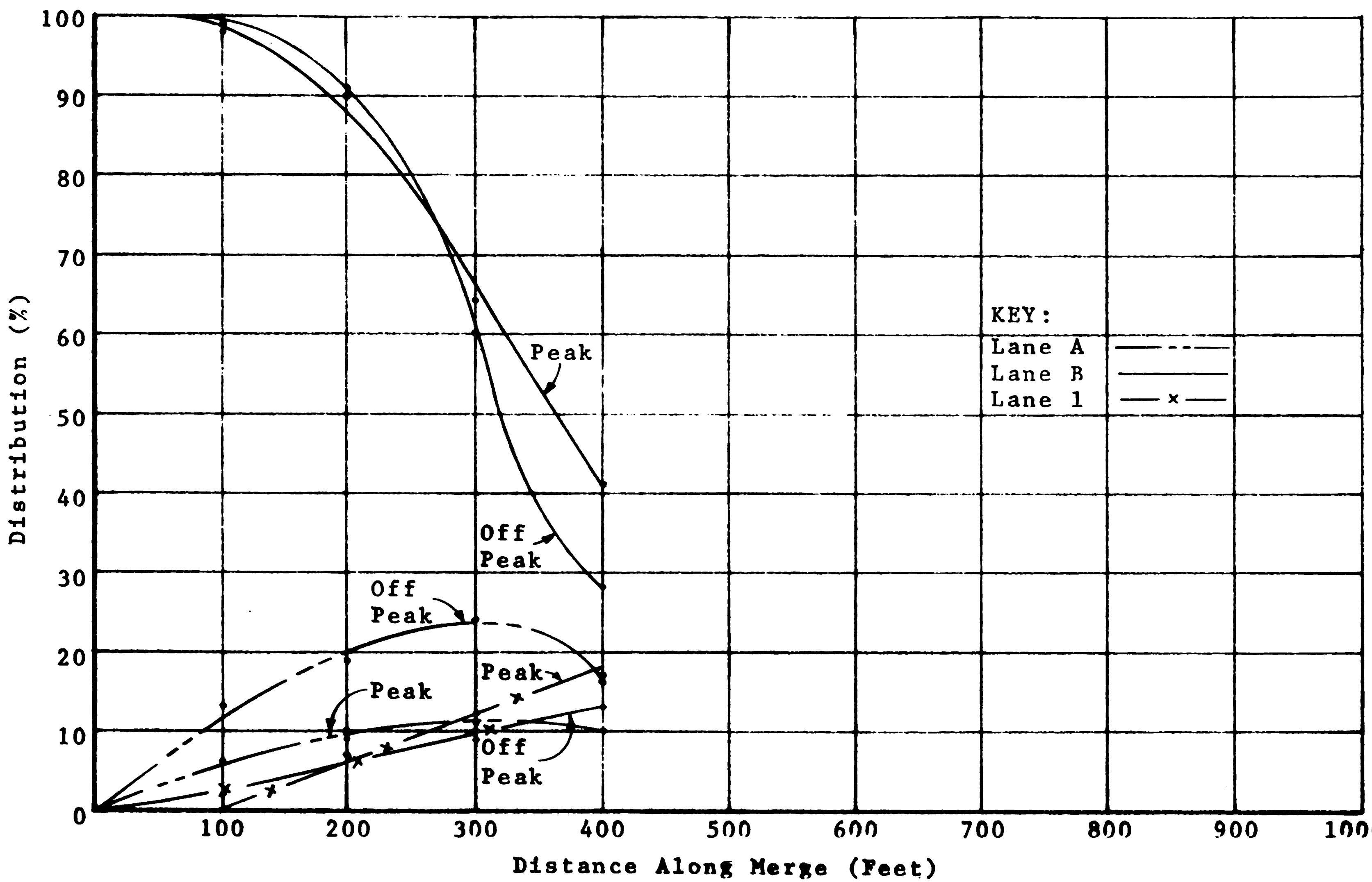
One goal of this thesis is to provide a rationale for determining which ramp lane to eliminate when designing a two-lane ramp merge. In order to draw some conclusions on this matter, it is necessary to discuss in detail the characteristics of each type of merge.

