Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-0-1846-1		

4. Title and Subtitle	5. Report Date
DESIGN GUIDELINES FOR PROVISION OF MEDIAN ACCESS ON PRINCIPAL ARTERIALS	December 2000
7. Author(s) Jillyn K. O'Shea, Randy B. Machemehl, and Thomas W. Rioux	6. Performing Organization Code 8. Performing Organization Report No. Research Report 0-1846-1
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650	10. Work Unit No. (TRAIS) 11. Contract or Grant No. 0-1846
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, TX 78763-5080	13. Type of Report and Period Covered Research Report (9/99 to 8/00) 14. Sponsoring Agency Code

15. Supplementary Notes

Project conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration, and the Texas Department of Transportation.

16. Abstract

Principal arterial class streets must move large traffic volumes while providing limited property access. Guidelines for median design and other characteristics that will maintain traffic flow potential are needed. Without such guidelines, principal arterials, over time, tend to lose traffic flow potential at the expense of property access functions. These guidelines are being developed as median selection criteria that consider flush medians with no left-turn lanes, flush medians with left-turn lanes, and raised medians with limited median openings

17. Key Words:	18. Distribution Statement
Median design, property access, flush medians, raised medians, median access	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	104	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

DESIGN GUIDELINES FOR PROVISION OF MEDIAN ACCESS ON PRINCIPAL ARTERIALS

by

Jillyn K. O'Shea
Randy B. Machemehl
Thomas W. Rioux

Research Report Number 0-1846-1

Research Project 0-1846
Development of Design Guidelines for the Provision of Median Access on Principal Arterials

Conducted for the

TEXAS DEPARTMENT OF TRANSPORTATION

in cooperation with the

U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN

December 2000

DISCLAIMERS

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration or the Texas Department of Transportation. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including an art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

This report was prepared in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

ACKNOWLEDGMENTS

The authors express appreciation to Gustavo Lopez (PHR), TxDOT project director for this study, and Homer Bazan (PHR) for their guidance and support.

NOT INTENDED FOR CONSTRUCTION, PERMIT, OR BIDDING PURPOSES

Randy B. Machemehl, P.E. (Texas No. 41921)
Research Supervisor

TABLE OF CONTENTS

LIST OF FIGURES

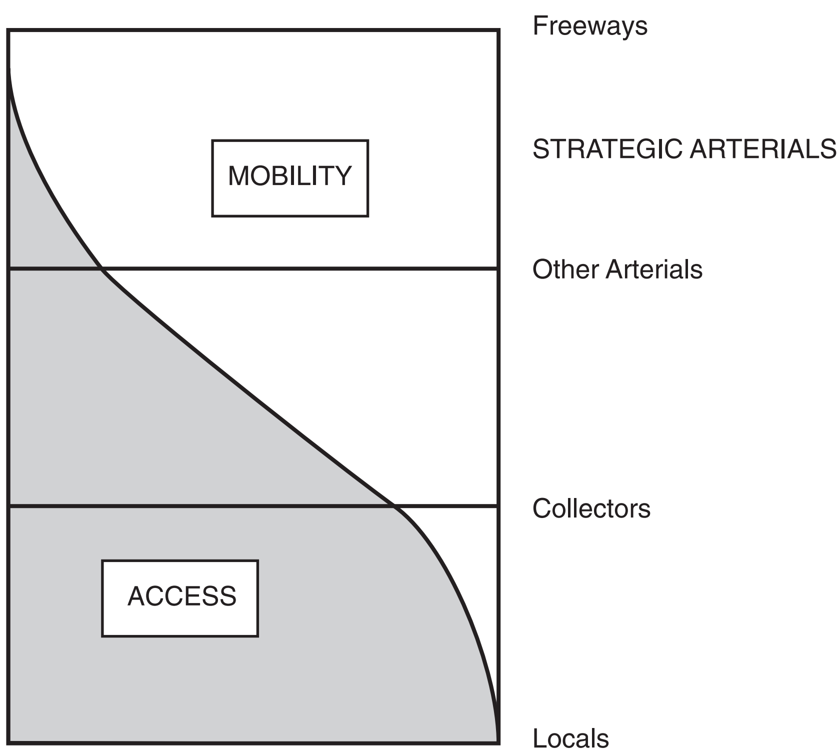
LIST OF TABLES

SUMMARY

Principal arterial class streets must move large traffic volumes while providing limited property access. Guidelines for median design and other characteristics that will maintain traffic flow potential are needed. Without such guidelines, principal arterials, overtime, tend to lose traffic flow potential at the expense of property access functions. These guidelines are being developed as median selection criteria that consider flush medians with no left-turn lanes, flush medians with left-turn lanes, and raised medians with limited median openings.

CHAPTER 1 INTRODUCTION

Public highways and streets have dual but competing roles, namely, providing property access as well as moving through traffic. Highway functional classification systems recognize the competition between access and flow, generally specifying that principal arterial streets primarily move traffic and secondarily provide access, while local streets primarily provide access and secondarily move traffic. This relationship is symbolically illustrated in Figure 1.1.



(Source: ITE Committee 6Y-19, Planning Urban Arterial and Freeway Systems, Institute of Transportation Engineers, Washington, D.C., 1988.)

Figure 1.1 Competing Mobility and Access Functions

Access provision is problematic for traffic flow because right turns and especially left turns into and out of driveways create traffic stream friction that often totally blocks through movements. Practical ways of controlling flow potential loss include limiting the number of property access driveways, restricting left-turn opportunities, and using good driveway geometric standards. While the Texas Department of Transportation (TxDOT) Design Manual addresses median design in paragraphs 4-202 G and 4-302 B, little specific guidance regarding choices between raised (curbed or barrier) versus flush (continuous one- or two-way left-turn lanes) is provided. Appropriate policies for principal arterials are urgently needed. The decision process for designing an arterial median is rather complex in that it involves a lengthy series of questions. These are presented as a stepwise decision tree in Figure 1.2. As indicated in the figure, a wide variety of different mid-block and intersection channelization features can be produced. The procedures developed through this research are intended to provide rational means to answer the questions posed in the figure.

Results of this research will provide a basis for amendment of part of current TxDOT median design policies. Current criteria are appropriate; however, they simply lack the specificity needed by busy designers dealing with property owners and developers. The study will provide specific, clear guidance reflecting safety, mobility, and economic impacts regarding:

1. Divided roadway and continuous center left-turn lane treatments,
2. Acceleration and deceleration lane design,
3. Raised and flush median treatments, and
4. Spacing between adjacent access points.

Results could be immediately applied by districts throughout the state and after field review incorporated into the Design Manual.

This research effort is divided into several sections. A literature review of median design was conducted and will be discussed in Chapter 2. Various median design scenarios were explored using computer simulation, and the experiments and results are explained in Chapter 3. Chapter 4 is an applications chapter that synthesizes the literature and simulations into a step-by-step instruction manual that can be easily used by the designer. The research conclusions are given in Chapter 5.

