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16. Abstract

This report includes three separate research efforts--one analysis of operational characteristics and two case studies of specific improvements. The first study was an analysis of the operation of auxiliary lanes under high volume conditions. Evaluation of four auxiliary lanes between closely-spaced ramp pairs in San Antonio showed that entering traffic tended to stay in the auxiliary lane longer during high volume conditions; exiting traffic entered the auxiliary lane earlier during high volume conditions than during free flow conditions.

A case study of a ramp reversal in Houston is presented next. The basic framework for analyzing such a modification is developed before the case study. Because there are substantial disbenefits associated with a ramp initiating plans to reverse a single ramp. Procedures for performing such an analysis are discussed.

The final portion of the report is a case study of a ramp pair in San Antonio that was grade-separated to eliminate a short, heavily-congested weaving section. Based on historical information, this study makes some fairly broad assumptions regarding operational effectiveness. While the grade separation is shown to be highly successful in reducing weaving accidents, the study indicates that accident reduction alone is not likely to produce sufficient savings to justify the construction cost and, therefore, operating, travel time and delay costs must be jointly considered.

This is the final report for the research study entitled "Evaluation of Urban Freeway Modifications."

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AN ANALYSIS OF URBAN FREEWAY OPERATIONS AND MODIFICATIONS

Final Report

by

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Research Report 210-12F
Evaluation of Urban Freeway Modifications
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INTRODUCTION

Urban growth in Texas has placed tremendous demands on freeway systems. With main lane expansion becoming an ever-diminishing possibility, many Districts of the Texas State Department of Highways and Public Transportation have begun modifying various elements of the freeway to maximize operations. Increased user savings are realized, as well as increases in effective main lane capacity.

SUMMARY OF REPORT CONTENTS

This report includes three separate research efforts--one analysis of operational characteristics and two case studies of specific improvements. The first study was an analysis of the operation of auxiliary lanes under high volume conditions. Evaluations of four auxiliary lanes between closely-spaced ramp pairs in San Antonio showed that entering traffic tended to stay in the auxiliary lane longer during high volume conditions; exiting traffic entered the auxiliary lane earlier during high volume conditions than during free flow conditions.

A case study of a ramp reversal in Houston is presented next. The basic framework for analyzing such a modification is developed before the case study. Because there are substantial disbenefits associated with a ramp reversal, the study suggests a detailed analysis be performed prior to initiating plans to reverse a single ramp. Procedures for performing such an analysis are discussed.

The final portion of the report is a case study of a ramp pair in San Antonio that was grade-separated to eliminate a short, heavily-congested weaving section. Based on historical information, this study makes some fairly broad assumptions regarding operational effectiveness. While the grade separation is shown to be highly successful in reducing weaving accidents, the study indicates that accident reduction alone is not likely to produce sufficient savings to justify the construction cost and, therefore, operating, travel time and delay costs must be jointly considered.

This is the final report for the research study entitled "Evaluation of Urban Freeway Modifications." A list of other reports published in connection with this study may be found in Appendix A.

AN ANALYSIS OF AUXILIARY LANES ON URBAN FREEWAYS

INTRODUCTION

Through the 1983 Cooperative Research Program with the Texas State Department of Highways and Public Transportation (SDHPT), the Texas Transportation Institute (TTI) conducted studies to identify the operational problems of auxiliary lanes under high volume conditions. The data collection effort was performed in San Antonio, Texas, during February and May of 1983. The results presented will serve to identify some of these problems which appear to be related to the design features and operational characteristics of auxiliary lanes.

Background

With increasing growth in urban areas in Texas, the traffic congestion on urban freeways continues to increase. Traffic congestion and other operational problems are being observed during high volume conditions near auxiliary lanes between closely spaced entrance and exit ramp pairs on urban freeways. Auxiliary lanes are provided to improve the level of service of the weaving section between high-volume entrance and exit ramp pairs. Present engineering evaluations are based on capacity procedures provided in the Highway Capacity Manual (1). The emphasis is usually placed on the weaving phenomena that occurs during moderate volume conditions. However, during high volume conditions (e.g., levels of service D, E, F) and closely spaced entrance-exit ramp pairs, classic weaving may not routinely occur.

STUDY OBJECTIVES

The primary objective of this study was to determine the extent of operational problems of auxiliary lanes during high volume urban freeway conditions. The high volume conditions were to consist of both free flow and congested flow to allow for comparison. Major problems to be considered were in areas of freeway performance and weaving characteristics of entering and exiting vehicles.

STUDY SITES

Site Selection

Table 1 lists the sites which were studied in accordance with the objectives of this study. All studies were conducted in San Antonio, Texas. They were selected based on their geometric characteristics after consultation with SDHPT highway design engineers. The sites were selected based on lengths ranging from approximately 1000 to 3500 feet. One site outside the suggested range was studied due to the existence of several short auxiliary lanes on older facilities. Although sites with auxiliary lanes with lengths of up to 3500 feet do exist, none with high traffic volumes (congested flow) were found. All of the sites listed in Table 1 experience some degree of congestion during peak periods.

Pavement Marking

The auxiliary lanes which were used in this study used two different pavement marking patterns. One pattern consisted of striping the entire auxiliary lane, while the other consisted of only partial striping. The study site on I-35 Northbound was the only site which used continuous striping. The marking pattern for 75 percent of the auxiliary lane's length was similiar to that of the mainlanes. Near the exit, it was marked as an "exit only" lane with pavement markings as well as a solid white line which separated the auxiliary lane and the mainlanes. The partial striping pattern consists of only striping areas adjacent to the entrance and exit ramp gore areas. The distance striped includes 25 percent of the total auxiliary lane length from the end of each gore area. This pattern, which is unique to the San Antonio area, results in a total marking of only 50 percent of the weaving area. This particular pavement marking pattern may have some effect upon the weaving

TABLE 1. AUXILIARY LANE STUDY SITES IN SAN ANTONIO.

LOCATION	DIRECTION	# OF MAINLANES	LENGTH (FEET)	RAMP VOLUME (ADT)	
				ENTRANCE	EXIT
I-10 West Cincinnati to Culebra	Eastbound	2	425	1,980	3,380
I-10 West I-410 to Callaghan	Westbound	3	935	15,650	13,660
I-35 North Main to St. Marys	Northbound	2	2072	12,060	5,930
I-410 North Broadway to Airport	Westbound	3	2230	11,640	13,080

characteristics of the users of the auxiliary lane. Figure 1 shows a schematic diagram of this partial striping pattern.

DATA COLLECTION

The data collection effort was performed by TTI during the months of February and May of 1983. The effort performed in February was primarily a pilot collection effort to test the data collection method. The data for the three remaining sites was collected in May. Data was collected for at least two days at each site. Each site was studied during the peak period in the peak direction of flow. A list of the dates each site was studied, as well as the peak period, is provided in Table 2. The study times for the AM and PM peak periods were 6:45 AM to 8:45 AM and 4:00 PM to 6:00 PM, respectively.

The data collected included freeway mainlane volumes, truck volumes, entrance and exit ramp volumes, and vehicular speeds. Mainlane traffic volumes, separated by lanes, were recorded manually at points upstream and downstream of the auxiliary lane. The ramp volumes were recorded using automatic traffic counters. All volumes were recorded in 5-minute increments. Speeds of random vehicles were determined using either a radar gun or by recording the travel time of vehicles over a predetermined distance. The speeds or travel times were recorded in 1-minute increments. Weaving characteristics of both entering and exiting traffic were observed by recording the lane changes in and out of the auxiliary lane. The freeway was initially broken into specific sections, and the lane change movements within each section were noted. TIMELAPSE cameras were used in the pilot study to aid in the observation of the weaving characteristics. After much consideration, it was determined that their use was not justified for the remainder of the study.

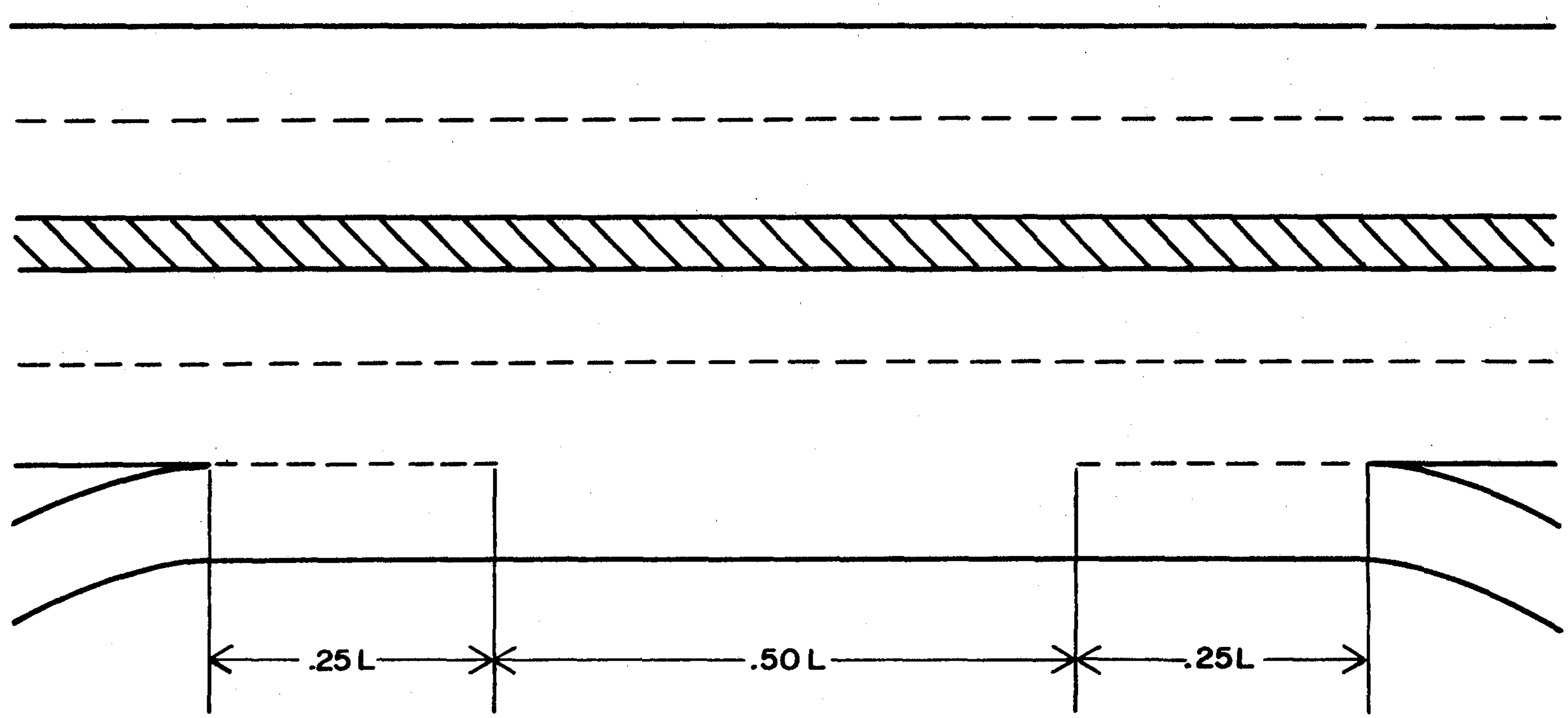


Figure 1. Pavement Marking Pattern

TABLE 2. STUDY PERIODS AT STUDY LOCATIONS.

LOCATION	February, 1983						May, 1983									
	Wednesday 23rd		Thursday 24th		Friday 25th		Monday 2nd		Tuesday 3rd		Wednesday 4th		Thursday 5th		Friday 6th	
	AM	PM	AM	PM	AM	PM	AM	PM	AM	PM	AM	PM	AM	PM	AM	PM
I-10 Eastbound Cincinnati to Culebra													X		X	
I-10 Westbound I-410 to Callaghan												X		X		
I-35 Northbound Main to St. Marys		X	X	X	X					X						
I-410 Westbound Broadway to Airport								X	X		X					

DATA ANALYSIS

The data collected from the field studies were analyzed from two different approaches. One approach involved an analysis of the operational performance of the freeway sections and auxiliary lanes studied. The other consisted of a study of the weaving characteristics of vehicles utilizing the auxiliary lane. The weaving characteristics were compared between free flow and congested mainlane traffic flow. This provides a comparison similiar to one based upon the level of service concept. With one exception, the data from all the collection periods were analyzed using both approaches. The data collected on I-35 Northbound on May 3, 1983 was not considered to be normal traffic flow and therefore was not analyzed. A work zone on I-10 near its interchange with I-35 appeared to meter the traffic to I-35. This resulted in somewhat lower traffic volumes within the study section.

Freeway Performance

The operational performance of freeway sections with an auxiliary lane may be determined on a level-of-service basis using a method described by TRB Circular 212 (2). The method may be performed either graphically or by using the equations provided. The alternative of using equations was selected and a SAS (3) computer routine was developed to facilitate the data reduction process. This allowed for a faster reduction of the field data to measures of level-of-service. The method determines the level-of-service by analyzing the freeway configuration as a ramp-weave section with a continuous auxiliary lane. The level-of-service for non-weaving vehicles is determined by calculating the average speed of the non-weaving vehicles. The resulting differential between the calculated speed for weaving vehicles and that of non-weaving vehicles determines the level-of-service for the weaving vehicles. Table 3 indicates these levels-of-service as well as the range of values for each.

TABLE 3. LEVEL OF SERVICE IN WEAVING AREAS.

NON-WEAVING VEHICLES	
Level of Service	Avg. Running Speed of Non-Weaving Vehicles MPH (km/h)
A	$S_{NW} \geq 50$ (80)
B	$S_{NW} \geq 45$ (72)
C	$S_{NW} \geq 40$ (64)
D	$S_{NW} \geq 35$ (56)
E	$S_{NW} \geq 30$ (48)
F	$S_{NW} < 30$ (48)
WEAVING VEHICLES	
Level of Service for Weaving Vehicles is ___ the Level of Service for Non-Weaving Vehicles	IF ΔS is MPH (km/h)
the same as	$\Delta S \leq 5$ (8)
1 level poorer than	$\Delta S \leq 10$ (16)
2 levels poorer than	$\Delta S \leq 15$ (24)
3 levels poorer than	$\Delta S \leq 20$ (32)
4 levels poorer than	$\Delta S \leq 25$ (40)

Source: Ref. 2.

The geometric input for this procedure includes the number of lanes in the study section and the length of the auxiliary lane. The traffic volumes must be broken down into weaving and non-weaving flows. The weaving flows are the entrance and exit ramp volumes. Non-weaving flows may be defined as the thru volume on the freeway mainlanes and that of vehicles utilizing the auxiliary lane as a thru lane (i.e., vehicles which use the auxiliary lane only and do not enter onto the mainlanes). Appendix B provides for a detailed explanation of the procedure used for the determination of the levels-of-service.

The levels-of-service for each data collection period, as determined by the procedure of Appendix B, may be found in Appendix C. This brings about a discussion of the validity of the above mentioned procedure for use with the data collected at these four sites. A major concern is the determination of the speed of the weaving vehicles (Equation 2, Appendix B). The equation is illustrated graphically by Figure 3.5 (a) of TRB Circular 212 (2). It suggests lengths of auxiliary lanes ranging from 500 to 2000 feet only. No suggestions are included in the text as to possible ranges for lengths in which the equation is valid, although it may be assumed to be the same as indicated by the figure.

In many cases, the calculated speed of the non-weaving vehicles, did not closely agree with that measured in the field. Cases also occurred in which higher levels-of-service were indicated than those which actually occurred in the field. Examples of this included situations of queued (stop-and-go) flow on the freeway which resulted in the determination of a level-of-service of A.

The speed differential between the speed of non-weaving vehicles and that of weaving vehicles is used to determine the level of service for the weaving vehicles. There were cases in which this resulted in a negative speed differential, which indicates that the weaving vehicles were moving faster than the non-weaving vehicles. TRB Circular 212 (2) does not indicate how to handle such cases.

Statistical tests were performed to test the "equality" between the calculated speed of non-weaving vehicles (S_{NW}) and that of the average speed which was measured in the field. The comparison was made for each data collection period in which speed data were recorded. A two-sample Student's t test was used for each comparison. A significance level of 5% (α = 0.05) was assumed for testing the hypothesis that the speed of the non-weaving vehicles (calculated) is the same as the average speed measured in the field. The results of these statistical tests are shown in Table 4. Each of the samples is based on a variable number of 5-minute intervals. The average speed measured in the field for each 5-minute interval was determined by averaging all speeds measured for that period.

Table 4 shows that the above mentioned hypothesis may be rejected four times, but it cannot be rejected on five occasions. This offers no conclusion concerning the use of this set of data with the procedure to determine the levels-of-service as described in TRB Circular 212. However, it is interesting to note that three of the four times in which the hypothesis was rejected were for periods of queued flow. In seven of the nine collection periods which were tested, the mean of the calculated speed of non-weaving vehicles (S_{NW}) was greater than that which was measured in the field.

Due to the inconclusiveness of the statistical tests, no conclusions may be derived from the level-of-service calculations of this field data. The lack of agreement of the calculated speed of non-weaving vehicles (S_{NW}) to that measured in the field may cause questions to arise concerning the accuracy of the data. However, the major problem appears to be with the lengths of the auxiliary lanes exceeding the valid ranges of the equations used in the evaluation procedure.