US-10 Merge

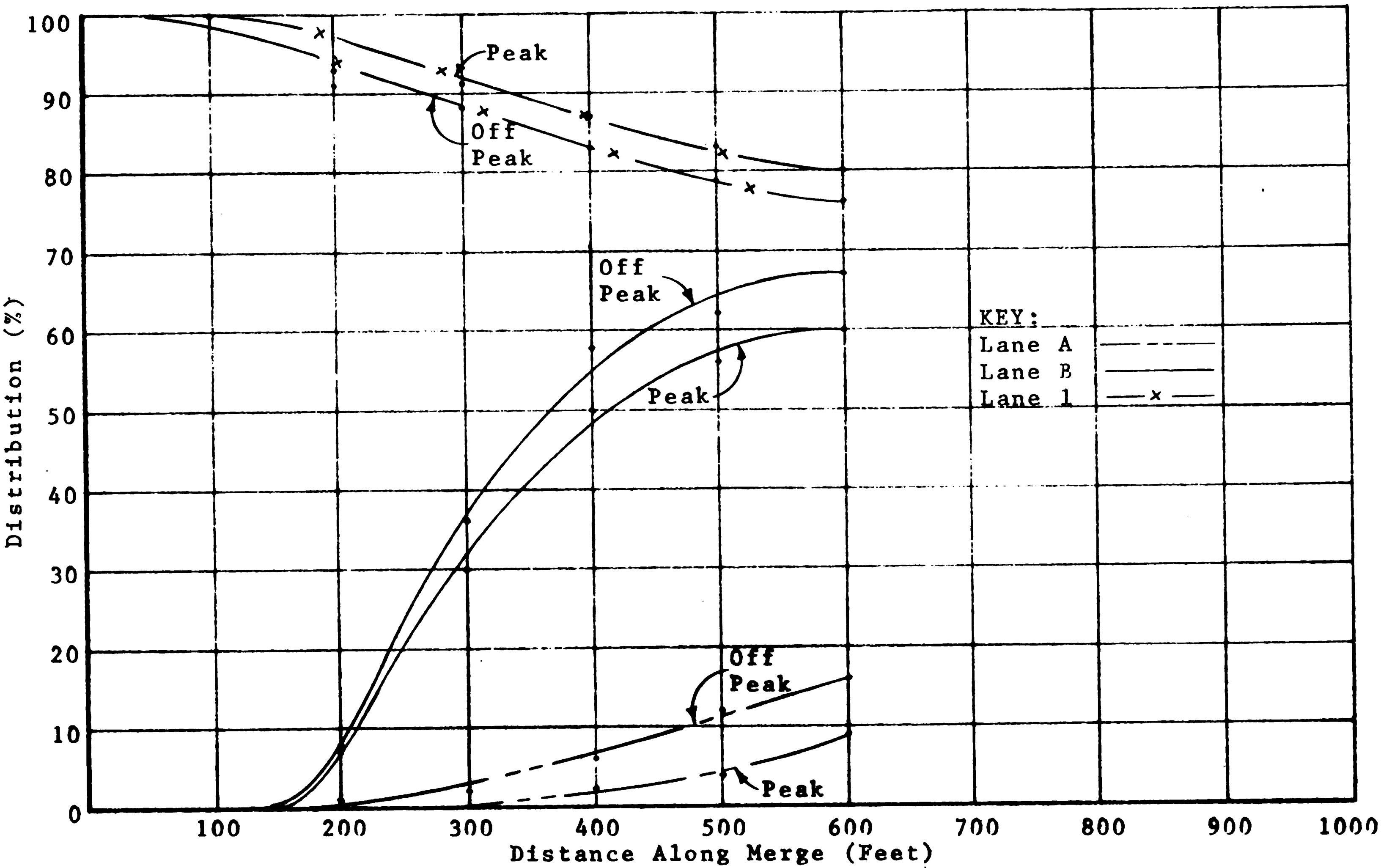
Lane B, the inside lane, is eliminated at this merge. Graphical representation of the percentage distribution of merging traffic during the peak and off-peak hours are shown in Graphs 1, 2 and 3. These curves were developed from the percentages shown on Figures 7, 8 and 9. The peak and off-peak curves are fairly consistent. It is evident that about 10% more traffic changes lanes during the off-peak than during the peak. This is to be expected since the gaps during the off-peak are large and allow the driver considerably more freedom of movement. The effect



GRAPH 1.--US-10 Merge: Percentage Distribution of Traffic in Lane A.



GRAPH 2.--US-10 Merge: Percentage Distribution of Traffic in Lane B.



GRAPH 3.--US-10 Merge: Percentage Distribution of Traffic in Lane 1.