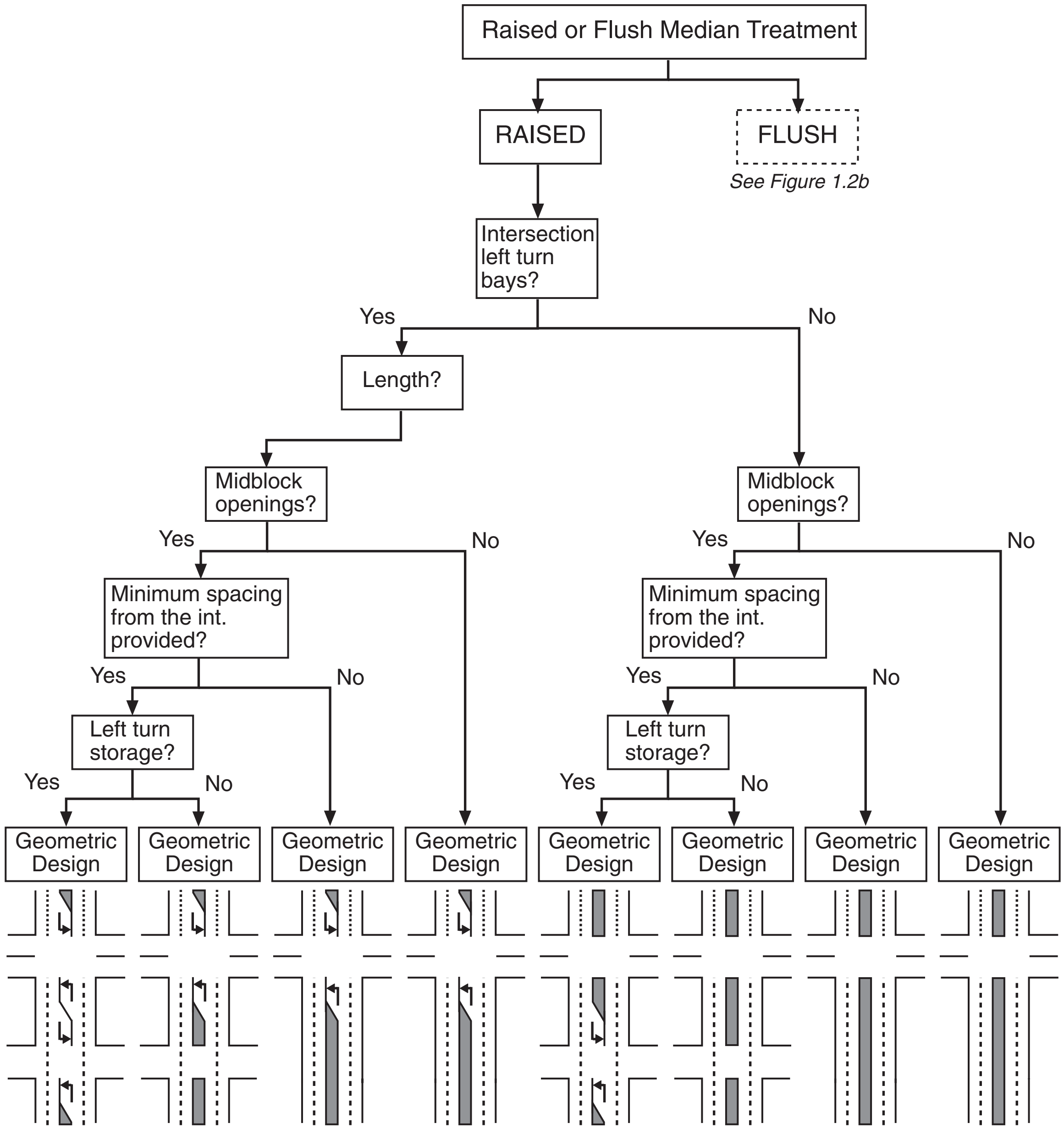


Figure 1.2a Decision Chart for Arterial Median Treatments

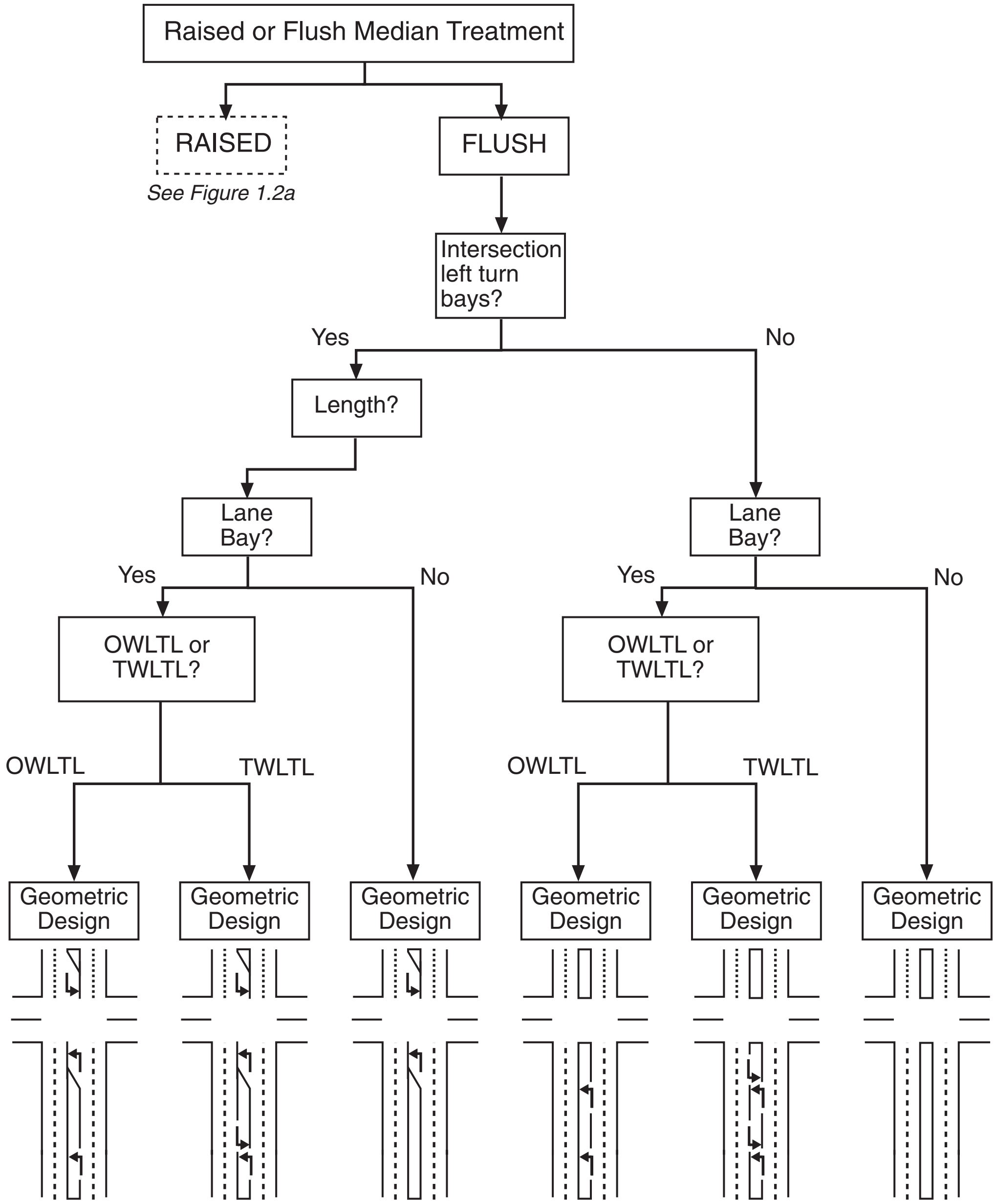


Figure 1.2b Decision Chart for Arterial Median Treatments

CHAPTER 2 SYNTHESIS OF BACKGROUND INFORMATION

Median design is an important aspect of roadway design. This is evident in the fact that it has been studied for nearly a half century. Efforts thus far have attempted to describe the effects of various median designs in terms of operations, safety, cost benefit ratios, and mathematical models. In some instances, the same conclusions are drawn and in others there is contradiction. Questions have been answered, but it is evident that median design is an intricate issue.