TABLE 4. RESULTS OF STUDENT'S t TEST.

HWY-DATE	Speed Sample	n	$\overline{x}$ (mph)	s (mph)	t_{calc}	C*	Comments
I-35 NB-PM Peak (2-23-83)	S_{NW}	11	57.6	6.54	1.350	2.086	Cannot Reject
	Avg. Meas.	11	54.6	2.57			
I-35 NB-AM Peak (2-24-83)	S_{NW}	23	63.4	7.90	2.632	2.018	Reject
	Avg. Meas.	22	58.6	3.35			
I-35 NB-PM Peak (2-24-83)	S_{NW}	21	58.5	5.87	4.774	2.023	Reject
	Avg. Meas.	20	39.3	17.43			
I-35 NB-AM Peak (2-25-83)	S_{NW}	12	56.2	5.21	-0.368	2.080	Cannot Reject
	Avg. Meas.	11	56.9	3.71			
I-410 WB-PM Peak (5-2-83)	S_{NW}	22	55.4	9.38	0.865	2.020	Cannot Reject
	Avg. Meas.	21	53.6	1.71			
I-10 WB-PM Peak (5-4-83)	S_{NW}	24	44.4	11.3	0.176	2.016	Cannot Reject
	Avg. Meas.	23	43.8	12.1			
I-10 WB-PM Peak (5-5-83)	S_{NW}	21	48.4	14.0	-1.096	2.025	Cannot Reject
	Avg. Meas.	19	52.2	5.95			
I-10 EB-AM Peak (5-5-83)	S_{NW}	21	50.7	5.75	12.16	2.025	Reject
	Avg. Meas.	19	28.7	5.67			
I-10 EB-AM Peak (5-6-83)	S_{NW}	21	49.9	6.55	2.434	2.027	Reject
	Avg. Meas.	18	42.8	11.36			

* Test value obtained from Student's t distribution (Source: Ref. 4.)

Weaving Characteristics

The weaving characteristics of vehicles using auxiliary lanes were analyzed on a graphical basis. The weaving patterns for both entering and exiting traffic were observed under queued as well as free flow freeway traffic conditions. The percentages of entering and exiting traffic in the auxiliary lane at various points were determined from the field data. These percentages were based on all the data collected at each site for both free flow and queued conditions. Individual curves for each 5-minute data collection period were not constructed. In all instances, the number of 5-minute intervals of queued and free flow traffic conditions on the freeway varied according to location, date, and peak period. The exact number of intervals for each data collection period is shown by Table 5.

Figures 2(a), 2(b), 2(c), and 2(d) show a graphical representation of the distribution of exiting traffic in the auxiliary lane for each site. The curves on each figure represent both queued and free flow conditions. Figure 2(a) is a representation of the exiting characteristics on I-10 Eastbound from the Cincinnati entrance to the Culebra exit. The resulting distribution for this extremely short auxiliary lane indicated that the differences between the exiting characteristics of periods of congested and free flow conditions may be minimal.

Figures 2(b) and 2(c) illustrate these same concepts for the study sites on I-10 Westbound and I-35 Northbound. Both of these indicate that under queued conditions, vehicles exiting the freeway tend to enter the auxiliary lane sooner than they do under free flow conditions. This is because of the nature of the drivers to try and avoid excessive delay by using the auxiliary lane, which may be less congested than the freeway mainlanes.

TABLE 5. NUMBER OF 5-MINUTE INTERVALS USED TO DETERMINE WEAVING CHARACTERISTICS

HIGHWAY	QUEUED FLOW	FREE FLOW
I-10 Eastbound	25	19
I-10 Westbound	9	36
I-35 Northbound	8	51
I-410 Westbound	4[(1)]	39

(1)NOTE: These queues were in the auxiliary lane only, and not in the freeway mainlane.

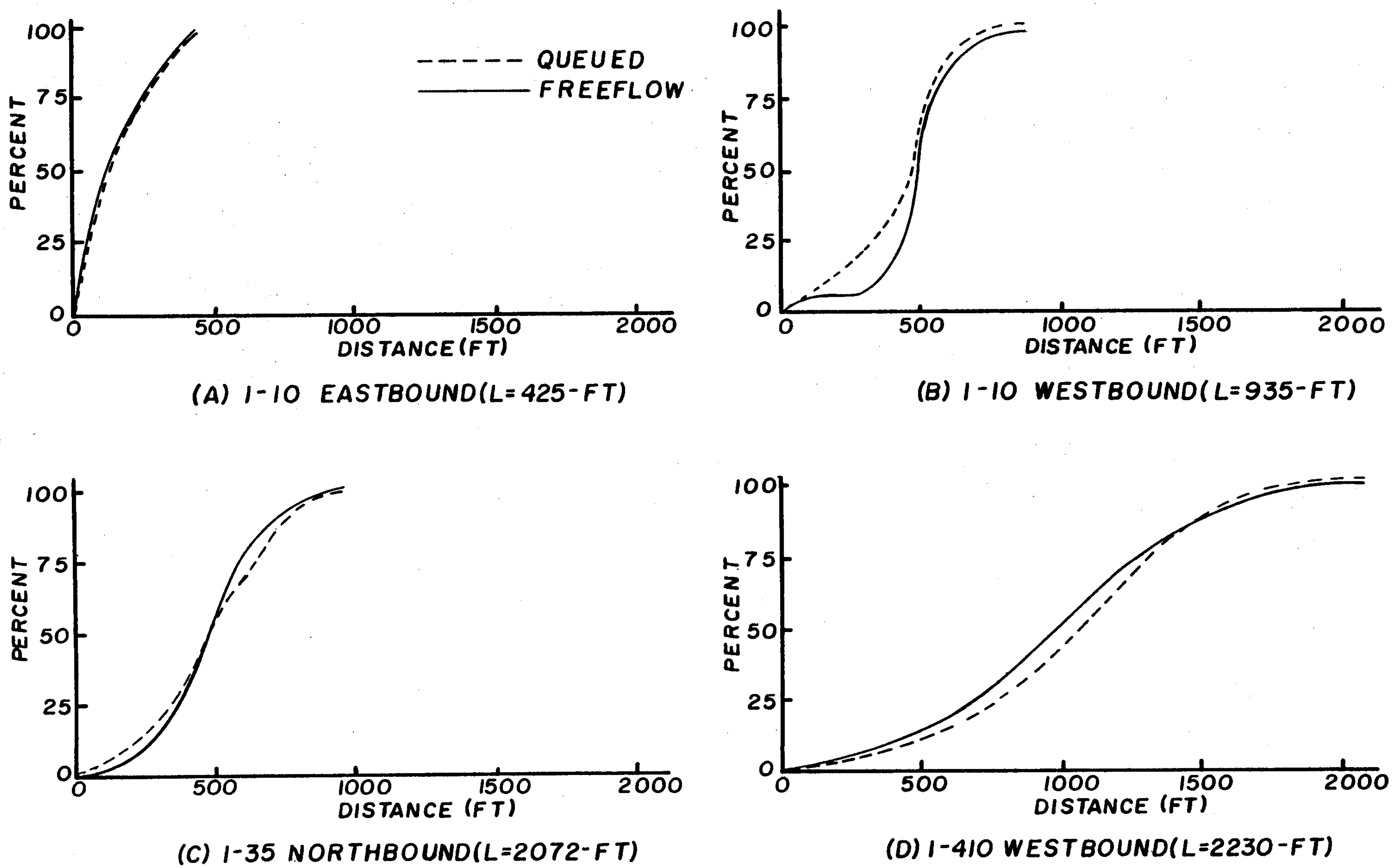


FIGURE 2. DISTRIBUTION OF EXITING TRAFFIC IN AUXILIARY LANE

Figure 2(d) represents the exiting characteristics of those using the auxiliary lane of the study section located on I-10 Westbound. This graph is inconclusive due to its definition of the periods of queued flow. In this instance, the only queue which occurred was in the auxiliary lane near the exit ramp. This graph should not be used in comparison with Figures 2(a), 2(b), and 2(c) when discussing queued conditions.

A graph was constructed to provide for a possible comparison between all the sites studied. It was prepared by assuming that the length of the auxiliary lanes was 1.0 and by proportioning the sections between. The result is Figure 3, which does not contain the data from I-410 Westbound under queued conditions. It shows that all the exiting traffic follows the same basic trend, with the exception of one site. In most cases, the traffic patterns are basically the same. However, the data collected on I-35 Northbound does not follow the pattern of the others. A better representation of the patterns for all of the sites would have resulted if each auxiliary lane was divided into a larger number of short sections for data collection purposes.

Weaving patterns for entering traffic were also analyzed and are illustrated graphically by Figures 4(a), 4(b), 4(c) and 4(d). These figures indicate that entering vehicles tend to stay in the auxiliary lane longer during free flow than during congested traffic conditions. Under such queued conditions, classic weaving does not occur. The drivers of the entering vehicles must force themselves into the mainlane traffic under queued conditions instead of a high-speed merge as under free flow conditions. During such high-speed weaving maneuvers, the speed alone of the entering vehicles "carries" them further along in the auxiliary lane than when queued. Figure 5 shows a comparison between the traffic patterns of entering traffic for all sites. Again, the data for I-410 Westbound under queued flow was not included in the preparation of this figure. As with that of the exiting traffic, the patterns

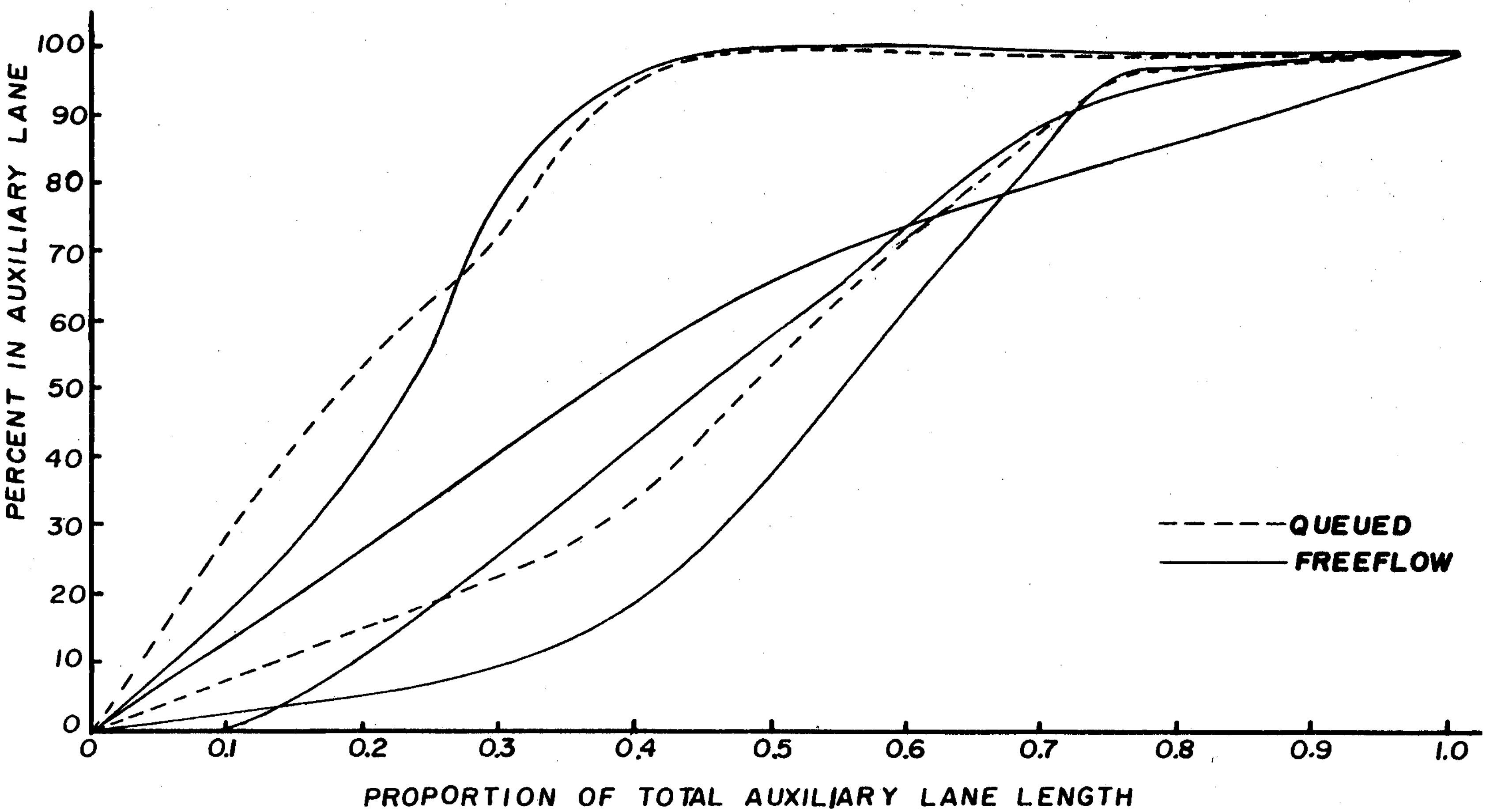
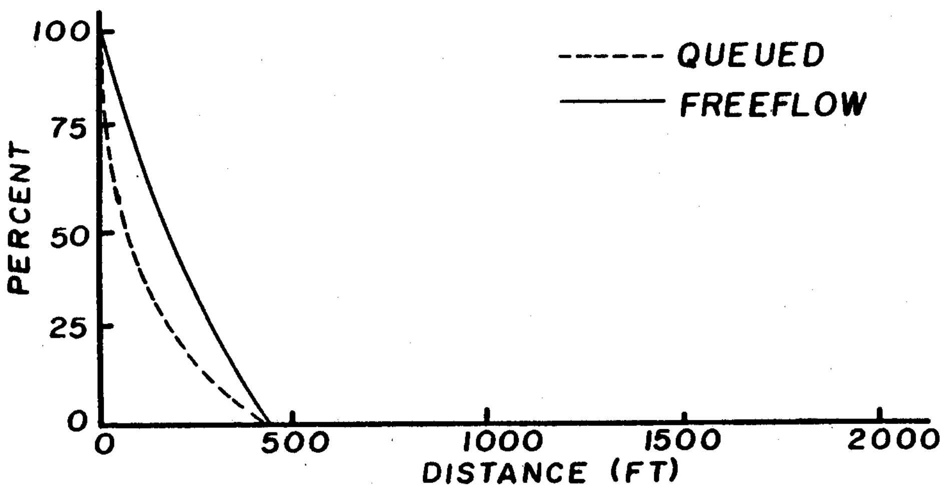
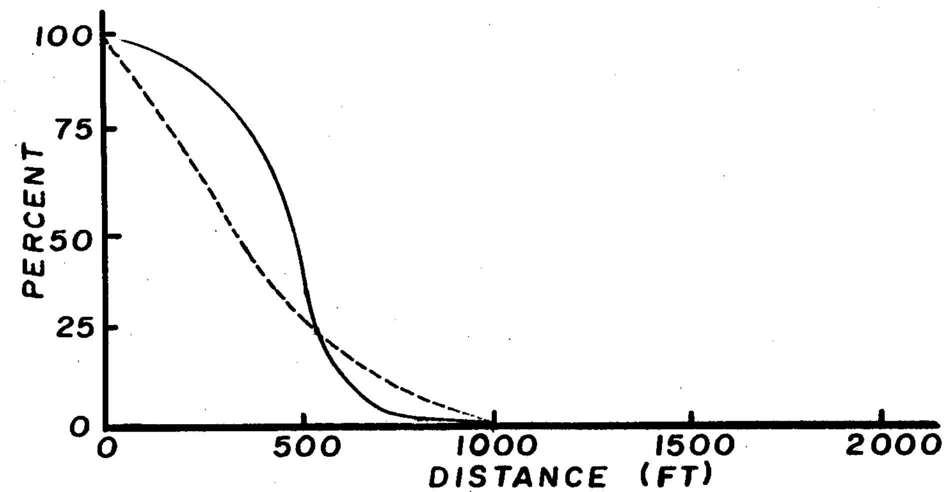