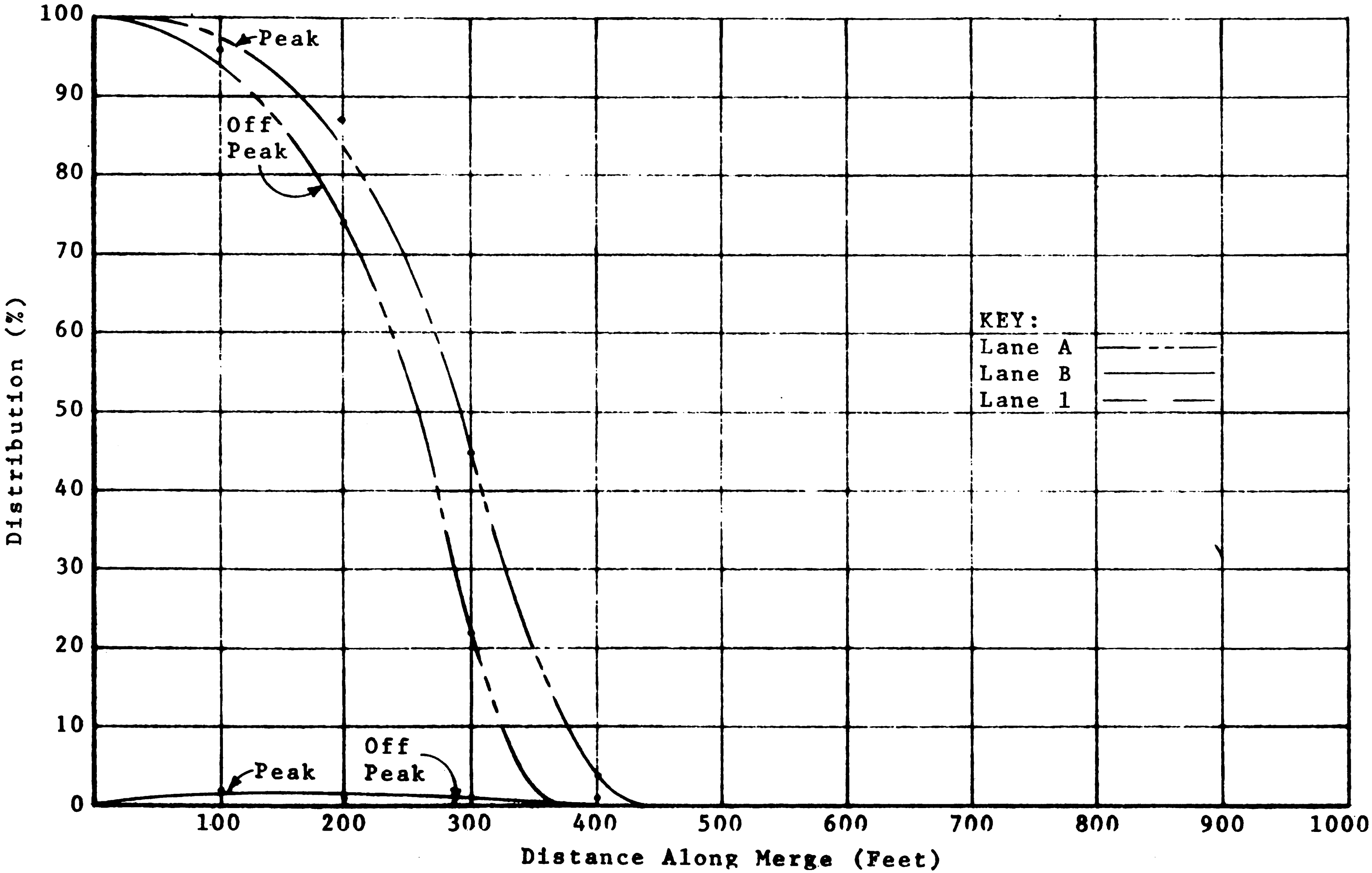
of lane 1 traffic on the operation of the ramp is evident in lane A (Graph 1). During the peak hour when the freeway was at capacity, 11% of lane A traffic merges into lane B, but only 4% ends up in lane 1. The other 7% merges back into lane A because no gaps are available in lane 1. This accounts for the hump in the curve at the 500 ft. point. As might be expected, this segment is critical for lane A.