The research accomplished thus far can be divided into several different categories and subcategories. The main classifications of study are operations, safety, and cost-effectiveness. Within these classifications, research is conducted by field studies, comparative analysis, before-and-after cases, and computer simulation.

OPERATIONAL STUDIES

Operational studies in the field and through computer simulation have found that some median treatment, either raised or two-way left-turn lane (TWLTL), is operationally superior to no median treatment (Bonneson 1998; Ballard 1988). However, the evidence is not conclusive as to whether the reduction of a through lane to provide left-turn treatment is beneficial. Meyer (1996) found that a two-lane road with a raised median and left-turn bay was superior to a four-lane road with no median treatment along a 1-mile stretch of state highway, while Nemeth (1978) found in a field comparison that the overall benefits of a TWLTL were offset by the reduction in capacity with the elimination of a through lane.

The discrepancies in the literature may be explained by further examination of additional variables such as adjacent land use. For example, McCormick (1983) reasoned that a two-lane cross section with a TWLTL would operate better than a four-lane section with no median treatment, if the width of the section is constrained and there is commercial development.

Most of the operational studies that have been conducted include computer simulation on some level. In 1992, Venigalla used computer simulation to show that the difference in left-turn delay for TWLTLs and nontraversable medians, which include raised and divided, was insignificant. However, the TWLTL caused less delay to the through traffic.

In the comparison of raised versus TWLTL, Walton (1980) stated that TWLTLs were effective in locations with frequent driveway openings experiencing moderate left-turn demand, while the raised design was more appropriate at locations of high left-turn demand. Modur

(1990) deduced that raised medians and TWLTLs were operationally equivalent at driveway spacings greater than 400 feet.

In looking at the decision to install a TWLTL as opposed to no median treatment, Ballard (1983) determined that a directional volume of greater than 700 veh/hr justified a TWLTL.

Numerous studies have attempted to quantify threshold values for using different types of medians. Parker (1991) conducted an extensive literature review only to state that there was no evidence of maximum volume levels for particular treatment types.

Oppenlander (1990) used computer simulation to develop tabular guidelines for left-turn lanes at intersections, both signalized and unsignalized, for two- and four-lane roadways. Modur (1990) also produced a table identifying conditions that require a left-turn lane. He also indicated that for speeds that exceeded 45 mph, a raised median design was recommended.

Other measures have been used to determine effectiveness of a median design. Acceleration noise was used by Balke (1993) to evaluate the impacts of different variables on the operation of a seven-lane section. The seven-lane section consisted of six through lanes and a TWLTL. He found that adjacent land use, driveway frequency, and average daily traffic (ADT) all affected acceleration noise along the segments.

SAFETY STUDIES

Studies that pertain to the safety aspects of median design generally fall into two categories: comparative and before versus after. Comparative studies analyze accident data from different locations with similar characteristics, ideally differing only in median design. Before-versus-after studies require a longer period of time because they include data from the same location both before and after improvements. The construction projects themselves can take years to complete. Both types are subject to inaccuracies because no locations exist with duplicate characteristics aside from median design. There is a source of unexplained variation that accompanies comparative studies. Likewise, if insufficient time has passed between completion of a project and data collection, the information gathered on traffic conditions may not have returned to an adjusted equilibrium. However, the studies still provide useful information on the understanding of median design characteristics.

Squires (1989) used a comparative study of accident rates between TWLTLs and raised medians to develop accident prediction equations. Through regression, his team determined that overall raised medians had lower accident rates than TWLTLs. A study of accident data in

California and Michigan supported the above claim for four-lane sections in commercial areas (Harwood 1986). However, they found that in residential areas divided cross sections had the highest accident rates.

The most comprehensive literature summary pertaining to safety issues in median design can be found in a paper prepared for the Florida Department of Transportation (DOT) by Stover (1994). The culmination of his research found that "median access control results in a substantial reduction in the number of crashes" and associated social and economic costs. Median access control includes the installation of a nontraversable median with specifically designed median openings.

His research concluded that roadways with a projected volume exceeding 24,000 ADT should have a nontraversable median incorporated into their design, citing that studies show nontraversable medians are safer at higher ADT values. Whenever possible, the median should be 30 feet in width to allow for a 6-foot nose and potential dual left-turn bays. The 6-foot nose width will accommodate pedestrians at intersection crossings.

In addition to other criteria pertaining to median design, the report also proposed a minimum spacing of median openings of 1,320 feet, or ¼ mile, on roadways with speeds in excess of 45 mph. For slower speeds, the optimal median opening was 660 feet, or 1/8 mile. The justification for these distances included interference with future expansions of left-turn bays at signalized intersections, excessive speed differentials, and prohibition of narrow S-shaped medians that were determined to be unsafe.

A TWLTL is still safer than no median treatment. In 1984, Thakkar reported that the severity and total accident rates had been significantly reduced on sections where a TWLTL had been installed. When only accident reductions were considered as benefits, the installation of a TWLTL was cost effective for all values of interest rates, service lives, and salvage values.

Driveway density can also affect accident rates. Margiotta (1995) found that although raised medians were generally safer than TWLTLs, the TWLTL faired better in segments with high driveway densities and low-to-medium traffic volumes. He concluded that driveway densities were an important contributor to accidents in raised median sections, but not in TWLTL sections.

Right-of-way is always an important issue in cross-section design and may prohibit certain otherwise desirable designs. However, in a before-and-after study examining the effects of conversion from an undivided four-lane road to a four-lane section with a TWLTL while retaining the original width of the roadway, Harwood (1990) found that there was no change in accident severity. He found that lane widths as narrow as 10 feet could be used effectively without increasing accident rates.

COST-EFFECTIVENESS STUDIES

One of the most important factors in a feasibility study is a cost-benefit analysis. The costs of construction of a superior technology must be compared to the benefits that will be gained over alternative solutions. Often the cost-benefit analysis will be the determining factor of a design.

Bonneson (1997) developed a set of cost-benefit tables to determine if the conversion from one alternative to another was justified under specific conditions. While most conversions were feasible under certain conditions, it was not recommended to remove a raised median and replace it with a TWLTL. The benefits of the TWLTL did not outweigh the construction costs under any situation.

A methodology for comparing costs and benefits of installing a TWLTL over an undivided cross section was developed by McCoy (1988). The effort did not include raised medians. Additionally, he stated that other factors such as sight distance, high pedestrian volumes, short block lengths, and inappropriate driveway configurations, among others, should contribute to the decision process. In an accompanying study, his team concluded that the accident cost savings at ADT greater than 7,100 veh/day justified the installation of a TWLTL regardless of driveway density or left-turn percentage.