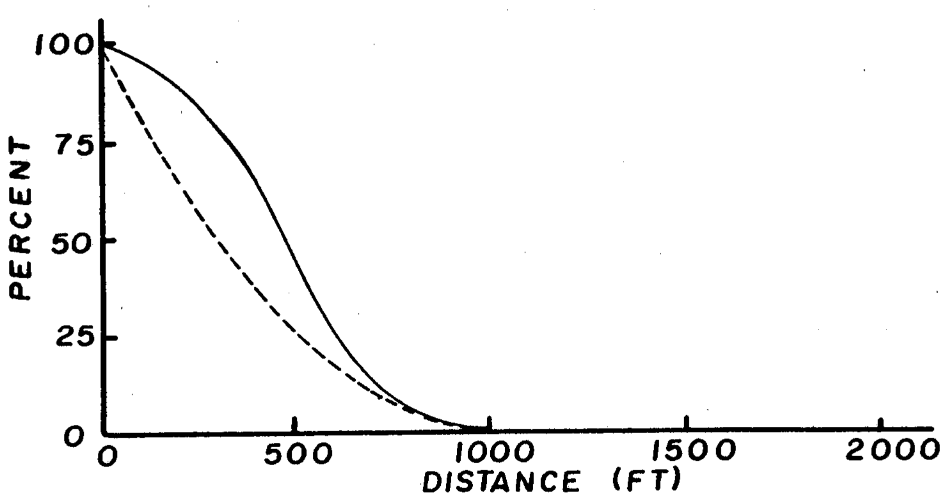
FIGURE 3. COMPARISON OF EXITING DISTRIBUTION AT ALL SITES



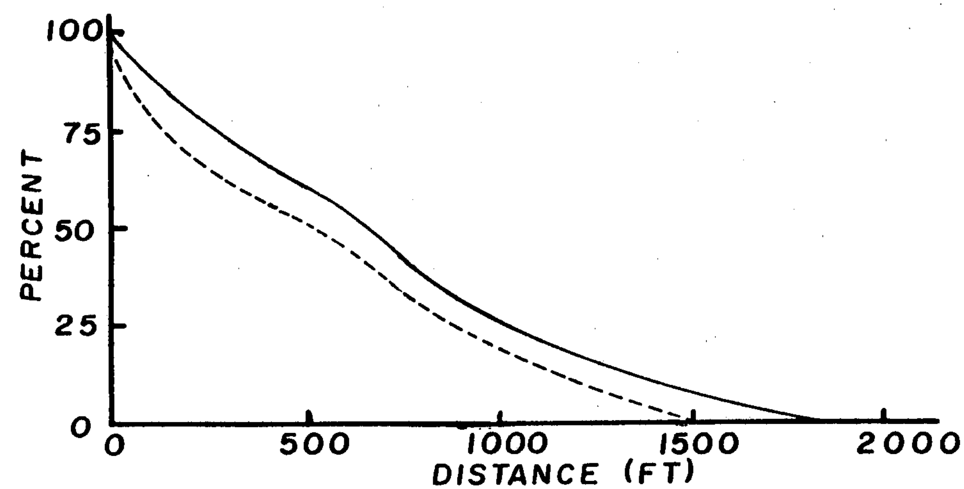
(A) I-10 EASTBOUND (L=425-FT)



(B) I-10 WESTBOUND (L=935-FT)



(C) I-35 NORTHBOUND (L=2072-FT)



(D) I-410 WESTBOUND (L=2230-FT)

FIGURE 4. DISTRIBUTION OF ENTERING TRAFFIC IN AUXILIARY LANE

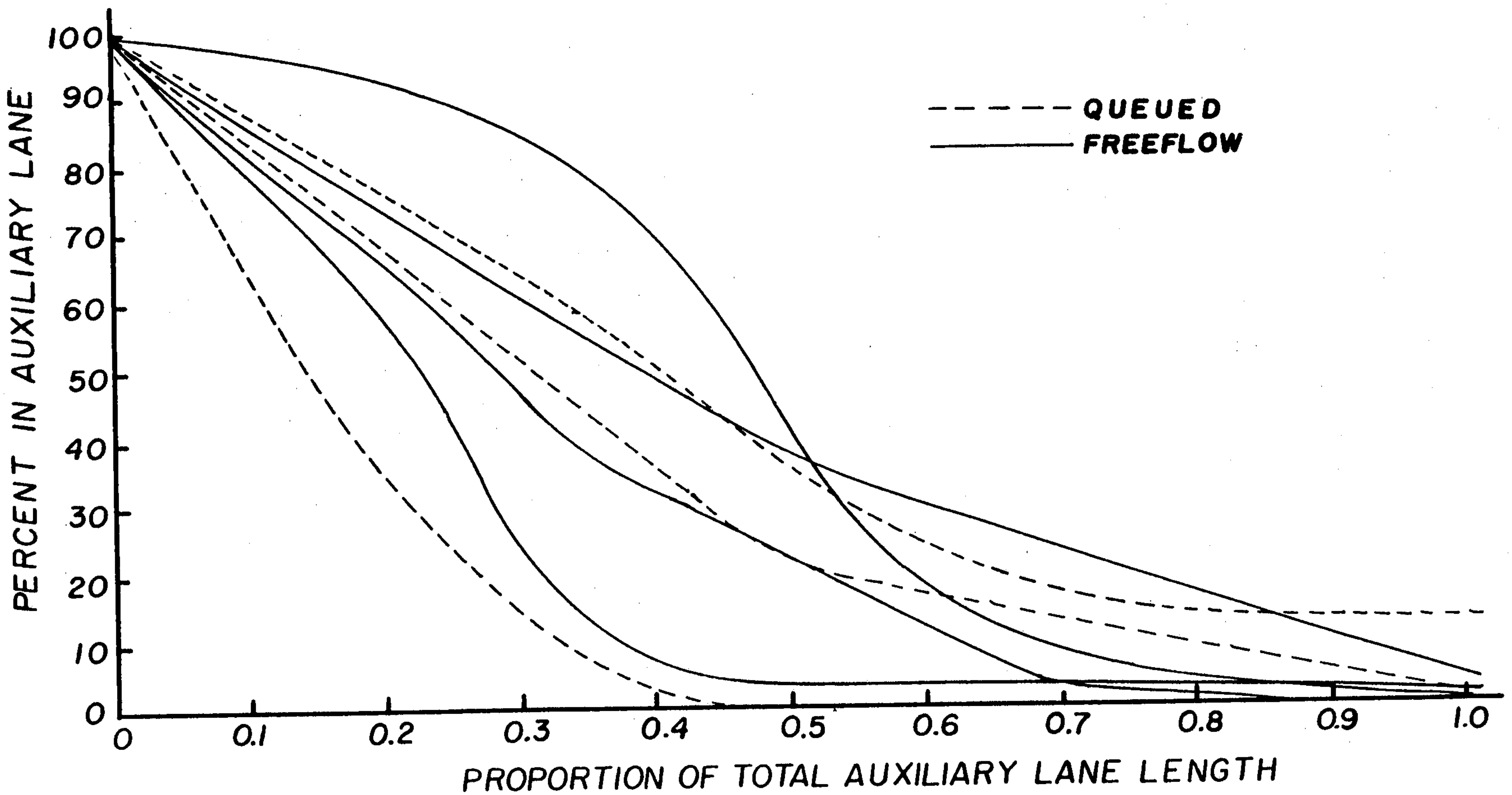


FIGURE 5. COMPARISON OF ENTERING DISTRIBUTION AT ALL SITES

of entering vehicles for the majority of the sites follow the same basic trend. The curves for that of I-35 Northbound and I-10 Westbound do not follow the same trend as the others. More vehicles tended to stay in the auxiliary lane for a longer time period on the I-10 Westbound auxiliary lane than at any of the other sites. This is most likely due to the geometric nature of this particular site. The entrance ramp to this auxiliary lane is a direct ramp from I-410 and not a ramp from a frontage road, as is true of all other sites. This allows for higher speeds of vehicles as they enter the auxiliary lane and begin to merge with the mainlane traffic. This particular site also had a high volume of thru traffic on the auxiliary lane. This is due to the large number of vehicles coming from I-410 whose destination is the Callaghan exit from I-10.

Other Characteristics

There were several other characteristics of the auxiliary lanes used for the field studies which could not be noted under either the freeway performance or weaving characteristics categories. The auxiliary lanes on I-410 Westbound and I-10 Westbound occasionally acted as storage for the signal queue from the intersection of the frontage road and the arterial. The queues, whose maximum length observed extended 10 vehicle lengths into the auxiliary lane, cleared with each green phase to the frontage road. From limited observations of this occurrence, it did not appear to affect the operation of the auxiliary lane. However, it should be noted that such a queue may cause operational problems for an extremely short auxiliary lane.

Under queued conditions, some drivers did not use the auxiliary lane throughout its full potential. Vehicles entering the freeway would sometimes stop completely near the end of the gore area and wait for a gap large enough to enter the freeway mainlanes. When this occurred, other vehicles queued up behind the stopped vehicle would cross over the gore area, causing the stopped vehicle to wait still longer for an acceptable gap. The drivers using this method of entering the freeway appeared more likely to "force" themselves into the mainlanes than most others. This phenomena of erratic maneuvers was observed most frequently on I-35 Northbound.

AUXILIARY LANES DURING INCIDENT CONDITIONS

After approximately one hour of data collection during the PM peak on February 23rd, an accident occurred in the mainlanes of the study section located on I-35 Northbound. Although the major portion of the data collection (volume counts, speeds, and weaving movements) was halted, visual observations continued. The left lane of this two-lane section of freeway was impassible to all traffic. Only the right mainlane was used by vehicles to pass this incident. Since this accident occurred at approximately the half-way point in the auxiliary lane, ample distance existed for vehicles to pass the incident. No observations of vehicles using the auxiliary lane in this capacity were made for the approximate 25 minutes during which the left lane was closed. This is one benefit of long auxiliary lanes which is not often used by motorists.

CONCLUSIONS

This study served to help identify the operational problems of auxiliary lanes of closely spaced entrance-exit ramp pairs on high volume urban freeways. The most notable conclusions concern the weaving characteristics of the entering and exiting vehicles. The results indicate that during congested flow, exiting vehicles enter the auxiliary lane sooner than during free flow conditions. This is most likely due to the nature of drivers to avoid excessive delay by using the congestion free auxiliary lane when exiting a freeway under queued conditions. Conversely, entering traffic tends to stay in the auxiliary lane longer during high-speed free flow operations than during periods of queued traffic flow. During queued conditions, entering drivers must "force" into the mainlanes by stopping and waiting for an acceptable gap. Also, the stopping of these vehicles may cause impatient drivers behind them to cross the entrance ramp gore area to enter the freeway. The study also suggested that under congested flow, auxiliary lanes of sufficient length may act as storage for the signal queue from the frontage road without hindering their operation. Long auxiliary lanes may also be used to avoid an incident or lane closure of the mainlanes more quickly, although such usage was not observed during this study.

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EVALUATION OF MINOR FREEWAY MODIFICATIONS - RAMP REVERSAL

INTRODUCTION

Many of the urban freeways in Texas were designed and constructed more than 20 years ago. Since that time, their designed capacities have been surpassed by current demands. The resultant congestion traditionally was countered with an expansion of the freeway system in terms of the number of lane-miles. However, the costs of constructing new facilities has increased at a rate greater than inflation. Consequently, alternatives other than the expensive construction of new facilities are desired. The State Department of Highways and Public Transportation (SDHPT) has implemented comparatively inexpensive programs to improve the existing freeways' ability to move people and goods.

Need of Study

As land development within freeway corridors increases, several changes to the transportation system occur, including increased traffic congestion, increased accident potential, and reduced operating speeds at exit ramps, entrance ramps, the intersections of frontage roads and arterials, and on the main lanes. In response, the SDHPT has implemented comparatively low-cost improvements within the freeway right-of-way such as new ramps, grade-separated ramps, and frontage road U-turns. In addition, it has modified the ramp configurations via ramp relocations and ramp reversals for the purpose of reducing vehicular queues at critical locations. The common purpose for each of these low-cost improvements is to maximize vehicular movement while minimizing cost.

It is generally accepted that these improvements can ease freeway congestion. However, because of funding and personnel constraints, it is crucial that the various improvements and alternative solutions can be easily prioritized according to the expected cost-effectiveness. The techniques currently employed tend to focus on the primarily-affected traffic stream. This can result in neglecting the negative effects imparted on a nearby secondary traffic stream. Consequently, there is a need for a technique that can be used to prioritize improvements while addressing the effects to both the primary and secondary traffic flows.

Purpose of Study

Recently, ramp additions and ramp relocations have been evaluated to assess the resultant benefits and to formulate a streamlined procedure for analyzing the cost-effectiveness of such minor freeway modifications. Although a detailed discussion of these other improvements is not within the scope of this report, full details of these evaluations may be found in the references listed in Appendix A. This study is directed toward a different minor freeway modification: ramp reversal, i.e., the replacing an exit with an entrance or vice versa. The reasons for studying ramp reversals are as follows: (1) to identify, quantify, and document all road user benefits that accrue from reversing the ramps; and (2) to develop a streamlined procedure for estimating the cost-effectiveness of a particular ramp reversal project before its implementation.

Study Procedure

To accomplish the objective of identifying road user benefits, the conditions and obstacles that prevent all the vehicles on the freeway and its frontage roads from traveling at free flow speeds must be considered. If reversing the ramps reduces the effect that such obstacles, e.g., queues, impart on the traffic stream, then a benefit is effected. When identifying benefits, disbenefits must also be considered and identified. Once the types of benefits are known, a method of measuring them and including them in a procedure to estimate the cost-effectiveness of reversing the ramps is necessary.

When striving to develop the cost-effectiveness evaluation procedure, it is necessary to identify the input parameters required to determine whether a particular ramp reversal project is worthwhile. These will include traffic data such as vehicular delay, peak-period volume, daily volume, percent trucks, and estimated volume of rerouted vehicles. These data will provide the basis for determining the benefits and disbenefits expected to result from the reversal of the ramps. An estimate of the construction costs, coupled with the net benefit will provide a benefit/cost ratio that quantifies the cost-effectiveness, provided that the project's amortization period and the capital recovery rate are known.

Benefit Types

The road user benefits derived from such minor freeway modifications include savings in four distinct areas: a) vehicle running costs, b) travel time costs, c) delay and idling costs, and d) accident costs. Quantification of these elements involves placing dollar values on time and on running, idling, and accident costs. The quantification of all the benefits made in this study are based on dollar values defined by the 210-5 report, "An Economic and Environmental Analysis Program Using the Results For the FREQ3CP Model" (1).

Running Cost Savings - These savings are based on the cost of operating a vehicle at the predominant operating speed, plus the cost of slowing or stopping at any intersection along the study route. Winfrey's (2) speed change cycle costs are used in determining the cost of slowing or stopping at any intersections. Running cost savings are calculated as the difference between the running cost to vehicles before the ramp reversal construction and those after the construction.

Travel Time Cost Savings - The savings in travel time is a function of the vehicle occupants' time value expressed in dollars. The time value of money used in the analysis in this study is based on the 210-5 report findings.

Delay and Idling Cost Savings - These savings are derived from the decrease in standing delay experienced at study area intersections. The average delay and idling times per delayed vehicle are recorded before construction. The average delay and idling times in the post-construction period are determined by assuming a linear relationship between pre-construction delays and volumes. The ratio of these two values is applied to the estimated post-construction volumes to determine post-construction delay.

Accident Cost Savings - In a true before-and-after study, the analysis would be performed after a sufficient amount of time for a post-construction accident rate to be established. However, because the cost-effectiveness evaluation procedure must be applicable prior to the construction, no realistic estimate of post-construction accidents can be made. Historic data from similar sites may provide some indication of the magnitude and direction of any expected change. Nevertheless, accident cost savings are not included in this procedure.

Disbenefits

Figure 6 illustrates the pre- and post-construction routes taken by vehicles whose drivers wish to enter or exit the main lanes. When comparing the routes taken before and after the ramps were reversed, it is apparent that some motorists will be forced to encounter an additional intersection which they had not traveled through before the construction. These motorists

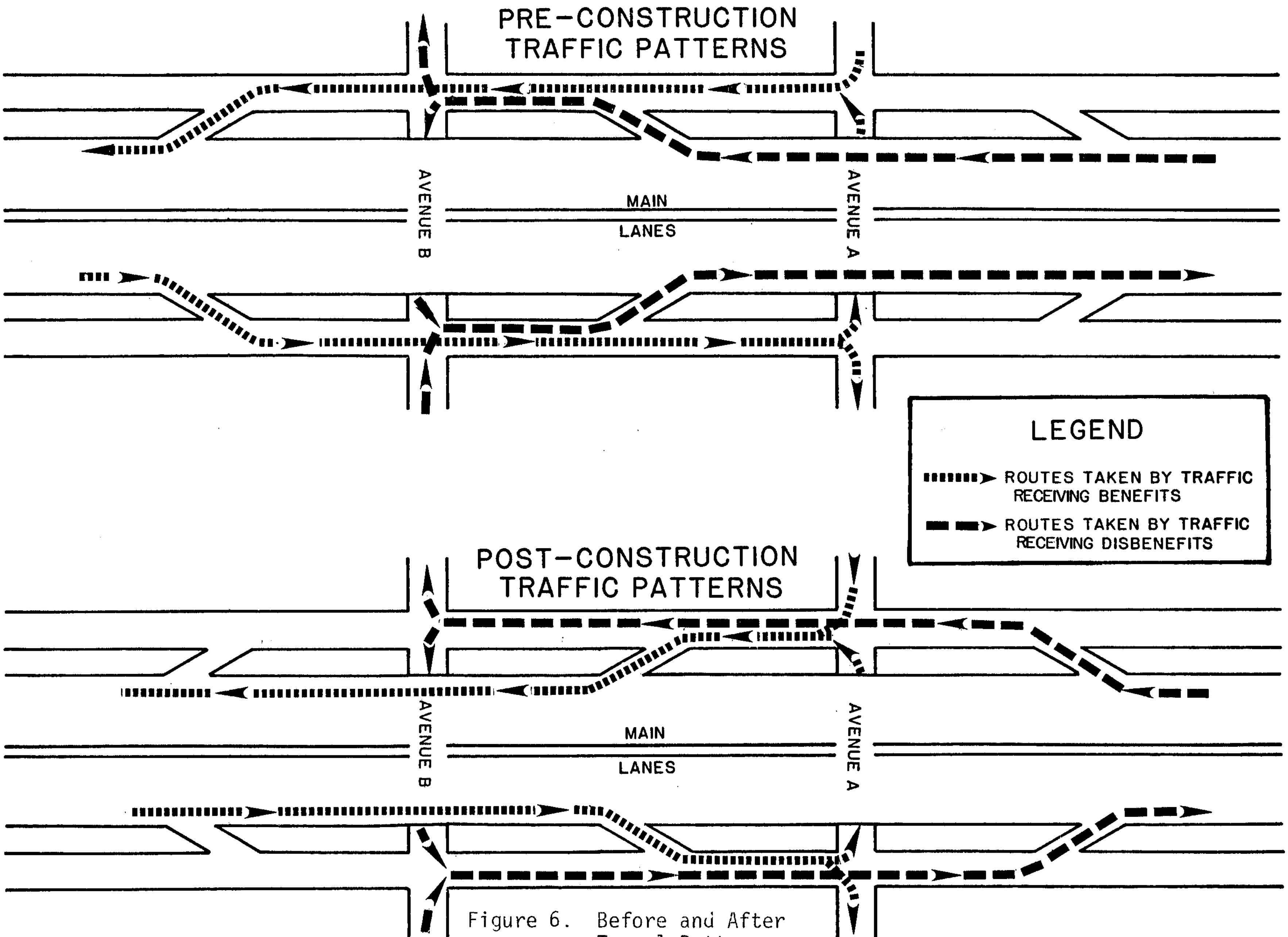

Figure 6. Before and After Travel Patterns

receive a disbenefit resulting from having to operate at a slower speed on the frontage road than on the main lanes and from being delayed at the intersection by virtue of awaiting the movement of a queue or other intersection-related delays.