Lane B traffic is faced with the same problem of small gaps in lane 1. Sixty percent of lane B peak hour traffic eventually merges into lane 1, however, this is about 7% less than during the off-peak. Even though lane B is eliminated, the vehicles in this lane did not appear to have difficulty in merging right or left and none were forced to come to a stop and wait for a gap at the end of the lane. The other 40% end up in lane A; however, most of this traffic waits until lane B is down to 6 ft. in width before shifting into lane A, which accounts for the sharp rise in the lane B curves (Graph 1). It is important to note that 20% of the lane 1 traffic shifts to the right during the off-peak and 24% during the peak. This amounts to a sizable number of cars considering the fact that an average of 1577 vehicles per hour approached the merge in lane 1. This movement added considerably to congestion in the merge primarily at the critical segment. Some of this desire to shift to the right can be attributed to a down-stream off-ramp; however, it appears that many drivers are shifting just to get more freedom to maneuver as both the freeway lanes were running near capacity during the filming.

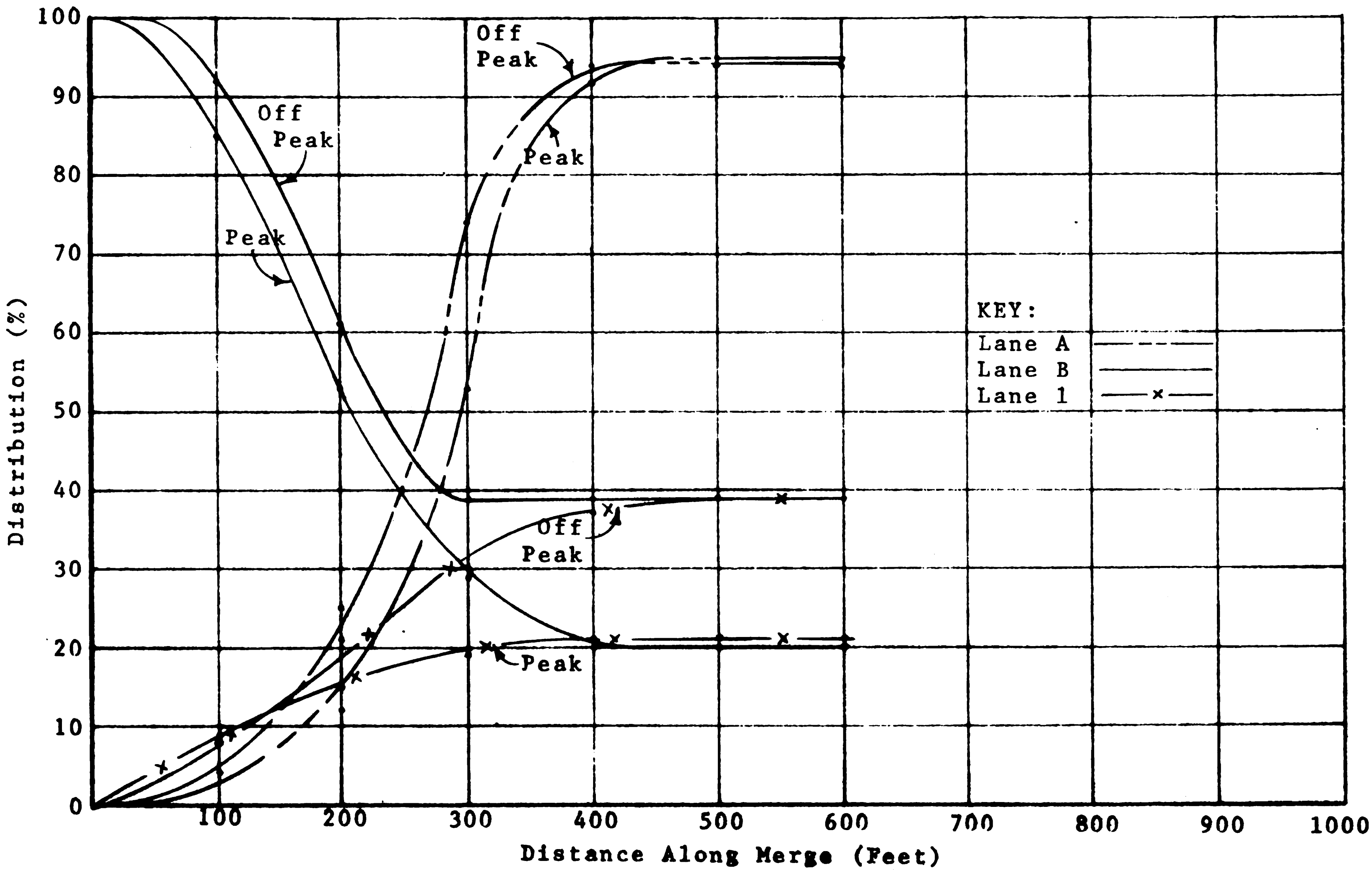
M-39 Merge