In another report determining the costs/benefits of installing any median treatment, Harwood (1978) found justification for median treatment requiring only pavement widening with ADT greater than 5,000 veh/day. If right-of-way acquisition was also involved, then only those roadways with ADT greater than 5,000 veh/day and driveway densities greater than 60 per mile, or roadways with ADT greater than 15,000 veh/day and driveway densities greater than 30 per mile, warranted median treatment. In comparing median design, he also concluded that the TWLTL option was the most desirable from a cost-benefit standpoint.

It is important to note that all cost-benefit studies will have to be adjusted for current-year construction and other prices, along with inflation rates and salvage values.

SUMMARY

Previous studies have shown that some type of median treatment, either raised or flush, provides operational and safety benefits on arterials. There are also many variables and factors in median design that affect operations and safety on arterials. An important aspect of proper median design is determining variable threshold values that combine to make one median design more or less effective than another.

Computer simulation is a tool that can be used to effectively quantify threshold values. These values, along with information obtained through accident analysis, can be combined to develop a tool for median design. This is the goal of this research effort.

CHAPTER 3 ANALYSES OF EXPERIMENTAL SCENARIOS

The analysis approach consisted of breaking the complex questions to be answered into simpler, more easily studied component parts. The following paragraphs describe that experimental procedure that deals with successively more complex questions through a series of five experiments.

EXPERIMENT A

Does distance from an intersection affect the maximum number of left turns into a driveway given maximum opposing flow?

The number of left turns that can be made into a driveway from an arterial roadway can be controlled by many variables. In this first experiment, fixing most values at carefully selected levels eliminated potential effects of many of these variables.

Geometrically, a four-lane arterial with 12-foot wide lanes, two in each direction of travel, was chosen. The cross street also has four lanes and the driveway has two lanes, one in each direction, which are also 12-feet wide. The through street is assumed to have no horizontal curves. Figure 3.1 is a schematic of the experiment A geometry.

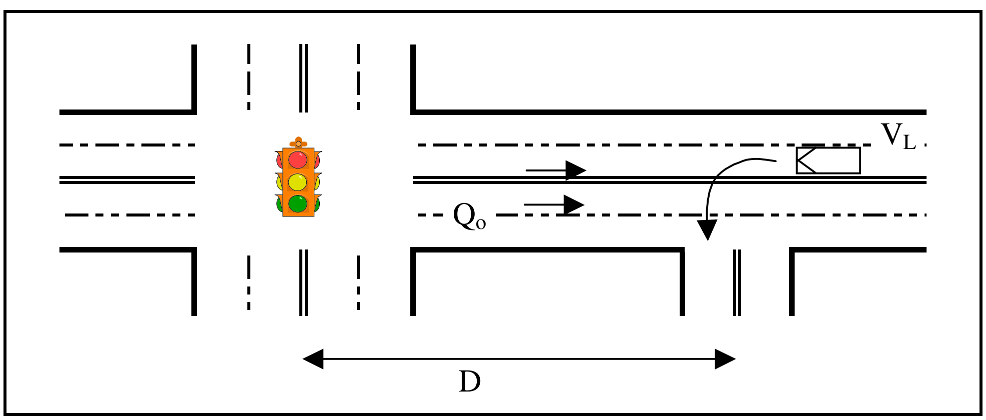


Figure 3.1 Schematic of Experiment A Geometry

The purpose of experiment A was to eliminate variable effects that could obscure a hypothesized relationship between distances from intersection to driveway upon maximum number of left turns into the driveway. Because the focus was on the maximum number of left turns that can be made into a driveway opening, only left-turning vehicles were introduced on link 100-30 (see Figure 3.2), from which left turns originated, at a rate that was guaranteed to exceed capacity. In other words, there was always a left-turning vehicle available when an appropriate gap in the opposing traffic stream presented itself.

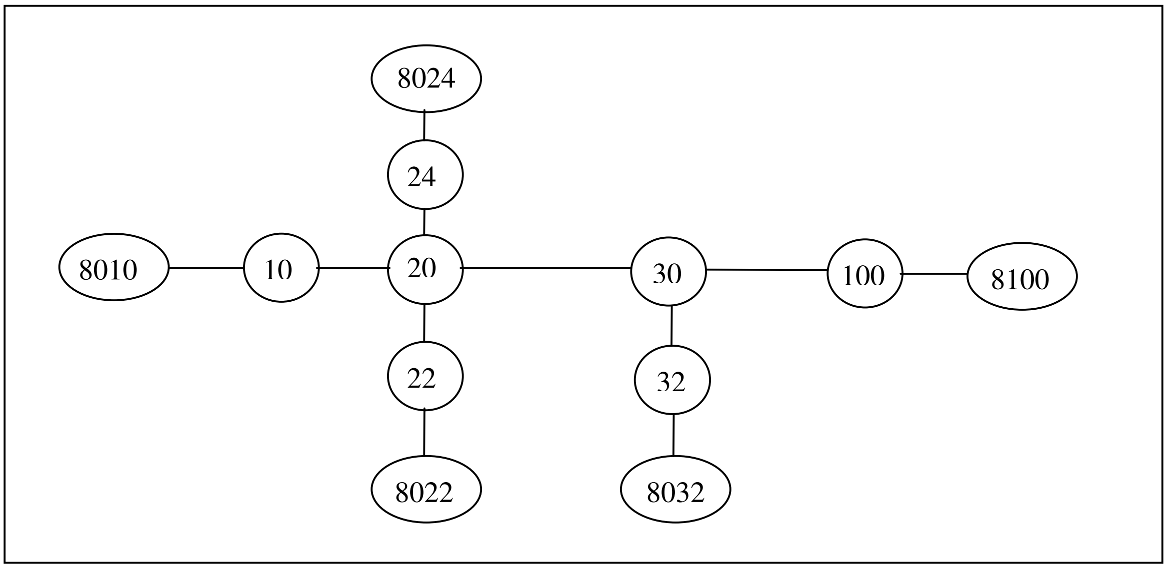


Figure 3.2 Experiment A Node Diagram

Likewise, the upstream intersection from which traffic opposing the left turns originated was loaded to capacity with vehicles. This scenario represents completely congested conditions, which in turn allow the most conservative number of left turns.

A simple network was coded into the micro-simulation software CORSIM and the results of several different simulations were recorded. The variable factors of the network included the cycle split, the cycle length, and the driveway distance from the intersection. The objective was to see if signal cycle characteristics and driveway distances had any effect on the number of left turns that could be made at a given driveway opening. Speed was assumed constant at 35 mph. The node diagram and signal timing plan used in CORSIM are shown in Figures 3.2 and 3.3.