It is apparent that ramp reversals cannot be cost-effective if the disbenefits are so great that they outweigh the benefits. Consequently, ramp reversals cannot be implemented indiscriminately.

Data Requirements

To quantify the benefits, as well as the disbenefits, several types of data must be available. For each of the eight approaches among the four intersections, the traffic data that are required in the pre-construction period are as follows:

1. daily volume
2. peak hour volume
3. percentage of trucks
4. vehicular delay
5. rerouted traffic volume

Daily Volumes and Peak Hour Volumes

The daily volumes and the peak hour volumes are easily collected via recording counters located at the frontage road approaches to the four intersections and at the four arterial exterior approaches (i.e., the four arterial approaches which are outside the freeway right-of-way).

Percent Trucks

Knowing the percentage of trucks in the stream is necessary because trucks have higher operating costs than passenger cars. The percentage of trucks should be determined by a classification count in the peak period at each of the approaches, or at least, at a representative intersection.

Vehicular Delay

Vehicular delay can be determined using the point sample method as described in detail in A Technique of Measurement of Delay at Intersections (3). This technique involved making counts of all stopped vehicles on an approach at 15-second intervals. The 15-second counts approximate a weighted average of delay time per stopped vehicle. This weighted average serves as the average delay and idling time necessary to calculate delay and idling cost savings. The data reduction time required for the point sample method is approximately one-eighth that of the input-output data collected with an event recorder. The accuracy of the point sample method is generally within a few percent of that of the event recorder.

Rerouted Traffic Volume

The final type of traffic data necessary as input into the cost-effectiveness evaluation procedure is the traffic volume that will be rerouted as a result of the exit ramp being replaced with an entrance or vice versa. An effective data collection technique for estimating such volumes is the lights-on study during the peak hour. This is accomplished by placing a temporary sign on the frontage road upstream of an exit ramp or an intersection. The sign's message instructs all motorists to turn on their headlamps for the next mile. At a point downstream of the sign, an observer records the number of vehicles with and without their lights on. In this way, the paths of those vehicles which passed by the sign can be identified. Moreover, the volume of traffic which arrived at the downstream point of interest with their headlamps on can be quantified. A lights-on study is useful in estimating the number of vehicles which would use a modified route if a freeway ramp were reversed. Figure 7 illustrates how the lights on study allows the observer to determine the number of vehicles that would benefit by replacing an exit ramp with an entrance ramp. If that ramp reversal were implemented, the frontage road traffic bound for the freeway main lanes would be able to enter the faster freeway lanes sooner and could avoid the intersection at the arterial cross street.

With all the traffic data collection completed, the benefits of the ramp reversal can be determined. The only remaining input information to go into the cost-effectiveness evaluation are the project amortization period and the capital recovery rate. These will be used in calculating the benefit/cost ratio. For highway construction projects such as this, a 20-year life and 10 percent interest rate can serve as default values if more specific information is unavailable.

COST-EFFECTIVENESS EVALUATION PROCEDURE

With the data collection completed, the user of this procedure can begin to combine these data in the manner illustrated in Figure 8 .

Beginning with the peak hour volumes at the eight approaches prior to construction and the results of the lights-on study, the peak hour volumes that can be expected after the ramp is reversed can be estimated. The combining of these data also leads to the determination of the rerouted peak hour volumes. These rerouted volumes can also be thought of as differential volumes since it is these vehicles which will receive most of the benefits or disbenefits from the ramp reversal project. The other vehicles will not have the opportunity to alter their routes after construction is complete and

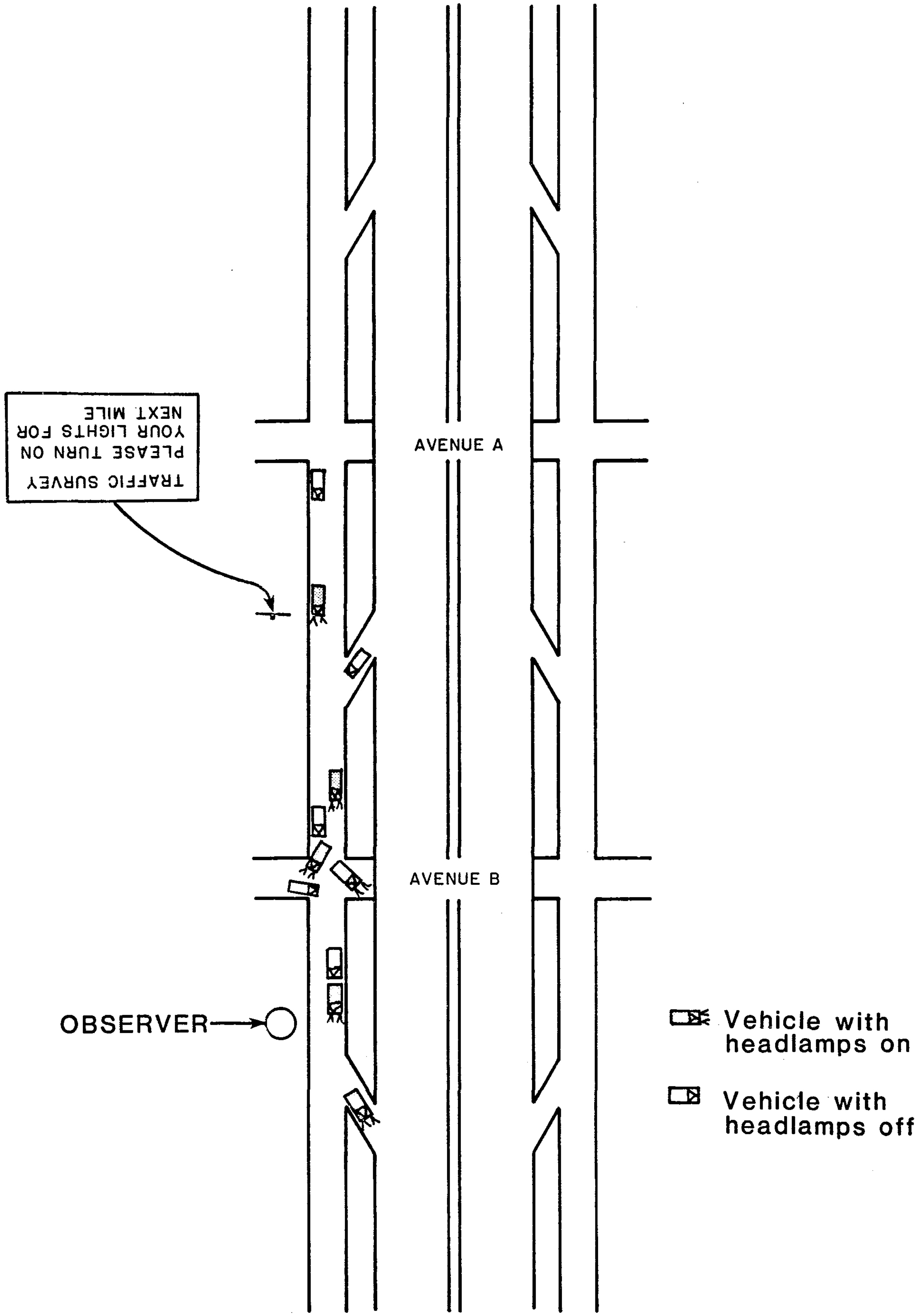

Figure 7. Lights On Study

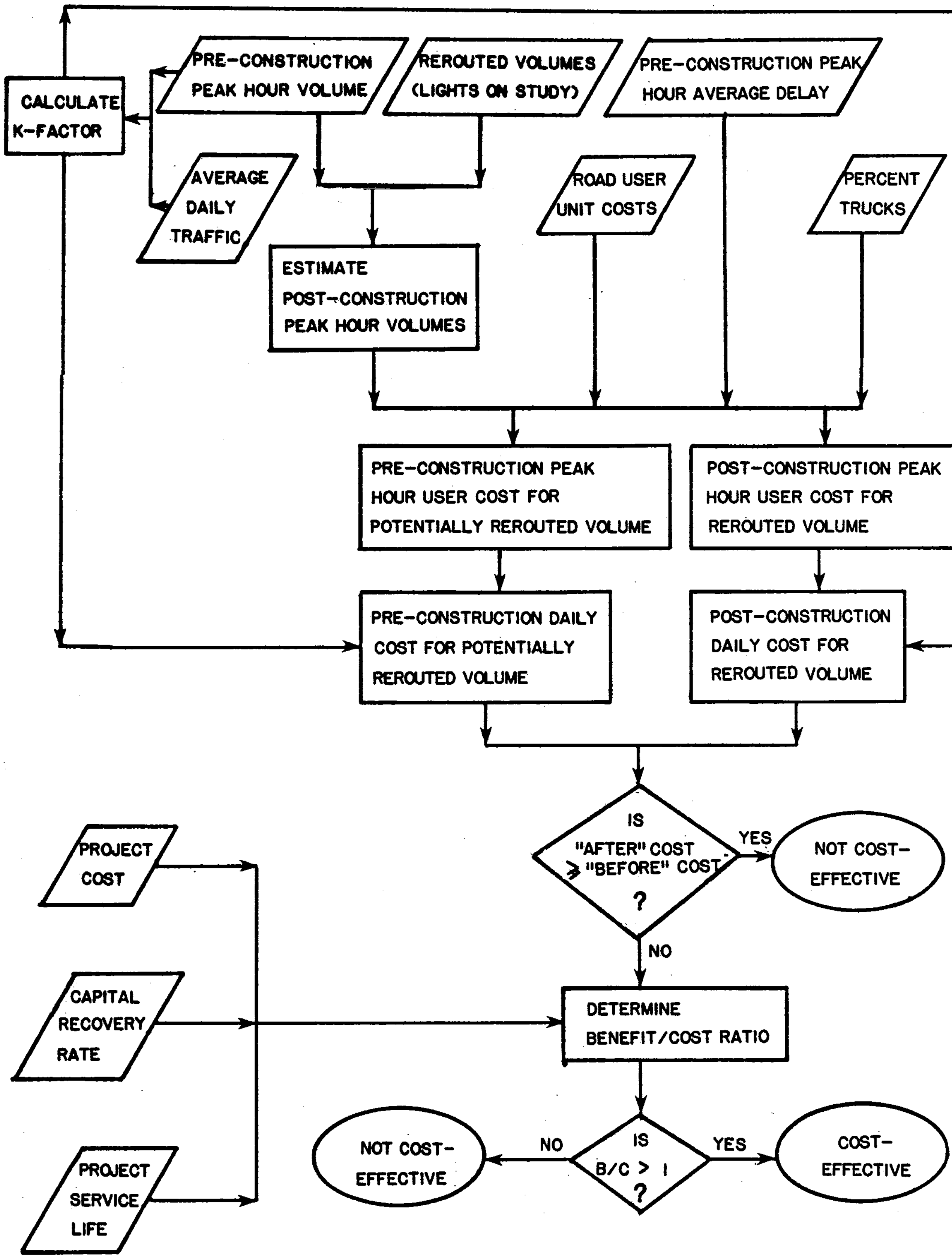

Figure 8. Cost-Effectiveness Evaluation Procedure

will not experience any benefit or disbenefit related to running costs or travel time costs. They will, however, be affected in terms of delay-related benefits or disbenefits.

Pre- and Post-Construction Peak Hour Costs

After the rerouted peak hour volumes are determined, they can be combined with the percent trucks, road user unit costs, and the average delay in the peak hour prior to construction. The analysis of these data will result in a peak hour cost for the potentially rerouted vehicles before construction. Because truck operators' time is more expensive and truck acceleration/deceleration costs are higher than those of passenger cars, the percent trucks should be measured or estimated to determine the appropriate cost rates. Before the delay cost rate can be applied, the delay must be considered. Because a change in the volume of any approach to a diamond interchange effects the delays on other approaches to that interchange, the overall interchange delay, as opposed to just the frontage road approach delay, must be addressed.

A method for estimating the total peak hour delay at an interchange in the post-construction period involved combining the estimated post-construction interchange approach volumes and the pre-construction interchange approach volumes and average delays. If a linear relationship between interchange volume and interchange delay is assumed, then the combining of these data is relatively simple and will not require the use of some computer programs that may be able to provide a more accurate estimate.

Once the total peak hour delays at each interchange for both the pre- and post-construction periods is known, total delay costs for the peak hour for these four scenarios (two interchanges and two ramp configurations) can be estimated.

The running costs and the travel time costs reflect benefits or disbenefits to only those vehicles which will be rerouted by the ramp reversal project. If the approximate typical speeds on the main lanes and on the frontage roads are known, and if the number of intersection approach vehicles which slow down to various speeds as they pass through the intersections are known, then the running costs for the pre- and post-construction periods can be estimated. In a similar manner, the travel time costs can be determined if the typical speed, the distance, the number of rerouted vehicles, and the unit cost is known.

Summing the running and travel time costs for the affected vehicles to the interchange delay costs for the interchanges involved provided a total cost for the peak period operation of the system for the before and after conditions.

Vehicles Receiving Benefits vs. Vehicles Receiving Disbenefits

In determining the peak hour cost in the pre-construction period for two groups of rerouted vehicles, the same procedure is used. One of these groups of vehicles will be those which will receive benefits by being rerouted from the frontage road to the main lanes; the other group of vehicles will be those which will receive disbenefits by being rerouted from the main lanes to the frontage road.

These same two rerouted volumes are used again in the two post-construction peak hour cost calculations. However, those vehicles which traveled on the main lanes in the pre-construction period will travel on the frontage road in the post-construction period and vice versa.

K-Factor and Daily User Costs

Figure 9 illustrates that about half the procedure has been accomplished at this point. The next step involves converting the newly calculated peak hour costs for the pre- and post-construction periods into daily costs for those rerouted vehicles. The first step in doing this is to calculate the k-factor, i.e., the ratio of the peak hour volume to the average daily traffic. Because the intended use for this k-factor in this cost-effectiveness evaluation procedure is to translate the peak hour costs for all directly affected vehicles into a daily cost for the whole system, a composite k-factor will serve satisfactorily. This factor is termed "composite" because it represents the peak hour/daily volume ratio for the whole system rather than each intersection or approach. Therefore, this composite k-factor is determined by the ratio of the sum of the peak hour volumes on the frontage road approaches at all the intersections to the sum of the daily volumes on those same approaches. With this single resultant k-factor, the previously determined peak hour costs can be translated into the daily costs for the whole system for the pre- and post-construction periods.

With the daily costs for these two scenarios estimated, the comparison of the pre- and post-construction road user costs can be performed. If the post-construction user cost is greater than the pre-construction user cost, then the construction project will not be beneficial and should not be implemented. However, if the reverse is true, the construction costs must be accounted for and the benefit/cost ratio should be calculated.