The M-39 merge is almost identical in design to the US-10 merge except that the outside lane is eliminated instead of the inside lane. While the volumes at this merge did not approach capacity, the distribution of traffic is felt to be representative of typical urban drivers. Graphs 4, 5 and 6 show the distribution graphically.

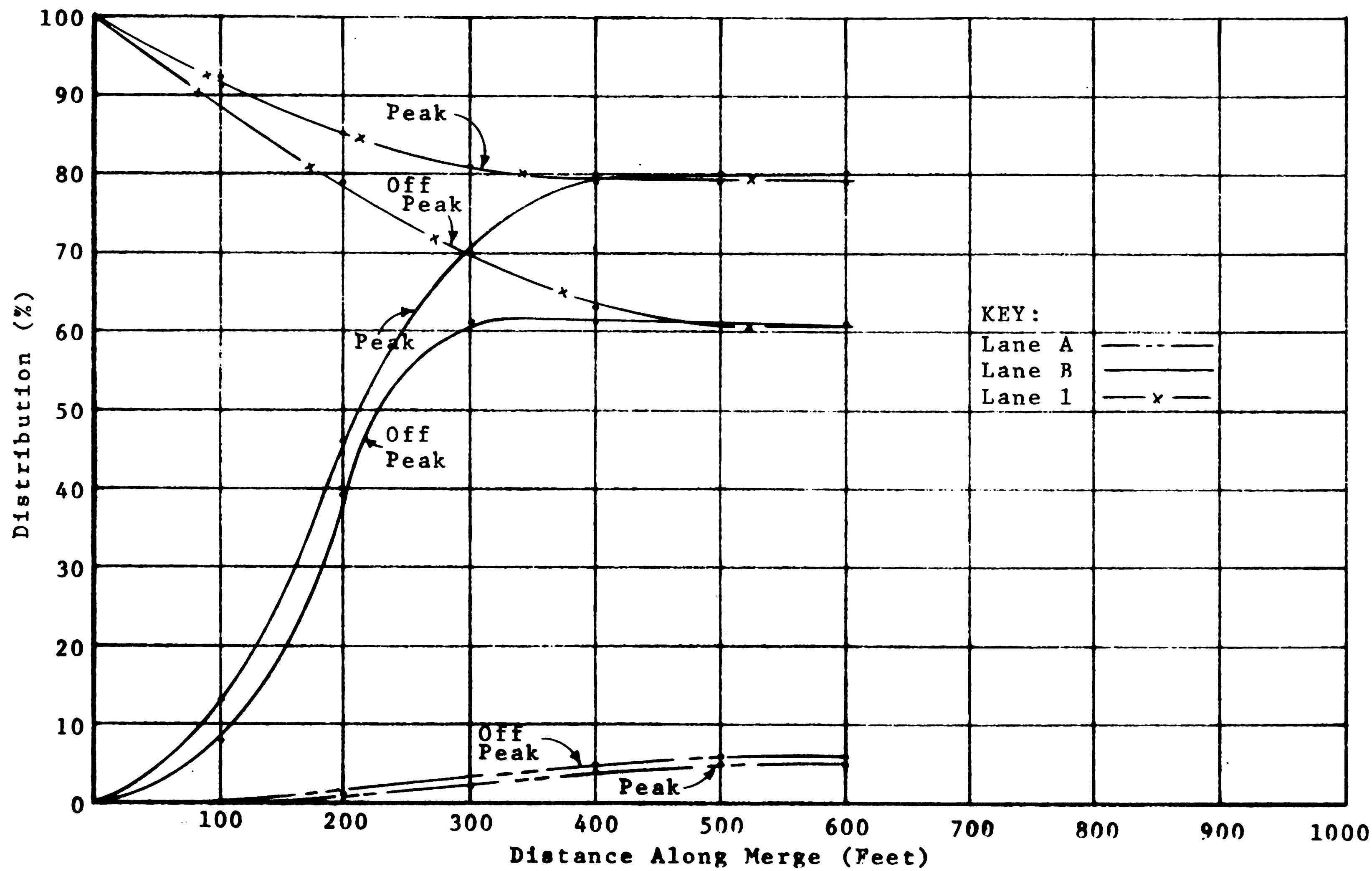
The lane A curve on Graph 5 shows that although lane A is carrying the greater share of traffic, that 96% of this traffic has merged into lane B within 300 feet of the ramp nose. The last 300 feet of lane A was practically unused by traffic. This occurs during both the peak and off-peak hours. This would indicate that traffic in lane A desired to shift to the left as soon as possible giving the impression that they knew their lane was ending and must merge quickly into lane B. The pressure that Lane A traffic exerts on lane B is evidenced by the fact that 70% of the lane B traffic merged into lane 1 in the first 300 feet even though a continuous lane is provided for lane B traffic. It should be noted that the freeway operated at fairly light volumes even during the peak hour which provided large gaps in lane 1 for ramp B traffic. In the presence of heavy freeway volumes, the percentage merging into lane 1 would probably not be as great. As was the