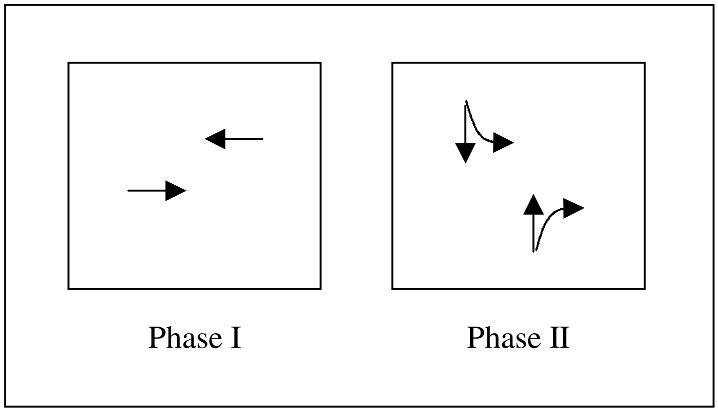


Figure 3.3 Signal Timing Diagram for Node 20

A list of variables and selected experimental values is as follows:

- Cycle length (L) = 60, 90, 120 seconds
- Cycle split (G/C) = 0.5, 0.6, 0.7
- Distance from intersection (D) = 110, 220, 330, 660, 990, 1320 feet

Results

The opposing traffic flow at the driveway remained fairly constant across the geometric and cycle variations. The mean value of opposing traffic flow for the simulations was 3,633 veh/hr. The standard deviation was 47.09 with a sample size of 54. Assuming that the population was normally distributed, all observations were within the 95% confidence levels.

Despite a consistent opposing flow, there were differences in the numbers of left turns that could be completed. It is apparent from Figure 3.4 that distance from the intersection had a significant impact on the number of left turns that could be made under nearly saturated flow conditions.

A much greater number of left turns can be completed when the driveway is close to the intersection. This is due largely to platooning effects that are created by the signal. At closer distances to the intersection, a left turner can take advantage of artificial "gaps" created by the yellow and all-red phases in a signal cycle. As the driveway is moved away from the signal, opposing vehicle arrivals more closely represent random events and the number of acceptable gaps that are available decreases. The number of left turns then begins to increase after a certain point owing to vehicle platooning caused by natural differences in drivers' desired speeds.

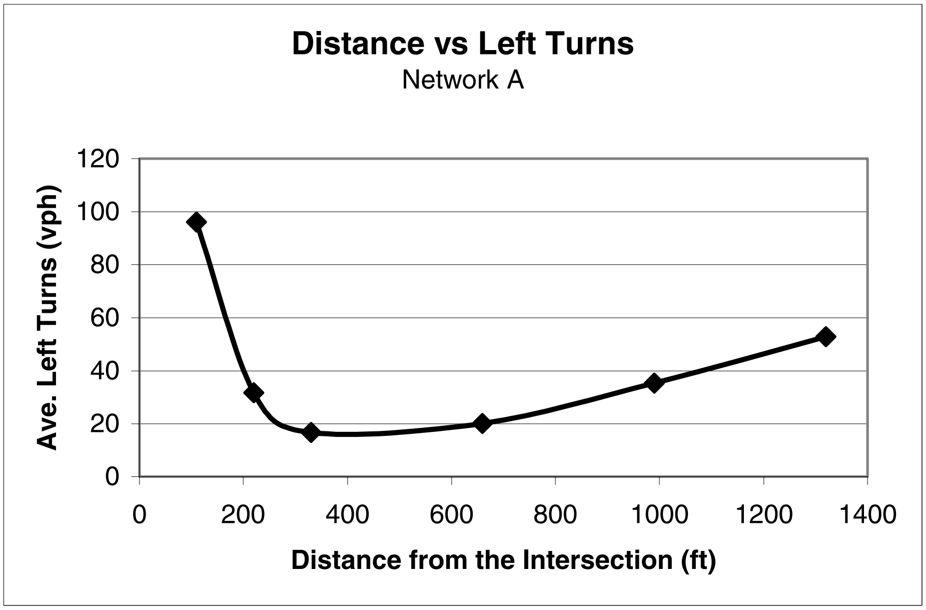


Figure 3.4 Relationship between Distance and Maximum Left Turns

A linear regression analysis was performed in order to quantitatively describe relationships between the variables and the number of possible driveway left turns. The distance from intersection to driveway was identified as a statistically significant predictor of the number of left turns; however, neither of the traffic signal cycle characteristics was statistically significant and therefore they are not used as predictors in the equations shown below. Owing to observed nonlinearity of the relationship between distance and numbers of left turns, two simple linear functions were chosen to replace a potentially more complex nonlinear relationship.

The final equations for the experiment:

$$Q_L = 127.578 - 0.361 * D \qquad \text{(D < 320') (Eq. 3.1)}$$

$$Q_L = 0.372 + 0.03748 * D \qquad \text{(D > 320') (Eq. 3.2)}$$

Where:

Q_L = maximum allowable number of left turns (vph)

D = driveway distance from the signalized intersection (feet)

Distance explains approximately 74% and 50% of the variation in the number of allowable left turns that can be made at a driveway entrance.

EXPERIMENT B

Can the combined effects of varying opposing traffic flows, as well as driveway-to-intersection distances, be captured?

The next step was to determine if additional variables would have an effect on the number of possible left turns. For example, if opposing traffic is light, more large gaps should be available allowing more left turns. In order to test this theory and quantify the hypothesized relationship, the volumes that contribute to the opposing volume stream, Q_1, Q_2, and Q_3, were reduced to three chosen, less-than-capacity conditions. The left-turn demand still exceeds capacity in order to allow a left turn to be made in every acceptable, available gap.

Additionally, it is hypothesized that opposing traffic speed will play an important role in the number of left turns that can be made into a driveway. The time taken by a vehicle to complete a left-turning movement is nearly constant regardless of opposing traffic speed. As the opposing traffic speed increases, the gap size that a driver will find acceptable will also increase. However, if the volume of the arterial remains constant while the speed increases, then the density on that link will decrease, which should allow a greater number of left turns to be completed.

A schematic of experiment B is shown in Figure 3.5. In experiment A, cycle length was not found to be a significant predictor in the number of left turns. Therefore, a signal cycle

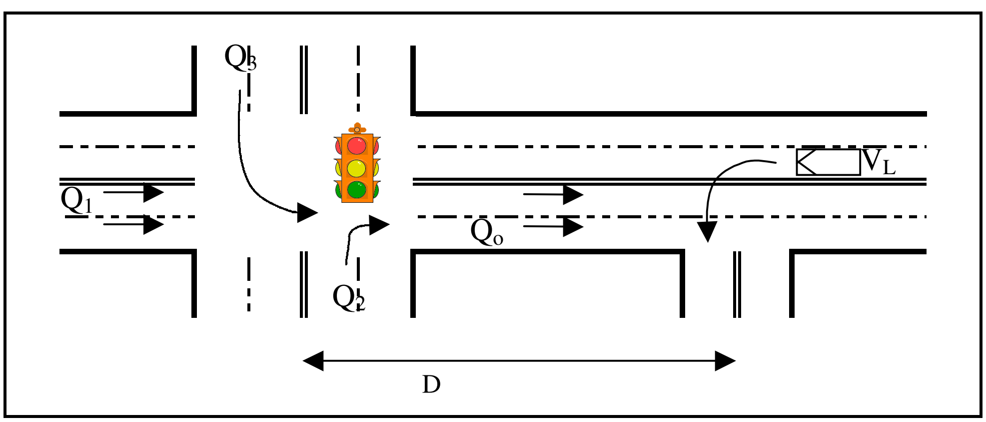


Figure 3.5 Schematic of Experiment B

length of 60 seconds was used with signal phases that were identical to experiment A.
A list of variables and experimental values is as follows:

- Q_1 = 1000, 1250, 1500 veh/hr
- Q_2 & Q_3 = 200, 500, 750 veh/hr
- Distance from intersection (D) = 110, 220, 330, 660, 990, 1320 feet
- Speed (S) = 25, 35, 45, 55 mph

Results

Both bivariate relationships and a regression analysis were examined with the new data. As expected, the location of a driveway entrance from the signalized intersection remains an important determinant in predicting the maximum number of left turns into that driveway. At distances less than 330 feet from the intersection, there is a strong negative relationship between the driveway distance and the number of left turns that can be achieved. As the distance of the driveway from the intersection increases beyond 330 feet, the relationship is still negative but the coefficient is significantly reduced. Therefore, the impact is less consequential. For simplicity, the distance variable is removed from the developed equation for distances greater than 330 feet.