Prior to calculating the benefit/cost ratio, the service life of the project and the capital recovery rate must be estimated. In addition, the difference in the user costs in the before and after period must be calculated

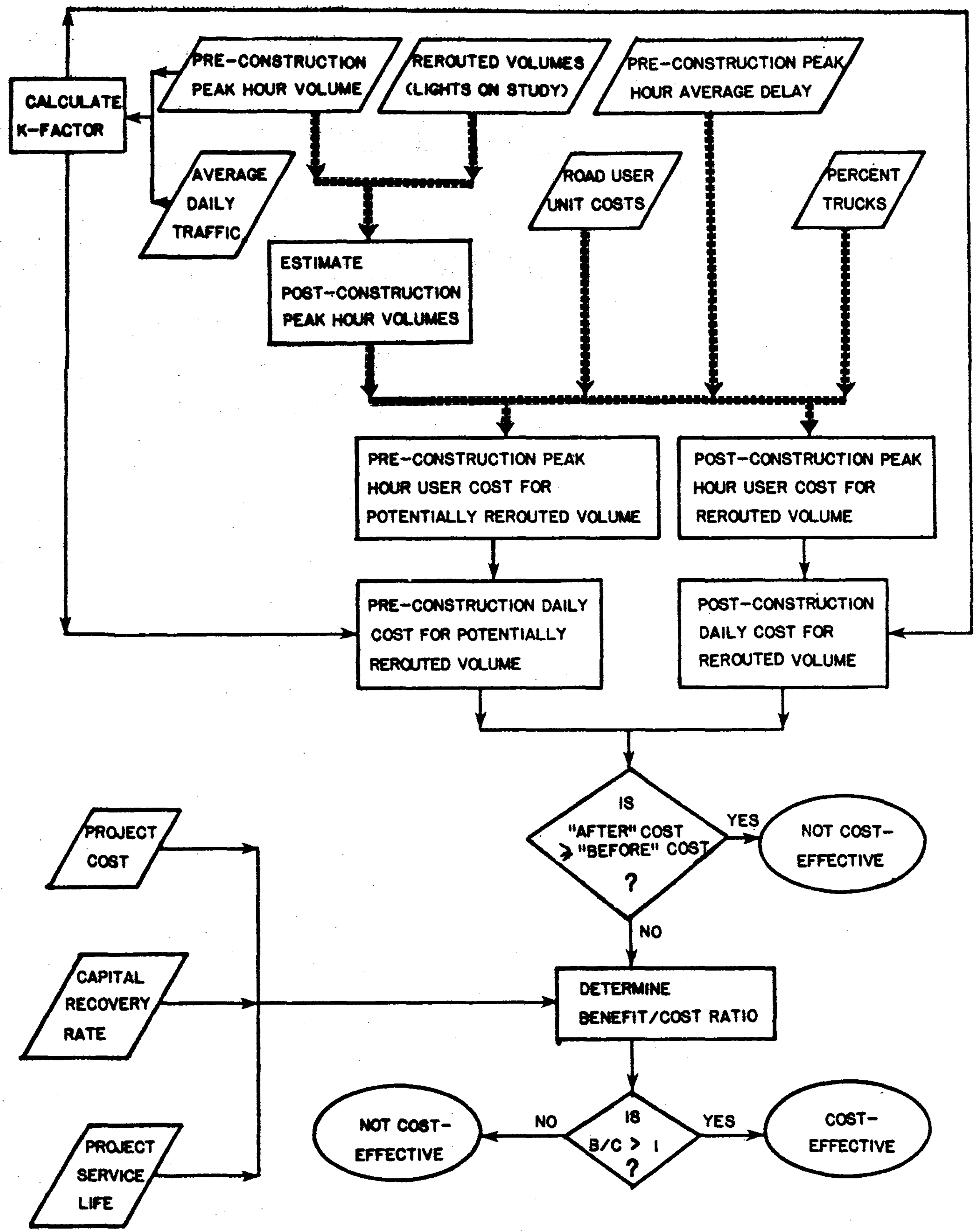

Figure 9. Portion of Cost-Effectiveness Evaluation Procedure Following Peak Hour User Cost Determination

and expanded to an annual basis. With this information, the benefit/cost ratio can be calculated. If it is greater than one, then the project will save more money than it costs and should be implemented, as illustrated at the bottom of Figure 8.

I-610 AT WALLISVILLE ROAD AND U.S. 90

In northeast Houston, Wallisville Road intersects I-610 just south of the U.S. 90 (McCarty Road) intersection with 610. As shown in Figure 10, there is no exit ramp to the northbound frontage road or entrance ramp from the southbound frontage road between these arterials. This geometry requires northbound drivers bound for U.S. 90 to take the Wallisville Road exit and pass through the Wallisville Road intersection. Additionally, southbound drivers originating from U.S. 90 must pass through the Wallisville Road intersection prior to entering the main lanes. Because of the presence of a bayou that crosses I-610 between Wallisville Road and U.S. 90, there is no room for both an entrance and an exit on each frontage road to exist at grade between these arterials. Consequently, the alternative of ramp reversals is worthy of investigation.

The obvious benefit provided in reversing the ramps between Wallisville Road and U.S. 90 will be to northbound vehicles bound for U.S. 90 and southbound vehicles originating from U.S. 90 and bound for the main lanes. However, reversing these two ramps will increase delay, idling, and travel time for northbound vehicles originating from Wallisville Road bound for the main lanes and for southbound main lane traffic bound for Wallisville Road. Because there is this trade-off, in the economic analysis the magnitude of the benefits must be decreased by the magnitude of the disbenefits prior to comparing the net benefit to the cost. The cost-effectiveness evaluation procedure developed in the preceeding section of this report is applicable to this potential ramp reversal project.

Figure 11 illustrates the p.m. peak hour (4:45 to 5:45) volumes for all the ramps and frontage road approaches in the system for both the pre- and post-construction conditions. A lights-on study indicates that 710 of the 1250

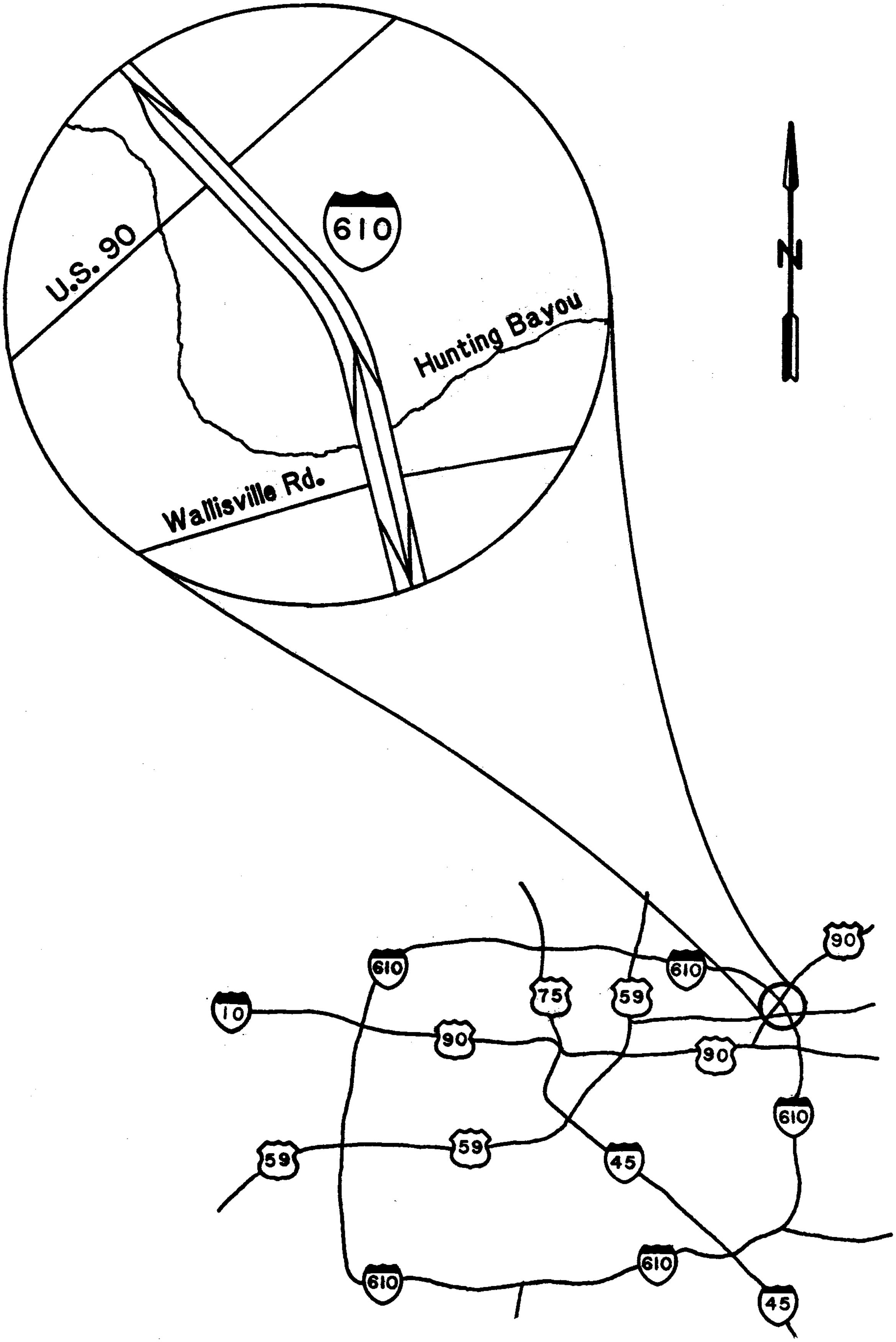

Figure 10. Houston Study Site

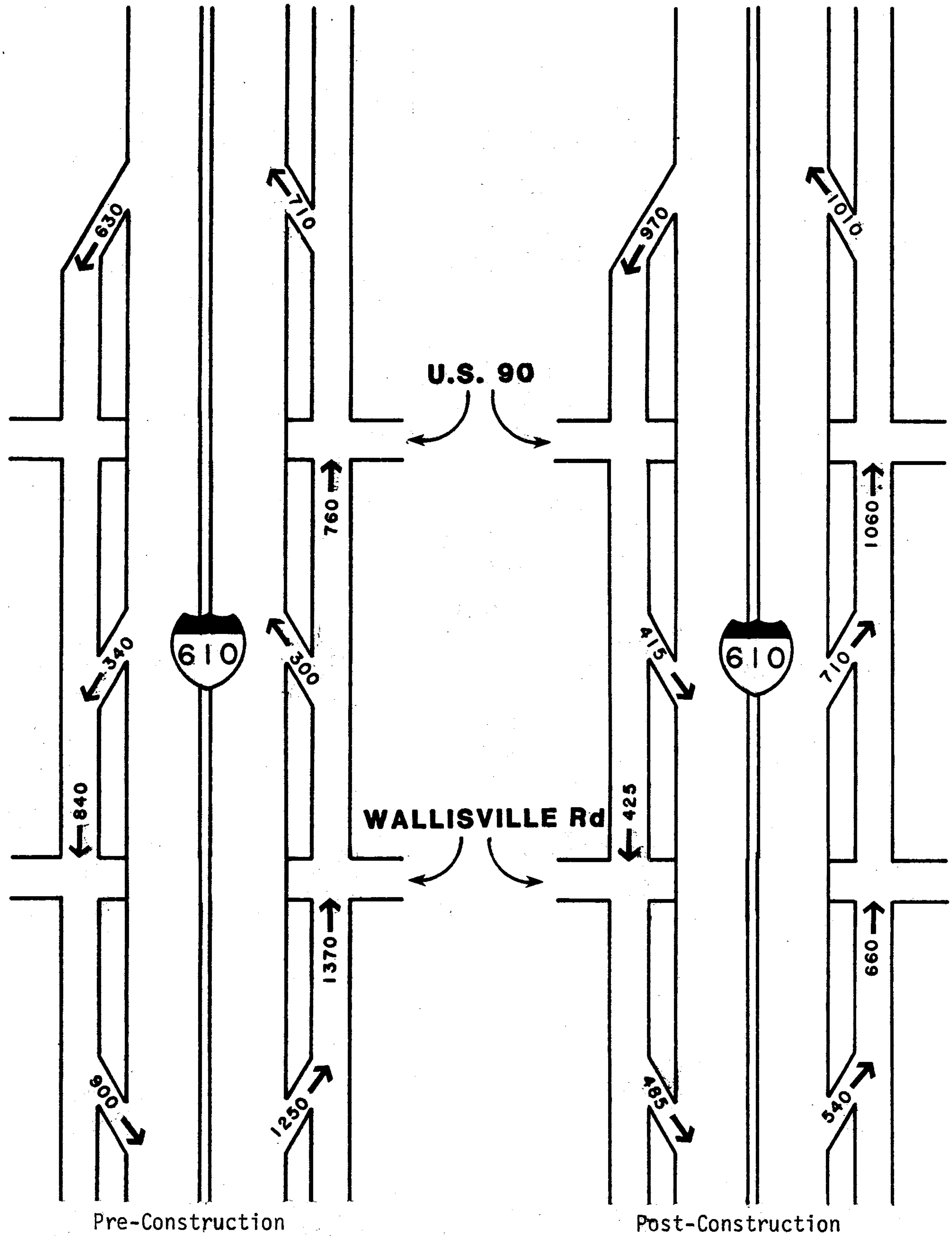

Figure 11. P.M. Peak Hourly Volumes

northbound vehicles which presently exit at Wallisville Road are actually bound for the U.S. 90 intersection; therefore, these vehicles would use the new northbound exit ramp if it were built. On this same side of the freeway, the peak volume data reveal that 300 vehicles use the existing entrance ramp. These vehicles would be forced to travel through the U.S. 90 intersection to get to the U.S. 90 entrance ramp if the ramp reversal project were implemented. These are the vehicles that would receive the disbenefits.

On the southbound side of the freeway, the lights on study indicates that 415 of the 900 vehicles in the p.m. peak hour would use the new entrance ramp if it were built and the volume counts reveal 340 vehicles would no longer be allowed to exit at Wallisville Road and thus would have to exit upstream and travel through the U.S. 90 intersection.

These four volumes (710, 300, 415, and 340 vph) are the rerouted volumes that receive either direct benefits or disbenefits by virtue of reversing the two ramps. In calculating running costs for these four volumes, two types of costs must be determined and summed: operating speed cost and speed change cycle cost. To calculate the operating speed cost for the 710 northbound vehicles which presently must leave the main lanes at the Wallisville Road exit, a running speed of 35 mph is estimated. The percent truck data indicates that there are 8 percent single unit trucks and 17 percent tractor-trailers. So, to obtain an average vehicle running cost based on the unit costs which are used in this study, the following calculation results:

passenger car	(\$0.11/veh-mi) (0.75)	=	\$0.08/veh-mi
single unit truck	(\$0.25/veh-mi) (0.08)	=	\$0.02/veh-mi
tractor trailer	(\$0.35/veh-mi) (0.17)	=	\$0.06/veh-mi
average			\$0.16/veh-mi

Knowing that the distance involved is 0.26 mi, the peak hour operating speed cost for these 710 potentially rerouted vehicles is $30. To get the runnning cost for these vehicles, the speed change cycle cost must now be determined.

Based on field observation, it is estimated that 50 percent of the approach vehicles came to a stop at the Wallisville intersection while 30 percent slowed down to 20 mph and 20 percent slowed just a small amount from 35 to 30 mph. By applying the 8, 17, and 75 percent vehicle type distribution figures to the unit costs, the average vehicle costs in Table 6 are generated.

Table 6. Speed Change Cycle Unit Costs

Speed Reduction	Passenger Car	SU Truck	Tractor-Trailer	Average
35 to 30 mph	$ 0.008	$ 0.024	$ 0.109	$ 0.026
35 to 20 mph	0.020	0.061	0.266	0.065
35 to 0 mph	0.036	0.109	0.446	0.111

By applying the 50, 30, and 20 percent figures to the three values in the last column of Table 6, an overall average cost of $0.0805 is obtained which can be applied to the 710 vehicles to result in a speed change cycle cost of $57 for these vehicles in the peak hour.

Summing the operating speed cost and the speed change cycle cost, the running cost becomes $87 and is reflected in Table 7 as are all the running costs in the pre-construction period.

The travel time costs are more easily obtained. By applying the unit cost source's dollar values for time to the already identified vehicle type percentages, the following is obtained:

passenger	($ 6.31/veh-hr)	(0.75)	=	$4.73/veh-hr
single unit truck	($11.72/veh-hr)	(0.08)	=	$0.94/veh-hr
tractor-trailer	($16.36/veh-hr)	(0.17)	=	$2.78/veh-hr
average				$8.45/veh-hr

Table 7. Pre-Construction P.M. Peak Hour User Costs

Traffic (Volume of Potentially Rerouted Vehicles)	COSTS			
	Running	Travel Time	Delay	Total
Northbound				
Entering (300)	$ 23.60	$ 23.28		$ 46.88
Exiting (710)	86.70	51.16		137.86
Southbound				
Entering (415)	57.55	29.90		87.45
Exiting (340)	26.74	26.38		53.12
Wallisville Interchange (all approaches)			$170.76	170.76
U.S. 90 Interchange (all approaches)			143.83	143.83
Total	$194.59	$130.72	$314.59	$639.90

Table 8. Post-Construction P.M. Peak Hour User Costs

Traffic (Volume of Rerouted Vehicles)	COSTS			
	Running	Travel Time	Delay	Total
Northbound				
Entering (300)	$ 47.68	$ 36.58		$ 84.26
Exiting (710)	33.00	32.56		65.56
Southbound				
Entering (415)	19.29	19.03		38.32
Exiting (340)	51.93	41.46		93.39
Wallisville Interchange (all approaches)			$ 76.52	76.52
U.S. 90 Interchange (all approaches)			215.02	215.02
Total	$151.90	$129.63	$291.54	$573.07

Assuming that the passenger occupancy rate is 1.2 per vehicle and using the \$6.31/hr rate for passenger (this does not depend on vehicle type), the overall average time value is determined as follows:

$$\left(\frac{\$8.45}{\text{veh-hr}}\right)\left(1.0\ \frac{\text{driver}}{\text{vehicle}}\right) + \left(\frac{\$6.31}{\text{veh-hr}}\right)\left(0.2\ \frac{\text{non-driver}}{\text{vehicle}}\right) = \$9.70/\text{veh-hr}$$

With this \$9.70/hr time value, and the known distance which the 710 potentially rerouted vehicles will travel, and their running speed along the frontage road, their travel time is calculated as follows:

$$(710\ \text{veh})\left(\frac{0.26\ \text{mi}}{35\ \text{mph}}\right)(\$9.70/\text{veh-hr}) = \$51.16$$

This travel time cost is also reflected in Table 7. The other running and travel time costs can be determined in this same manner.