GRAPH 4.--M-39 Merge: Percentage Distribution of Traffic in Lane A.



GRAPH 5.--M-39 Merge: Percentage Distribution of Traffic in Lane B.



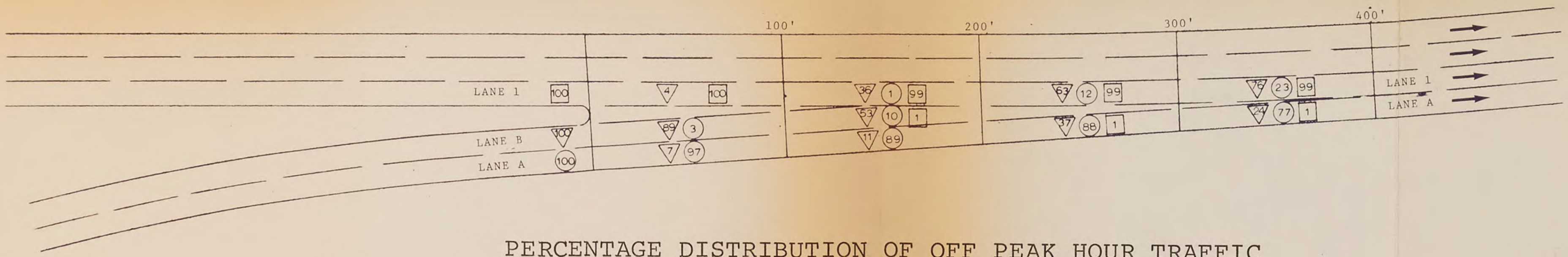
GRAPH 6.--M-39 Merge: Percentage Distribution of Traffic in Lane 1.

case for the inside merge, 20% of lane 1 traffic shifts to the right in the merge section during the peak hour and 40% during the off-peak.

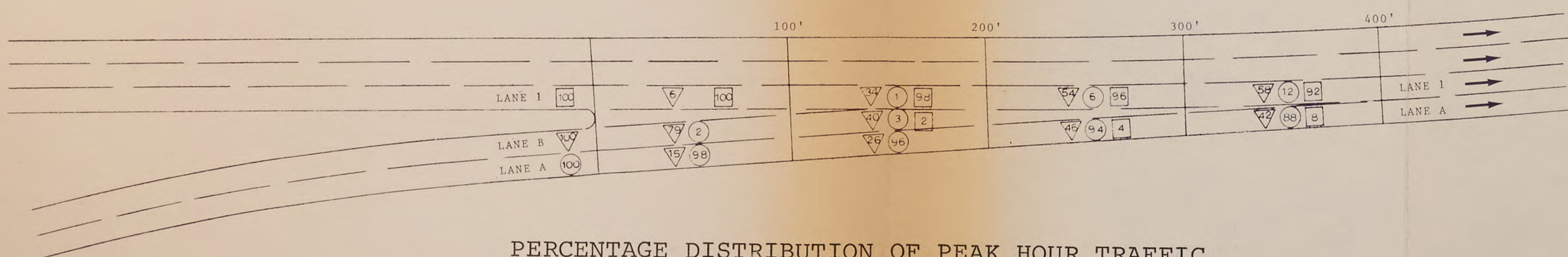
In conclusion, this research has indicated that at the M-39 outside merge, a portion of lane A is not utilized by ramp traffic which could be attributed to the driver's desire to get out of lane A before it ends. In contrast, the US-10 inside merge does not experience this phenomenon in the lane to be dropped which in this case is lane B. The total merging length is utilized. This leads to the conclusion that when the inside merge is provided, more efficient use is made of the total two-lane merging area. Further research is needed, however, to verify this conclusion since at the time the films were taken, the centerline paint lines were faint. Except for the construction joint lines, the driver had little indication that his lane was running out.