As expected, the opposing traffic volume was very strongly related to the maximum number of driveway left turns. Given that the relationship with this opposing volume is negative, the maximum number of lefts that can be accomplished therefore decreases with an increase in the opposing traffic flow.

Speed was also an important factor in determining the number of left turns that can be made into a driveway entrance. The relationship here is positive indicating that an increase in speed will result in an increase in the number of left turns. This relationship is intuitive when examining the relationship between density, volume, and speed. As mentioned previously, density is the product of volume and the inverse of speed. If the volume remains constant and the speed along an arterial is increased, then the density along that same arterial will decrease. One would expect a decrease in density to accompany an increase in the allowable left turns. The following graph better illustrates this trend.

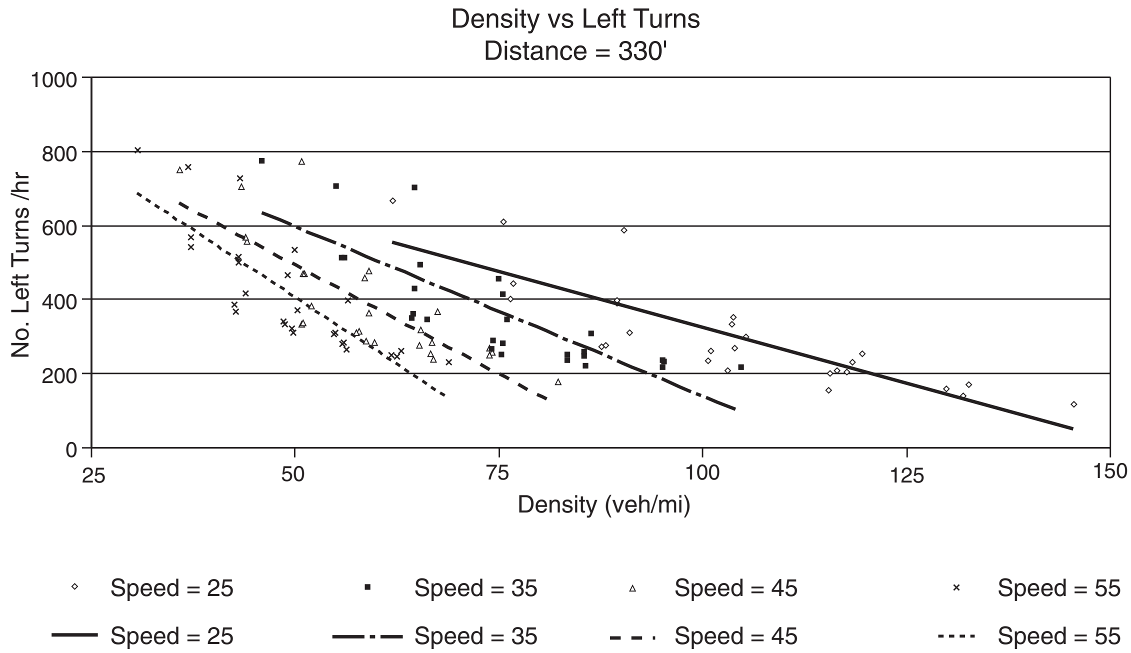


Figure 3.6 Left Turn and Density Relationship

As Figure 3.6 indicates, at a given density more left turns can be made at lower speeds. This is due to the fact that smaller, acceptable time gaps are required at lower speeds if the left-turn time is assumed constant across different opposing traffic speeds. As the speed on the arterial increases, a larger time gap is necessary. However, as evident from the regression analysis, the number of lefts will increase with an increase in speed. These two observations would seem to be contradictory until it is realized that there is interaction between the two variables.

In the regression analysis, density was also examined for its predictive abilities. While significant, it did not provide the same level of certainty as speed and volume provided as separate variables. This is due to the fact that while the coefficient for opposing volume is relatively similar for both equations (0.374 and 0.328), the coefficient for speed is vastly different (4.191 and 2.625). This indicates that the distance a driveway opening is from the intersection also has an effect on the additional left turns that are allowed by an increase in speed. Therefore, combining speed and volume into density and using it to forecast the number of lefts would result in a loss of information.

The regression analysis also indicated that there was correlation between the maximum number of left turns and the makeup of the opposing traffic stream. Two shifter variables were introduced in the model specification. One indicated that if the arterial volume going through the intersection (Q_1) was greater than the sum of the cross-street volumes turning onto the arterial, a positive relationship exists between the number of left turns and a larger Q_1. This is logical due to the signal-timing plan that was used in the experiment. Both phases were given equal amounts of green time. Therefore, if the sum of the turning movements is lighter than the through movement, then there will be essentially two different densities created in the opposing traffic stream. This would allow more left turns than would be possible if opposing volumes during both signal phases were equivalent.

The other shifter term designated that there was a larger right-turning movement from the cross street than left-turning movement. Right turns on red were allowed at the signalized intersection, and this causes right turns to be made in available gaps that are in turn not available for left turns into the driveway. As expected, this relationship is negative.

Despite the fact that both variables proved to be significant to the 95% confidence level, they were not included in the final model specification for several reasons. First, it is assumed that any signalized intersections will be properly timed to provide an optimal movement of traffic through the intersection. This should be the most important function of the signal and any effects that it causes to turning volumes downstream would be secondary. In the most conservative instance, the signalized intersection would operate under capacity conditions, and as shown in Experiment A, the ratio of the green time allotted to each phase would not have an impact on the downstream number of lefts at the driveway entrance.

Secondly, traffic volumes on arterials are dynamic and vary during different times of the day. During the a.m. peak hour one, movement may be quite heavy while another may be greater during the p.m. peak hour. It would be a cumbersome signal-timing task to determine for each driveway opening if there were a specific heavy directional movement that should be addressed.