The delay costs in Table 7 are the only other road user costs considered. These costs are those associated with the whole interchange rather than just the potentially rerouted vehicles since all the motorists in the interchange will be affected by a sizeable change in one or more of the approach volumes. The Wallisville interchange delay cost is determined by aggregating all the vehicle-seconds of delay in the interchange for the peak hour (53,380 veh-sec). Then the \$9.70/veh-hr value of time is applied to produce a delay cost of \$144. This and the U.S. 90 interchange delay are included in Table 7.

The post-construction road user costs are determined in a similar manner. These are illustrated in Table 8. Since the post-construction total cost is less than the pre-construction total cost, it cannot yet be determined that the project is not cost-effective. The difference between the total peak hour costs in these two tables is \$67. Applying the system-wide k-factor to this will result in daily cost savings for the system of ramps, main lanes, and frontage roads.

The k-factor is determined by the ratio of the sum of the peak hour frontage road approach volumes (Figure 11) to the sum of the daily frontage road approach volumes (Figure 12). This results in a k-factor of 0.075, which, in turn, results in a daily savings of $891 over the whole system. On an annual basis, the user cost savings is $222,800, as shown in Figure 13.

To determine the benefit-cost ratio, a service life of 20 years and a capital recovery rate of 10 percent are assumed. The cost of removing one ramp and replacing it with another is estimated to be $250,000. So, with a total project cost of $500,000, the annualized construction cost is $58,700. Consequently, the benefit/cost ratio is 3.8, which means that this project on I-610 is cost-effective and should be implemented.

Conclusion

Where freeway geometry makes it difficult or impossible to construct both an entrance ramp and an exit ramp between two cross streets, only one ramp can be built. Without the other ramp, access to or from the main lanes is prohibited. After several years of operation, the traffic demands may indicate that an exit ramp ought to be replaced by an entrance ramp, or vice versa. However, reversing ramps must not be done without sufficient analysis to determine if the resulting benefits outweigh the resulting disbenefits and cost. Because of the need for determining the cost-effectiveness of a ramp reversal project, a procedure has been developed and applied to the Wallisville Road/U.S. 90 area on I-610 in Houston, Texas. In addition to the development of the cost-effectiveness evaluation procedure, the types of benefits and disbenefits were addressed.

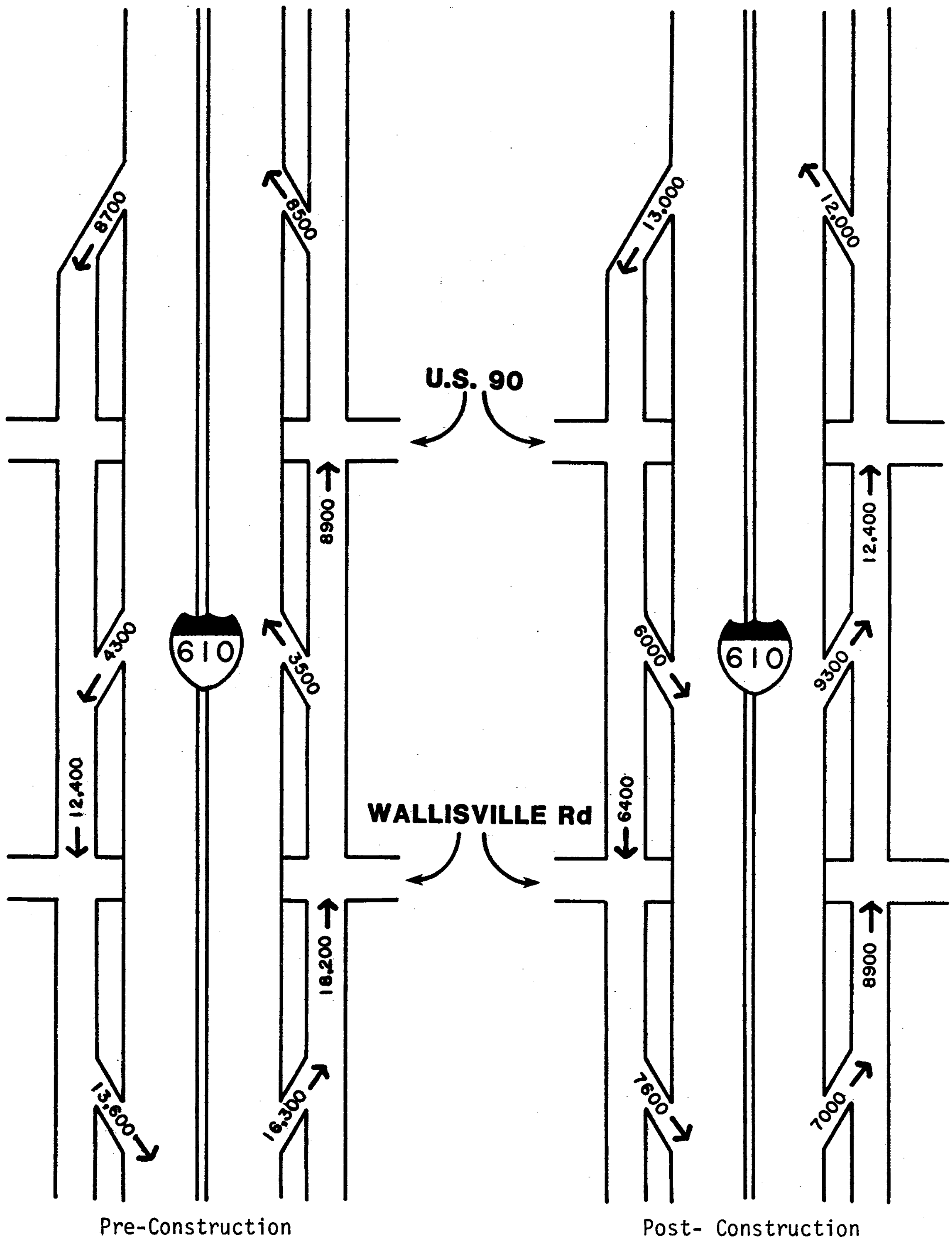

Figure 12. Average Daily Traffic Volumes

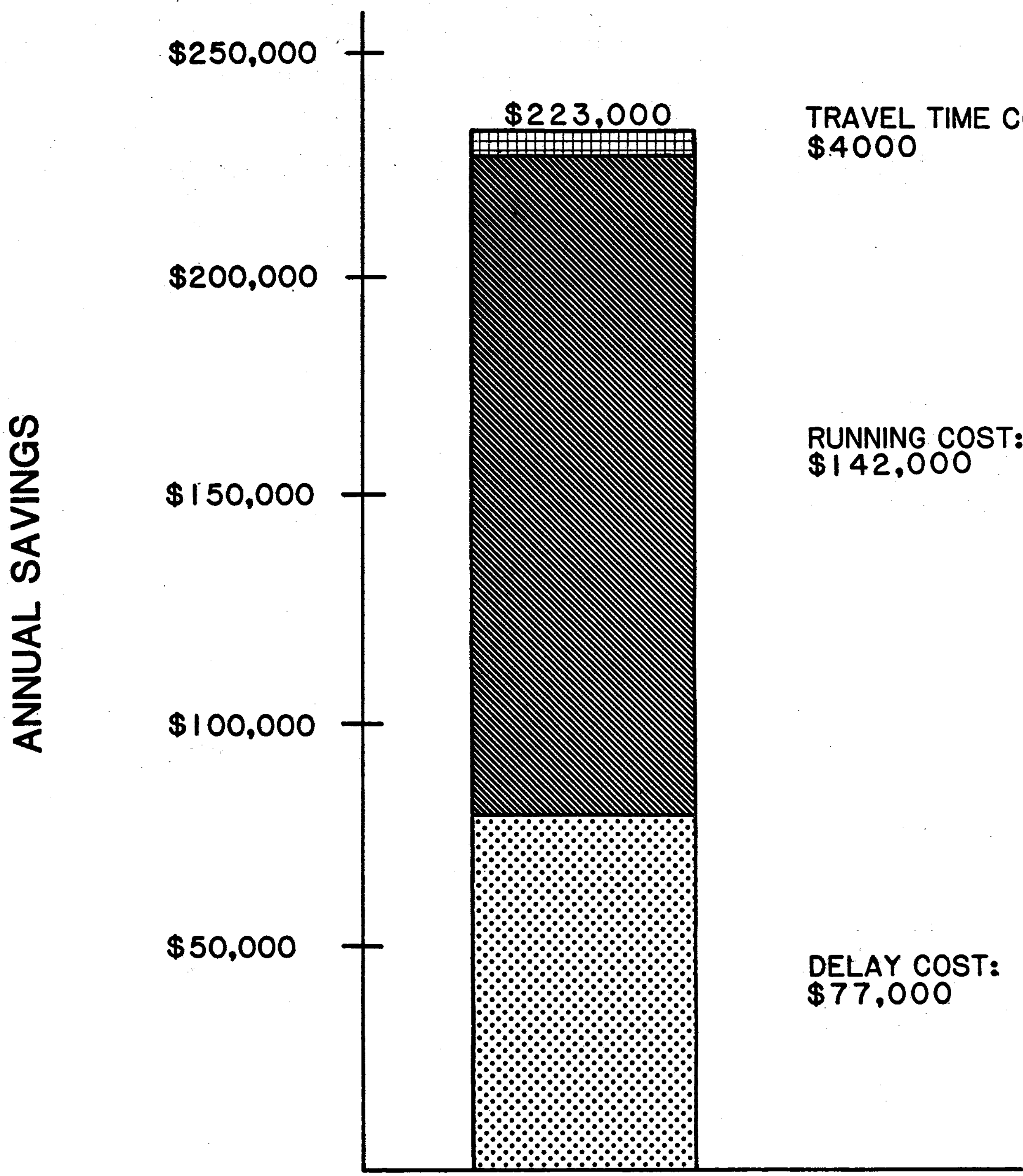

Figure 13. Cost Savings Derived from Ramp Reversal Project

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EVALUATION OF MINOR FREEWAY MODIFICATIONS

GRADE-SEPARATED RAMPS TO ELIMINATE WEAVING

INTRODUCTION

Report 210-11 included a detailed analysis of the grade-separation of ramps to provide additional access. Another purpose for grade-separating ramps is to separate flows, thereby eliminating a weaving section. This treatment would normally be applied in areas where weaving creates significant operational problems or has produced a serious accident history. This report is a case study of a ramp pair in San Antonio that was grade-separated to eliminate weaving.

FREDERICKSBURG ROAD/I-10 CONNECTOR GRADE SEPARATED RAMPS

Prior to 1980, a short weave existed between the Fredericksburg Road entrance ramp to I-410 and the connector to I-10 eastbound in northwest San Antonio. Figure 14 shows the geometric alignment before and after the grade-separation of the ramps. Heavy peak period volumes (Figure 15) produced significantly degraded operations. Because this project was implemented prior to the inception of this research study, all of the analyses are based on historical, rather than measured, data. Where estimates were necessary, they were made such that the actual benefits would be equal to or greater than those estimated.

Operations Experience

Northbound I-410 vehicles bound for the I-10 connector were regularly queued in the right lane due to the weaving conditions. Operating speeds in the right lane of I-410 were typically 30 miles per hour (mph) for at least 1,000 feet upstream of the merge. In addition to the 1,095 exiting vehicles,