Signing

As mentioned previously, the I-75 and I-94 merges were signed down to one lane in advance of the nose at the time of filming. Figures 10 and 11 show the percentage distribution of traffic for the peak and off-peak hour. The I-94 merge illustrates the operational problems created by introducing an on-ramp on a freeway curve. Even though this merge is signed down to one lane with 90% compliance of the signing, once the traffic passes the nose 57% of

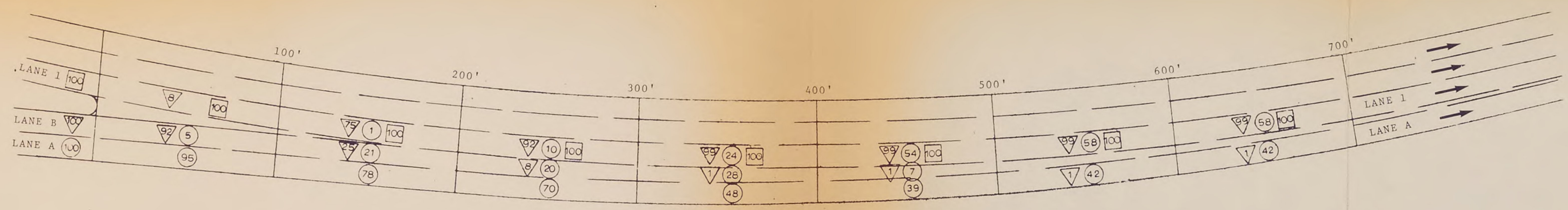


PERCENTAGE DISTRIBUTION OF OFF PEAK HOUR TRAFFIC

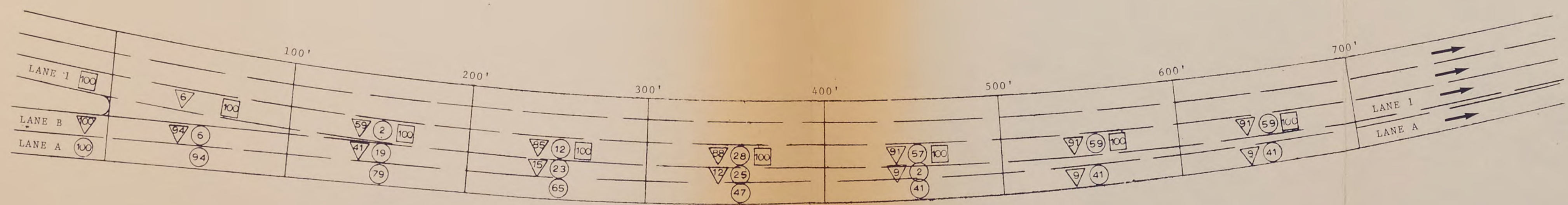


PERCENTAGE DISTRIBUTION OF PEAK HOUR TRAFFIC

FIGURE 10.--I-75 Merge: Percentage Distribution of Off-Peak Hour and Peak Hour Traffic.



PERCENTAGE DISTRIBUTION OF OFF PEAK HOUR TRAFFIC



PERCENTAGE DISTRIBUTION OF PEAK HOUR TRAFFIC

FIGURE 11.--I-94 Merge: Percentage Distribution of Off-Peak Hour and Peak Hour Traffic.

lane A traffic and 91% of lane B traffic moves directly into lane 1. Although the merge length for lane B is considerably longer than at the other three locations, traffic did not follow the merge around the curve but instead drove in a straight line to lane 1.

The I-75 merge is introduced on a very slight curve but the major problem is the short merge distance for lane B. Again 85% to 90% compliance of the signing is experienced which is fortunate since the capacity of lane B would be very limited due to the short merging distance. It is interesting to note that very little traffic in lane 1 desires to shift into the added lane at this merge compared to the 20% to 40% at the M-39 and US-10 merges. This can probably be attributed to having three freeway lanes approaching the merge allowing the driver considerably more maneuvering room which is not the case on two-lane freeways.