The following are the final equations for the experiment:

$$Q_L = 1354.064 - 0.960 * D + 4.191 * S - 0.374 * Q_o \qquad (\text{D} < 320') \text{ (Eq. 3.3)}$$

$$Q_L = 948.665 + 2.625 * S - 0.328 * Q_o \qquad (\text{D} > 320') \text{ (Eq. 3.4)}$$

Where:

Q_L = maximum allowable number of left-turns (vph)

D = driveway distance from the signalized intersection (feet)

S = opposing traffic speed (mph)

Q_o = opposing volume (vph)

LAND USES

It is now appropriate to introduce relationships between varying levels of land use and driveway traffic demands into the experiment. So far, the experiments have been concerned only with the maximum number of left turns that could be made into driveways under specific conditions. In the next experiment, types of land uses associated with maximum median left-turn movements were identified.

The Institute of Transportation Engineers (ITE) Trip Generation Manual is a compilation of studies that have been conducted regarding trip generation to assorted land-use types. In order to quantify numbers of left turns, data included in the 5th edition of the ITE Trip Generation Manual were studied.

The following four types of data sets were examined:

- The average vehicle trip ends (AVTE) versus the independent variable (trip generator size descriptor such as square feet of floor space) for a peak hour of adjacent street traffic in the a.m.
- AVTE versus the independent variable for the a.m. peak hour of the generator
- AVTE versus the independent variable for a peak hour of adjacent street traffic in the p.m.
- AVTE versus the independent variable for the p.m. peak hour of the generator

An overall weighted average for the independent variable was calculated by summing the number of studies multiplied with the quantity of its independent variable for each data set and dividing by the total number of studies for all the data sets. Independent variables used in the manual vary with land-use types. Although many use square feet of gross floor area, a variety of other measures are used as well.

The average trip generation rate and entering directional distribution were extracted from the data sets and multiplied to obtain a trip generation rate for an individual data set. An average number of entering trip ends for each data set was calculated by multiplying this rate by the previously found average-size independent variable. These trip ends were compared in the a.m. cases and the p.m. cases.

An “optimal” number of trip ends for each land use was chosen from the four categories. This amount was then halved under the assumption that of the entering volume, 50% would enter the driveway by making a right-hand turn, and 50% would enter by making a left-hand turn. The experiment was only concerned with vehicles entering the driveway by making a left-hand turn.

Table 3.1 summarizes land uses categorized by similar amounts of generated left turns as selected for use in Experiment C. A full list of the land uses that were examined from the ITE Trip Generation Manual is shown in Table A1 in Appendix A.

Table 3.1 Example Land Uses and Corresponding Generated Left Turns

Land Use	**Inbound Lefts (vph)**
Restaurants, Convenience Market (24 hours), Medical Building, Drive-In Bank	50
Supermarket, Post Office, Small Shopping Center	100
Discount Store, High School, Research & Development Center	200
Medium Shopping Center, Office Park	400
Large Shopping Center, Commercial Airport	900

EXPERIMENT C

Can effects of waiting driveway left turners upon same direction through traffic be quantified?

As derived from the previous experiment, a midblock left turner will be affected by the opposing traffic flow, the link speed, and the distance from the signalized intersection. However, a left turner will also have a reciprocating effect on the advancing traffic stream as well. In Experiment C, the through-vehicle movement was introduced into the specification.

A schematic of Experiment C is shown in Figure 3.7. As in the previous experiments, CORSIM was used to run a series of simulations while having selected variable value combinations. Variables included the opposing volume, Q_o, made up of Q_1, Q_2, and Q_3; the distance of the driveway opening from the signalized intersection, D; and the link speed, S. The number of left turns into the driveway was adjusted according to the different land uses that were discussed in the previous section. The advancing volume, Q_A, is made up of the through volume, Q_T, and the left-turning vehicles, V_L, and is equal to the opposing volume, Q_o.

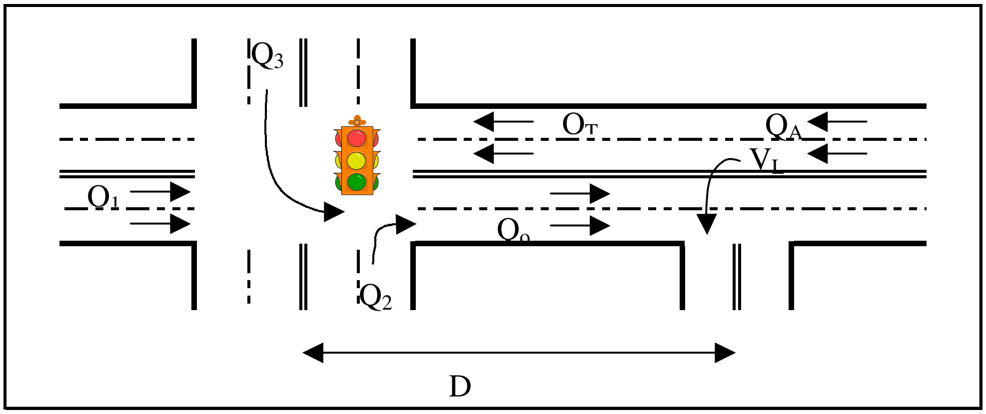


Figure 3.7 Schematic of Experiment C

Where:

$$Q_o = Q_1 + Q_2 + Q_3 \qquad \text{(Eq. 3.5)}$$

$$Q_A = Q_t + V_L \qquad \text{(Eq. 3.6)}$$

$$Q_1 = \frac{1}{2}Q_2 = \frac{1}{2}Q_3 \qquad \text{(Eq. 3.7)}$$

$$Q_A = Q_o \qquad \text{(Eq. 3.8)}$$

With the introduction of through vehicles in the experiment, a large queue may develop at the signal, thereby blocking the driveway opening and preventing left turners from using an available gap. Even when the driveway opening was close to the signalized intersection there were relatively few occurrences.

In experiment A, the length of the signal cycle and the phase split had no effect on the number of midblock left turns that could be made. However, a longer cycle length would allow for more continuous flow for the through vehicles at the heavier traffic conditions. Therefore, the length of the cycle was increased to 150 seconds, maintaining the same two-phase timing plan with four seconds of yellow and one second of all red for each phase. Additionally, the green time was divided proportionally between the through movements and the turning movements based on unadjusted volumes. Table A2 lists the respective timing plans and is located in Appendix A.

A list of variables and experimental values is as follows:

- Opposing Volume (Q_o) = 1000, 1500, 2000, 2500, 3000 veh/hr
- Left-Turn Demand (V_L) = 0, 50, 100, 200, 400, 900 veh/hr
- Distance from intersection (D) = 110, 220, 330, 660, 990, 1320 feet
- Speed (S) = 25, 35, 45, 55 mph

Results

Overall, as the left-turn demand increases for a given advancing volume, speed, and driveway distance from a signalized intersection, the operational characteristics of the link decline. Delay for both the left-turning movement and the through traffic increases as vehicles waiting for acceptable gaps form a queue at the driveway opening. The link speed decreases as well as the link capacity. These changes result in the link density from which left turns are made approaching fully congested conditions.

The objective of this experiment was to quantify the conditions under which the no median treatment (no channelization) cross section fails. From experiment B, maximum allowable numbers of left turns that can be made at specific speeds, volumes, and driveway distances were determined. However, the introduction of through vehicles in the advancing vehicle traffic stream further reduces the number of left-turning opportunities due to interaction among vehicles. Additionally, the delay to both left-turning vehicles and through traffic and the reduction of speed along the link are also important issues in determining the likelihood of cross-sectional failure.