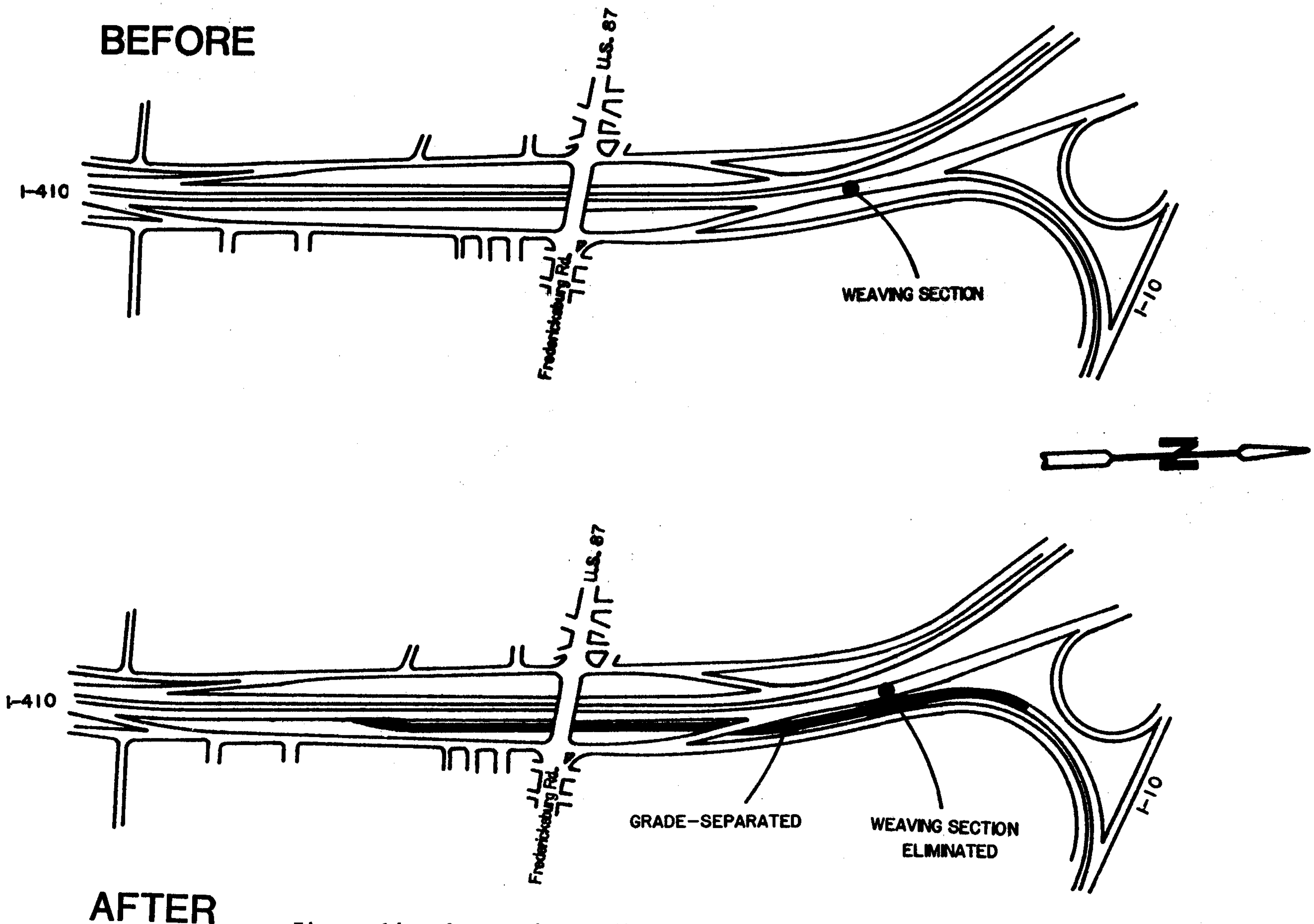


Figure 14. Geometric Configuration of Ramps at Study Site

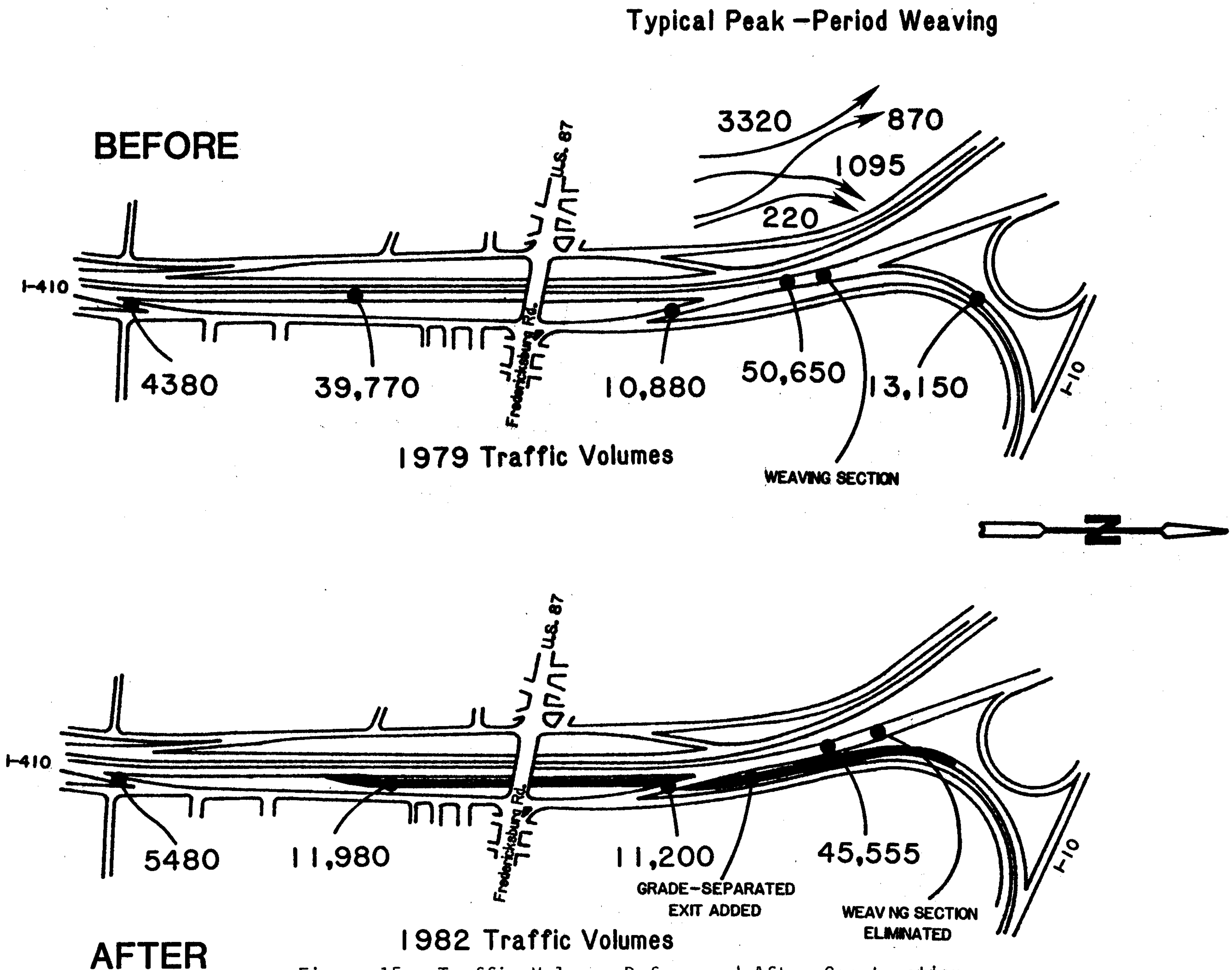


Figure 15. Traffic Volumes Before and After Construction

about 800 through vehicles could also be expected to use the outside lane, according to the Highway Capacity Manual (1). Therefore a total of about 1,895 vehicles experienced excess travel time and operating costs.

Entrance ramp traffic from Fredericksburg Road frequently queued into the intersection. About 20 percent of the entering traffic from Fredericksburg Road stayed in the auxiliary lane and exited to I-10. The average speed for entrance ramp traffic is assumed to be about 20 mph throughout the peak period. The resulting flow patterns are diagrammed in the upper right on Figure 15.

Table 9 shows some estimates of the expected p.m. peak savings in travel time and operating costs. The p.m. peak period traffic received the most benefits, although the a.m. peak traffic volume was assumed to be about 90 percent of the p.m. peak. Under those assumptions, user savings approached $260 per day, or $65,000 per year. These estimates are probably considerably lower than the actual savings. They also presume that there were no other adverse impacts on the main lanes, nor any benefits during the off-peak.

Accident Experience

While the above savings are important, the reduction in accidents was more important. Figure 16 shows the accident experience, by milepoint, for 18 months before and 18 months after the ramps were modified. Accidents included in this analysis were either in the right lane or on the entrance ramp. The separation of the flows resulted in a 71 percent reduction in accident frequency. Over this 0.5 mile section of freeway the accident rate dropped from 1.69 accidents per million vehicle-miles (MVM) to 0.55 accidents per MVM for the main lanes alone. If the traffic and accidents on the

Table 9. Estimated Daily P.M. Peak Savings in Travel Time and Operating Costs

Affected Roadway Section	Affected Length	Operating Speed Before	Operating Speed After	Unit Savings	Peak Volume	Daily Savings*	Annual Savings*
Right Lane of I-410	1,750'	30 mph	50 mph	3.45¢/veh	1,895	$ 65.38	$16,345
Entrance Ramp from Fredericksburg Road	1,450'	20 mph	50 mph	6.48¢/veh	1,090	70.63	17,658
Total	- -	- -	- -	- -	- -	136.01	$34,003

* Shows only P.M. peak savings. Other analyses assumed A.M. peak savings of about 90 percent of P.M. peak.

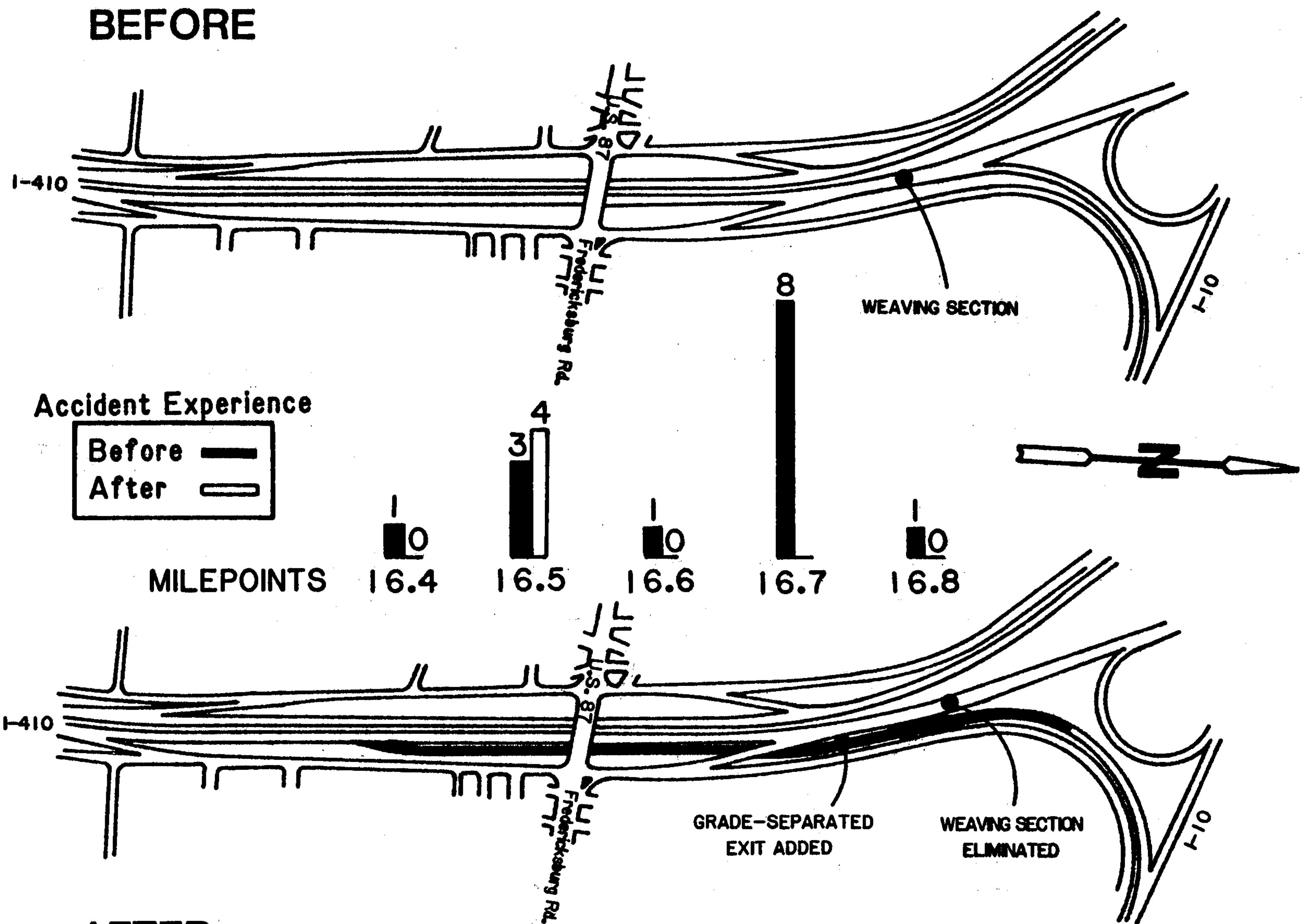


Figure 16. Accident Experience during 18-Month Intervals Before and After Grade-Separation of Ramps

newly-constructed I-10 connector are included in the after analysis, the accident rate drops even further to 0.42 accidents per MVM.

An estimate of before and after accident costs for the same time frames was prepared. Prior to grade-separating the ramps, approximately 35 percent of the accidents were injury accidents (estimated at $22,350 each), and 65 percent property damage only ($995 each) (2). The sampling period did not include any fatal accidents. Based on these 1975 cost estimates (updated to 1980 using Consumer Price Indexes) (2), the annual accident costs in the section were about $79,000 prior to the modification. Accident costs after the modification were estimated at $16,000 per year. Annual road user savings in accident costs of about $60 - 65,000 are estimated.

Total Annual Savings

Total savings in operating and accident costs are estimated at approximately $130,000 per year. If no growth were experienced in this freeway section, the present worth of the annual benefits of this modification would be about $1,107,000. The construction cost specifically related to this modification could not be exactly determined because it was a part of several area improvements. Previous estimates of $800,000 - $1,000,000 appear reasonable to assign to the grade-separation. Therefore, it could be concluded that the benefit/cost ratio would be reasonably close to 1:1.

However, growth has occurred in the section, at about 4.5 percent per year since 1979. If growth were projected at that rate until the main lanes approach capacity (about 6.25 years), and if the unit savings remains fairly constant, then the present worth of future benefits approaches $1,500,000. Thus a peak period benefit/cost ratio in excess of 1:1 is virtually certain.

CONCLUSION

The intent of this case study has not been to document measured conditions, but rather to estimate some probable results after-the-fact. In such an analysis there is considerable room for error. However, the estimates used were fairly conservative and applied only to the peak periods. It seems very unlikely that the present worth of the savings would be less than the cost of construction.

On the other hand it has also been shown that, due to the cost of grade-separated ramps, it is unlikely that accident reduction alone can provide economic justification for grade-separated ramps. Such construction must also provide extensive savings in operating, travel time or delay costs to be justified.

REFERENCES

1. Highway Capacity Manual, Highway Research Board, Special Report 87, 1965.
2. McFarland, W. F., _et al_., "Assessment of Techniques for Cost-Effectiveness of Highway Accident Countermeasures." U.S. Department of Transportation, Federal Highway Administration, Report No. FHWA-RD-79-53, January 1979.

APPENDIX A

Previous Reports

"Evaluation of Urban Freeway Modifications"
Research Study 2-18-77-210

210-1 "Automatic Detection of Freeway Incidents During Low-Volume Conditions" D. B. Fambro, G. P. Ritch, September, 1979; 71 pp. (PB81-180408)

210-2 "The Use of Freeway Shoulders to Increase Capacity" W. R. McCasland; September, 1978; 51 pp. (PB300952)

210-3 "Analyzing the FREQ3CP Freeway Operations Simulation Model" G. P. Ritch, J. L. Buffington; October, 1978; 106 pp. (PB301117)

210-5 "An Economic and Environmental Analysis Program Using the Results for the FREQ3CP Model" J. L. Buffington, G. P. Ritch; September, 1981; 151 pp.

210-6 "LVID - Software Documentation" G. P. Ritch; March, 1980; 152 pp. (Not Published)

210-7 "Feasibility Study for the Total Demand Management of the Inbound Southwest Freeway" W. R. McCasland; May, 1980; 72 pp.

210-8 "A Comparison of Freflow and FREQ3CP Optimization Models" C. W. Blumentritt, G. P. Ritch; (Not Published)

210-9 "An Application of RF Data Transmission in Freeway Ramp Metering" G. P. Ritch; September, 1981; 82 pp.

210-10 "The Use of Freeway Shoulders to Increase Capacity -- A Review" W. R. McCasland; January, 1984; 38 pp.

210-11 "Evaluation of Minor Freeway Modifications" J. A. Nordstrom, W. R. Stockton; November, 1982; 53 pp.

APPENDIX B

Calculation of Level of Service

The following is a step-by-step procedure which was followed for the determination of the level-of-service for a weaving section with an auxiliary lane. A more detailed explanation may be found in TRB Circular 212 (Ref. 2).

Step 1. Convert the 5 minute volumes to peak flow rates in passenger cars per hour (PCPH). This also involves the construction of a weaving diagram. The percentage of trucks is also determined by using the mainlane volume counts (which also included truck counts). The average peak flow rate thru each study section was calculated by

$$AC = ((INPUT + OUTPUT - ENTR - EXIT)/2) /Q \qquad (1)$$

where

AC = Average peak flow rate (PCPH)

INPUT = Mainlane flow rate before entrance ramp (veh/hour)

OUTPUT = Mainlane flow rate after exit ramp (veh/hour)

ENTR = Entrance ramp flow rate (veh/hour)

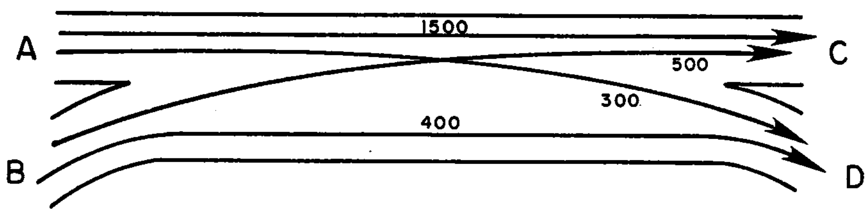
EXIT = Exit ramp flow rate (veh/hour)

Q = Commercial/recreational vehicle factor

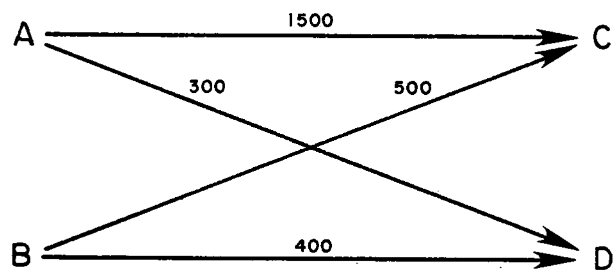
Q = 100/ (100 + % Trucks)

Step 2. Construct a weaving diagram and compute the weaving parameters as shown below.

Step 3. Assume a value for S_{NW} (speed of non-weaving vehicles). This is a trial and error procedure, and it is important that trials start with a high value and proceed toward lower speeds. A S_{NW} of 50 mph was assumed as a starting point for all calculations.



SECTION AND FLOWS



WEAVING DIAGRAM

THEN:

V_{W1} = Weaving flow with the highest numeric value (500)

V_{W2} = Weaving flow with the smallest numeric value (300)

V_W = Total weaving flow (500 + 300 = 800)

V_{o1} = Non-weaving flow with the highest numeric value (1500)

V_{o2} = Non-weaving flow with the smallest numeric value (400)

V = Total volume (500 + 300 + 1500 + 400 = 2700)

R = Weaving Ratio = V_{W2}/V_W (300/800 = 0.375)

VR = Volume Ratio = V_W/V (800/2700 = 0.296)

EXAMPLE CONSTRUCTION OF WEAVING DIAGRAMS AND COMPUTATION OF PARAMETERS

Source: Ref. 2.

Step 4. Determine S_W (speed of weaving vehicles). (2)

$$\text{LOG } S_W = 0.142 + 0.694 \text{ LOG } (S_{NW}) + 0.315 \text{ LOG } (L_H$$

where

S_W = average running speed of weaving vehicles (mph)

S_{NW} = average running speed of non-weaving vehicles (mph)

L_H = length of weaving section (hundreds of feet)

Step 5. Determine N_W(max). For ramp weaves, N_W(max), the number theoretically utilized by weaving vehicles, is 2.0.

Step 6. Determine $N_{W/N}$.

$$\text{LOG } N_{W/N} = 0.340 + 0.571 \text{ LOG (VR)} - 0.438 \text{ LOG } (S_W) + 0.234 \text{ LOG } (L_H) \quad (3)$$

where:

$N_{W/N}$ = ratio of the number of lanes theoretically utilized by weaving vehicles to the number of lanes in the weaving section

VR = volume ratio--calculated in Step 2

S_W = average running speed of weaving vehicles (mph) calculated in Step 4

L_H = length of weaving section (hundreds of feet)

Step 7. Compute $N_W = N \times (N_{W/N})$ and compare with $N_W(\text{max}) = 2.0$.

If $N_W > N_W$(max), the section is constrained (go to Step 8).