To summarize, signing a two-lane ramp down to one lane was found to be 90% effective. A merging situation should never be introduced on a freeway curve unless the curve is very slight regardless of the merging distance provided. The motorist will tend to merge in a straight line and not make use of total merging length. The lane B merge length should be at least 600 feet.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The method of data collection, the 16mm colored films, proved to be very adequate for plotting vehicle paths. The films provided an accurate permanent record of the four merges selected for study in this thesis. Care must be taken, however, to avoid distortion of the picture due to curvature of the roadways being filmed. At the I-94 merge the camera was located in the gore of the merge; however in the films the curvature of the freeway and ramp distorted the end of the merge to the extent that the observer found it very difficult to determine which lane a vehicle was in. When the freeway and ramp were on tangent, placing the camera well back in the gore of the merge proved to be the best point of observation, as long as the camera was at least 30 feet above the pavement. This height or higher must be maintained, otherwise a vehicle being observed may be blocked from view by a following car or truck.

The proposed method of analyzing the capacity of two-lane merges is based on the criteria that no more than 1800 vph can be handled on any one segment of a freeway lane or ramp lane. The nomographs, which were developed from the percentage distribution of ramp and lane 1 traffic, provided a simple method of determining whether or not the ramp lanes or lane 1 would exceed 1800 vph. Nomographs were developed for the M-39 outside merge and the US-10 inside merge, providing a comparison of the two designs.

The question of which lane to eliminate at two-lane merges has not been resolved by this thesis, although the vehicle path analysis indicated that more efficient use of the total merge area was experienced when the inside lane or lane B was eliminated. A driver interview would have to be conducted, however, to determine what effect the presence of an inside or outside merge actually has on the motorist.

Two of the four merges studied, the I-75 and the I-94 merges, were signed down to one lane. An average of 90% of the ramp vehicles complied with the signing at both locations. It appears therefore that the signing has accomplished its intended purpose of operating the ramp as a single-lane leading directly into the added freeway lane beyond the merge. The I-75 merge experienced flow rates as high as 1800 vph in the outside lane and operated very smoothly primarily due to the fact that 80% of this traffic continued into the added lane. It must be remembered, however, that signing is a special treatment to eliminate operational problems created by the geometrics of the two merges. The sharp freeway curvature at the I-94 merge and the short lane B merge at the I-75 merge necessitated the special signing.

Recommendations

The nomographs developed in this thesis may be utilized for the capacity analysis of other two-lane merges with similar operational and geometric characteristics as the ones studied. However, further research is needed to develop merge distributions for other two-lane merge situations such as the merge of a three-lane freeway and two-lane ramps. Furthermore, research is needed to provide a basis for predicting the amount of traffic in each lane of the ramp in advance of the merge. This distribution would in all probability be a function of the geometrics of the ramp proper.

As a result of this research, the determination of which ramp lane to eliminate can only be based on some general criteria. Further research is needed to develop specific warrants. It was concluded that for the inside merge more efficient use of the merge area was experienced. In many cases, the outside ramp or lane A will carry the greatest share of the ramp traffic. If this is the case, then the inside lane or lane B should be eliminated so that the predominant volume which is in lane A will be carried directly into the added freeway lane. However, there are locations where lane B will be carrying the predominant volume in which case the outside merge should probably be utilized. The predominant volume which is in lane B will then be carried directly into the added freeway lane. Where ramp volumes are as high as 3000 vph the traffic will tend to distribute itself evenly between lane A and lane B. In this case it would appear that the inside merge should be utilized, thereby assuring the most efficient use of the merge area.

While it was concluded that signing a two-lane ramp down to one lane is effective, it must be recognized that this operation will only work efficiently up to volumes of 1800 vph which is the capacity of a single-lane ramp merge. It is assumed that the reason for building the two-lane ramp in the first place was the future need to carry volumes greater than 1800 vph. The signing therefore is an interim measure to avoid operational problems created by the geometrics of the merge. Before ramp volumes increase to above 1800 vph consideration must be given to rebuilding or relocating the ramp to eliminate the operational problem.

It is apparent from the film analysis that a two-lane ramp must merge at a point where the freeway is on tangent alignment. Furthermore, the ramp approach to the merge should be on tangent with an angle of convergence of 2° or less. The taper length should be a minimum of 600 ft. and the ramp and freeway grades should be as flat as possible to insure a clear view of the merge area. In designing two-lane merges, if these minimum geometric requirements are adhered to, maximum operational efficiency of the merge can be expected.

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