If $N_W < N_W$(max), the section is unconstrained (go to Step 9).

Step 8. Compute S_{NW}. The calculated S_{NW} may then be used to determine the level-of-service using Table 3. Calculate S_{NW} by

$$S_{NW} = \frac{1500 (N - 2.0) - (AC + BD/Q) + 1900}{50} \quad (4)$$

where

S_{NW} = average running speed of non-weaving vehicles (mph)

N = number of lanes in the weaving section

AC = average peak flow rate for thru traffic (PCPH)

BD = average peak flow rate for non-weaving traffic using the auxiliary lane (PCPH)

Q = commercial/recreational vehicle factor

<u>Step 9.</u> Determine S_{NW}. If the calculated S_{NW} is not equal to S_{NW} assumed (within + or - 2 mph), another speed must be assumed and all steps beginning with Step 3 repeated. If the two are equal, the level-of-service may be determined from Table 3. S_{NW} may be determined by

$$S_{NW} = \frac{1500 \quad (N - N_W) - (AC + BD/Q) + 1900}{50} \tag{5}$$

where

S_{NW} = average running speed of non-weaving vehicles (mph)

N = number of lanes in the weaving section

N_W = number of lanes theoretically utilized by weaving vehicles--determined in Step 7

AC = average peak flow rate for thru traffic (PCPH)

BD = average peak flow rate for non-weaving traffic using the auxiliary lane (PCPH)

Q = commercial/recreational vehicle factor

APPENDIX C

Levels-of-Service

I-35 Northbound--Main to St. Mary's

February 23, 1983

PM Peak

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	Weaving	Avg. Measured Speed (mph)
16:15	516	2822	55.4	1.0	A	A	56.7
16:20	708	1760	69.9	15.4	A	D	58.4
16:25	636	2216	63.8	9.4	A	B	52.6
16:30	816	2551	56.1	1.7	A	A	55.5
16:35	816	2611	55.2	0.7	A	A	52.3
16.40	780	3169	45.5	-5.1	B	-	55.4
16:45	744	2538	57.2	2.8	A	A	51.8
16:50	828	2461	57.5	3.1	A	A	58.4
16:55	1044	2100	61.1	6.6	A	A	53.9
17:00	852	2202	61.4	7.0	A	B	50.9
17:10	924	2826	50.4	-4.0	A	-	54.9

Note: Data collection was halted at approximately 5:20 PM due to an accident in the freeway mainlanes.

Levels-of-Service

I-35 Northbound--Main to St. Mary's

February 24, 1983

AM Peak

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	L-O-S Weaving	Avg. Measured Speed (mph)
6:50	732	2034	65.4	11.0	A	C	56.3
6.55	816	1992	65.0	10.6	A	C	59.9
7:00	540	2726	56.7	2.3	A	A	59.5
7:05	1092	2215	58.8	4.4	A	A	57.5
7:10	996	2677	52.2	-2.2	A	-	56.6
7:15	1020	2653	52.4	-2.0	A	-	56.7
7:20	1068	2785	49.8	-4.6	B	-	55.4
7:25	876	2857	50.4	-4.0	A	-	55.3
7:30	720	1807	69.0	14.6	A	C	57.5
7:35	1032	2029	62.3	7.8	A	B	61.8
7:40	828	2515	56.6	2.2	A	A	49.9
7:45	1116	2352	56.5	2.0	A	A	----
7:55	744	2275	61.5	7.0	A	B	57.0
8:00	432	2101	68.9	14.5	A	C	58.2
8:05	396	1831	73.9	19.5	A	D	62.3
8:10	552	2359	62.7	8.3	A	C	66.2
8:15	708	1531	73.1	18.7	A	D	56.4
8:20	756	1945	66.5	12.1	A	C	58.2
8:25	864	1669	69.3	14.9	A	C	61.9
8:30	648	1614	72.8	18.4	A	D	58.0
8:35	528	2047	68.1	13.7	A	C	60.7
8:40	732	1531	72.8	18.4	A	D	63.2
8:45	864	1405	72.9	18.5	A	D	61.6

Levels-of-Service

I-35 Northbound--Main to St. Mary's

February 24, 1983

PM PEAK

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	L-O-S Weaving	Avg. Measured Speed (mph)
16:05	1032	2329	57.6	3.2	A	A	54.3
16:10	1260	1987	61.0	6.6	A	B	54.9
16:15	1224	2503	53.2	-1.3	A	-	41.3
16:20	1044	2245	58.8	4.4	A	A	55.6
16:25	1272	2035	60.2	5.8	A	B	55.1
16:30	1068	2299	57.7	3.3	A	A	55.8
16.35	948	2515	55.4	0.9	A	A	56.4
16:40	1308	1825	63.1	8.7	A	B	52.0
16:45	1272	2857	46.2	-4.4	A	-	56.2
16:50	1032	2076	61.5	7.1	B	B	50.9
16:55	948	2263	59.4	5.0	A	A	56.1
17:10*	864	2767	52.1	-2.4	A	-	26.1
17:15*	456	2971	53.8	-0.6	A	-	14.1
17:20*	408	2197	67.8	13.4	A	C	15.8
17:25*	600	2881	53.2	-1.2	A	-	18.1
17:30*	240	2353	68.9	14.5	A	C	15.2
17:35*	660	2659	56.2	1.8	A	A	16.4
17:40*	396	2275	66.8	12.3	A	C	20.6
17:45*	960	2791	50.7	-3.7	A	-	29.9
17:50*	984	2125	61.2	6.8	A	B	41.5
17:55	936	1974	64.0	9.6	A	B	----

* Indicates stop-and-go mainlane traffic flow

Levels-of-Service

I-35 Northbound--Main to St. Mary's

February 25, 1983

AM Peak

Beginning Time of 5-minute Interval	Traffic Flowrates		S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S		Avg. Measured Speed (mph)
	Weaving (pcph)	Non-weaving (pcph)			Non-weaving	Weaving	
6:50	1116	2467	54.6	0.2	A	A	52.7
6:55	780	2587	56.0	1.5	A	A	60.0
7:00	1128	1939	62.8	8.4	A	B	53.4
7:05	1104	1909	63.4	9.0	A	B	50.1
7:10	1164	2605	52.0	-2.4	A	-	54.8
7:15	1104	2611	52.4	-2.0	A	-	56.4
7:20	1188	2713	50.0	-4.4	A	-	58.9
7:25	1128	2419	55.3	0.9	A	A	61.6
7:30	936	2467	56.3	1.9	A	A	59.0
7:35	1212	1645	66.4	12.0	A	C	58.9
7:40	1104	2539	53.6	-0.9	A	-	60.5
7:45	1224	2581	51.9	-2.5	A	-	----

Levels-of-Service

I410 Westbound--Broadway to Airport

May 2, 1983

PM Peak

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	L-O-S Weaving	Avg. Measured Speed (mph)
16:10	2292	3180	56.2	0.5	A	A	----
16:15	2244	2832	61.9	6.2	A	B	50.4
16:20	2244	2292	70.0	14.4	A	C	53.2
16:25	2268	3024	58.8	3.1	A	A	53.5
16:30	2184	3654	49.1	-6.6	B	-	54.4
16:35	2064	3756	48.1	-7.6	B	-	53.8
16:40	1980	3588	51.4	-4.3	A	-	52.0
16:45	2052	3432	53.5	2.2	A	A	54.6
16:50	2268	2970	59.6	3.9	A	A	53.7
16:55	2196	3534	51.0	-4.7	A	-	56.4
17:00	2184	3006	59.5	3.9	A	A	54.5
17:05	2640	4692	23.2	---	F	-	53.1
17:10	2604	3486	49.6	-6.1	B	-	49.9
17:15	2352	3486	50.9	-4.8	A	-	51.2
17:20	2100	3348	54.6	-1.1	A	-	55.0
17:25	2352	3030	58.2	2.5	A	A	53.3
17:30	2016	3342	55.2	-0.5	A	-	54.6
17:35	2448	3372	52.2	-3.5	A	-	53.2
17:40	2196	2892	61.3	5.6	A	B	53.1
17:45	2256	2856	61.5	5.8	A	B	53.8
17:50	1752	2634	68.1	12.4	A	C	54.7
17:55	1728	2922	63.9	8.2	A	B	56.9

Levels-of-Service

I410 Westbound--Broadway to Airport

May 3, 1983

AM Peak

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	L-O-S Weaving	Avg. Measured Speed (mph)
7:00	1392	2911	66.8	11.1	A	C	----
7:05	1536	2265	69.3	13.6	A	C	----
7:10	1860	2286	63.5	7.8	A	B	----
7:15	1932	3427	54.4	-1.3	A	-	----
7:20	2112	3300	55.3	-0.4	A	-	----
7:25	2136	3973	41.6	-6.1	C	-	----
7:30	1992	3462	53.4	-2.3	A	-	----
7:35	2076	3756	48.0	-7.7	B	-	----
7:40	2280	5076	18.9	---	F	-	----
7:45	2268	4416	30.1	-8.9	E	-	----
7:50	2172	3630	49.5	-6.2	B	-	----
7:55	1848	3186	58.8	3.1	A	A	----
8:00	1860	3432	54.8	-0.9	A	-	----
8:05	2160	3031	59.3	3.6	A	A	----
8:10	2196	4243	35.3	-8.2	D	-	----
8:15	2484	3618	46.8	-5.0	B	-	----
8:20	1848	3613	51.9	-3.8	A	-	----
8:25	1896	3060	60.5	4.8	A	A	----
8:30	1980	3013	60.7	5.0	A	A	----
8:35	1812	2659	67.3	11.6	A	B	----
8:40	1824	2046	76.2	20.5	A	E	----

Note: Although no speeds were measured, freeflow conditions existed throughout this study period.

Levels-of-Service

I410 Westbound--Broadway to Airport

May 4, 1983

AM Peak

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	L-O-S Weaving	Avg. Measured Speed (mph)
7:45	2064	3841	45.6	-6.2	B	-	----
7:50	2172	3607	49.9	-5.8	B	-	----
7:55	2352	3163	56.1	0.4	A	A	----
8:00	1956	3319	56.0	0.3	A	A	----
8:05	1896	3007	61.3	5.7	A	B	----
8:10	2292	3427	52.2	-3.5	A	-	----
8:15	2004	3462	53.3	-2.4	A	-	----
8:20	1836	2911	63.3	7.6	A	B	----
8:25	2268	3145	56.9	1.2	A	A	----
8:30	1860	2353	71.6	15.9	A	D	----
8:35	2004	2437	69.4	13.7	A	C	----
8:40	1908	3079	60.1	4.4	A	A	----

Note: Although no speeds were measured, freeflow conditions existed throughout this study period.

Levels-of-Service

I10 Westbound--I410 to Callaghan

May 4, 1983

PM Peak

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	L-O-S Weaving	Avg. Measured Speed (mph)
16:00	960	2665	45.1	5.7	B	C	56.5
16:05	1716	1296	65.7	23.3	A	E	53.2
16.10	1836	1663	60.7	18.4	A	D	54.3
16:15	1740	2101	53.4	11.1	A	C	52.8
16:20	1392	1819	59.0	16.6	A	D	54.8
16:25	1488	2389	49.0	6.7	B	C	53.3
16:30	2064	1206	67.2	24.9	A	E	52.8
16:35	1800	2010	51.6	9.2	A	B	52.7
16:40	1500	2473	44.6	5.2	C	E	52.8
16:45	1632	1944	54.1	11.8	A	C	50.3
16:50	1116	2671	40.3	4.0	C	C	51.2
16:55	1248	2958	35.2	2.1	D	D	52.4
17:00	1044	3331	26.1	---	F	-	52.0
17:05*	1188	3036	34.8	1.7	E	E	35.2
17:10*	864	2185	46.2	6.8	B	C	25.8
17:15*	1260	3175	34.3	1.2	E	E	24.0
17:20*	828	3139	28.9	-0.8	F	-	23.9
17:25*	1620	2424	46.9	7.5	B	C	27.2
17:30*	936	2827	36.3	3.3	D	D	27.0
17:35*	840	2743	38.0	1.7	D	D	29.5
17:40*	1092	2832	35.5	2.4	D	D	34.2
17:45*	924	2911	34.8	1.7	E	E	41.5
17:50*	960	2983	35.0	1.9	E	E	50.2
17:55*	1200	2263	43.9	4.6	C	C	----

* Indicates stop-and-go mainlane traffic flow

Levels-of-Service

I10 Eastbound--Cincinnati to Culebra

May 5, 1983

AM Peak

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	L-O-S Weaving	Avg. Measured Speed (mph)
7:00*	216	3936	41.3	13.0	C	E	23.2
7:05*	204	3720	45.9	15.2	B	E	27.4
7:10*	252	3528	48.9	15.8	B	E	22.9
7:15*	204	3354	53.1	20.0	A	D	27.2
7:20*	408	3433	48.2	15.1	B	E	21.4
7:25*	228	3366	52.4	19.3	A	D	28.3
7:30*	384	3192	52.9	19.9	A	D	26.7
7:35*	384	3066	55.2	22.2	A	E	25.2
7:40*	360	3655	44.4	13.7	C	E	33.3
7:45*	348	3217	53.0	20.0	A	D	26.9
7:50*	468	2881	57.3	24.3	A	E	24.4
7:55*	372	3126	54.3	21.3	A	E	26.5
8:00*	240	3451	50.5	17.5	A	D	29.5
8:05*	288	3703	44.7	14.0	C	E	23.6
8:10*	348	3571	46.2	15.4	B	E	32.6
8:15*	192	3114	57.8	24.7	A	E	31.1
8:20*	252	3925	40.8	12.5	C	E	42.4
8:25*	240	3517	49.3	16.3	B	E	32.8
8:30*	384	3457	48.1	15.0	B	D	40.6
8:35*	276	2725	63.2	30.2	A	E	----
8:40*	180	3187	56.7	23.7	A	E	----

* Indicates stop-and-go mainlane traffic flow.

Levels-of-Service

I10 Westbound--I410 to Callaghan

May 5, 1983

PM Peak

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	L-O-S Weaving	Avg. Measured Speed (mph)
16:15	1800	1578	66.2	23.8	A	E	----
16:20	1632	2761	46.3	6.9	B	C	52.8
16:25	1932	2035	58.3	16.0	A	D	61.7
16:30	1788	2977	41.2	4.9	C	C	59.1
16:35	1596	3439	33.5	0.4	E	E	54.5
16:40	2088	2616	46.9	7.5	B	C	54.1
16:45	2028	1884	60.5	18.1	A	D	59.4
16:50	1716	3474	30.8	1.1	E	E	57.8
16:55	1956	1770	62.2	19.8	A	D	56.2
17:00	1368	3907	25.5	---	F	-	54.4
17:05	1560	2365	54.6	12.3	A	C	53.4
17:10	1608	2658	49.4	7.0	B	C	46.9
17:15*	1512	3360	35.2	2.2	D	D	44.7
17:20*	1968	1842	61.3	18.9	A	D	37.7
17:25*	1896	3708	26.0	---	F	-	48.1
17:30*	2508	1537	64.3	21.9	A	F	45.5
17:35	1740	3757	25.9	---	F	-	48.8
17:40	1872	2676	47.0	7.6	B	C	49.8
17:45	1812	1950	59.9	15.5	A	D	51.9
17:50	1980	2040	57.5	15.1	A	D	55.4
17:55	1920	1728	63.2	20.9	A	E	----

* Indicates stop-and-go mainlane traffic flow

Levels-of-Service

I10 Eastbound--Cincinnati to Culebra

May 6, 1983

AM Peak

Beginning Time of 5-minute Interval	Traffic Weaving (pcph)	Flowrates Non-weaving (pcph)	S_{NW} (calculated) (mph)	ΔS (mph)	L-O-S Non-weaving	L-O-S Weaving	Avg. Measured Speed (mph)
7:00	240	4253	34.5	8.7	E	F	56.1
7:05	360	3961	38.4	10.1	D	F	53.0
7:10	228	3588	48.2	15.2	B	E	50.4
7:15	264	2928	59.8	26.7	A	E	52.1
7:20	270	3492	49.1	16.1	A	D	55.4
7:25	216	3480	50.5	17.4	A	D	48.1
7:30*	396	3463	47.8	14.8	B	D	33.2
7:35*	216	3235	55.0	22.0	A	E	28.0
7:40*	240	3577	48.2	15.1	B	E	----
7:45*	180	3223	56.0	23.0	A	E	36.0
7:50*	336	3390	50.0	17.0	A	D	21.0
7:55*	264	3492	49.3	16.3	B	E	27.5
8:00*	204	3349	53.2	20.1	A	E	31.8
8:05*	396	3757	41.6	13.3	C	E	32.2
8:10*	300	3595	46.5	15.8	B	E	41.0
8:15	216	3660	46.8	16.1	B	E	53.0
8:20	132	3348	54.8	21.8	A	E	50.7
8:25	180	3721	46.4	15.7	B	E	49.5
8:30	108	3043	61.3	28.3	A	E	51.0
8:35	216	3223	55.3	22.2	A	E	----
8:40	108	3409	54.3	21.3	A	E	----

* Indicates stop-and-go mainlain traffic flow