In looking first at the number of possible left turns, a regression analysis was conducted in order to predict the maximum number of left turns into a driveway when through traffic was also present in the advancing traffic stream. Only data from scenarios where the left-turn demand exceeded the left-turn capacity were used in the analysis. The graph in Figure 3.8 illustrates the fact that for any opposing volume there is a maximum number of left turns or threshold. The slope of the trend line is similar to what was found in the regression analysis in experiment B. Therefore, points along this threshold were extracted and used in the regression analysis.

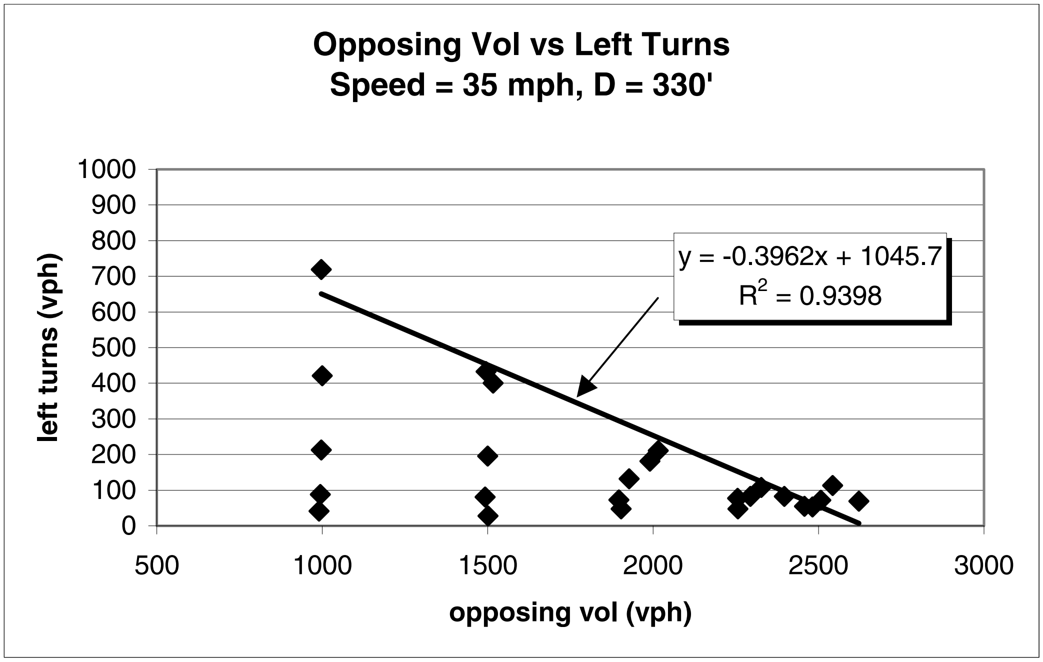


Figure 3.8 Relationship Between Opposing Volume (vph) and Left Turns (vph)

The final relationships that were found during the regression analysis are described in equations 3.9 and 3.10. They were developed using 240 observations found in the data sets. As expected, the coefficients of the independent variables are similar in magnitude and direction to those found in experiment B. Where driveway distances are greater than 330 feet, speed is no longer a significant predictor of the number of left turns. This can be rationalized by the fact that at closer distances to the driveway entrance there is more variation in the speed of individual vehicles. When the signal changes from red to green, some motorists will accelerate quickly, while others will increase their speed at a more conservative rate. At distances farther from the intersection, vehicle speeds have less divergence.

As expected, the equations found in this experiment predict lower volumes of left turns under the same conditions as in experiment C.

The following are the final equations for the experiment:

$$Q_L = 1190.454 - 1.270 * D + 6.072 * S - 0.369 * Q_o \qquad (\text{D} < 320') \text{ (Eq. 3.9)}$$

$$Q_L = 916.611 - 0.334 * Q_o \qquad (\text{D} > 320') \text{ (Eq. 3.10)}$$

Where:

Q_L = maximum allowable number of left turns (vph)

D = driveway distance from the signalized intersection (feet)

S = opposing traffic speed (mph)

Q_o = opposing volume (vph)

The left-turn volume across an opposing traffic stream also has an impact on the left-turn and through-traffic delay. As the number of opposing vehicles increases, there is a greater probability that a vehicle that wishes to turn left at a driveway will incur delay. Similarly, as the service rate of the driveway approaches the demand rate of the left-turning vehicles, an increase in delay to the left turners will occur.

For delay associated with the left turners, the rate varies according to the opposing volume. Delay to the left-turning vehicle increases exponentially, but at a reduced rate for smaller opposing volumes. This point is further illustrated in Figure 3.9.

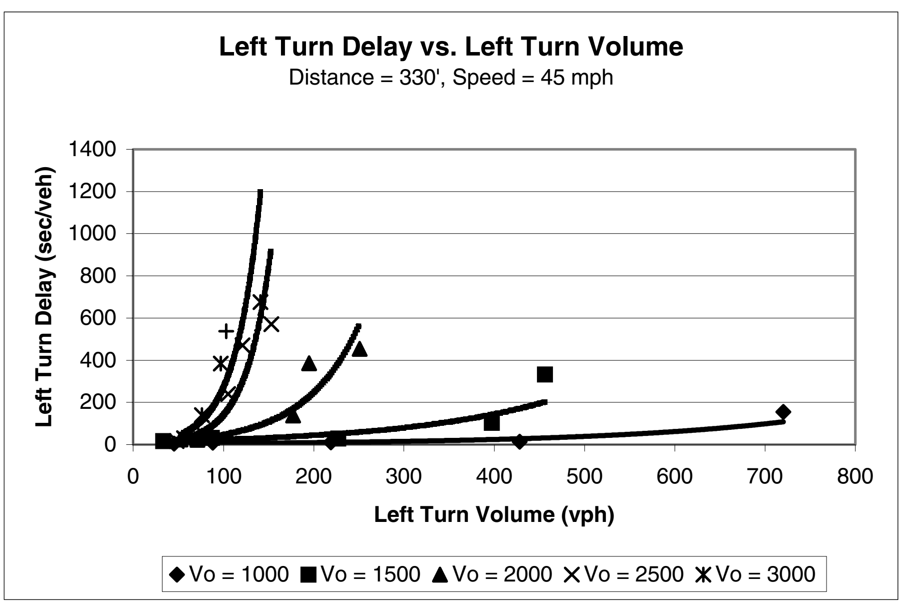


Figure 3.9 Effects of Left-Turn Volume on Left-Turn Delay

Lin (1984) found similar results when describing the relationship of left-turn delay and left-turn volume at a signalized intersection. In developing guidelines for protected left-turn signal warrants, he used an average left-turn delay of 35 sec/veh to warrant a separate left-turn phase. While there are differences between midblock left turns and left turns at an intersection, when no left-turn bay exists, an increase in left-turn delay will similarly cause an increase to through-traffic delay and a reduction in operational capacity along the roadway.

Speed is negatively impacted by the increase in left-turn demand. As illustrated in Figure 3.10, speed on the approach link declines as the left-turn demand increases. When the opposing volume is greater, the rate of speed reduction is greater for smaller left-turn demands. Under conditions where the left-turn demand exceeds the left-turn capacity, the simulations converge on a particular "minimum" link speed.

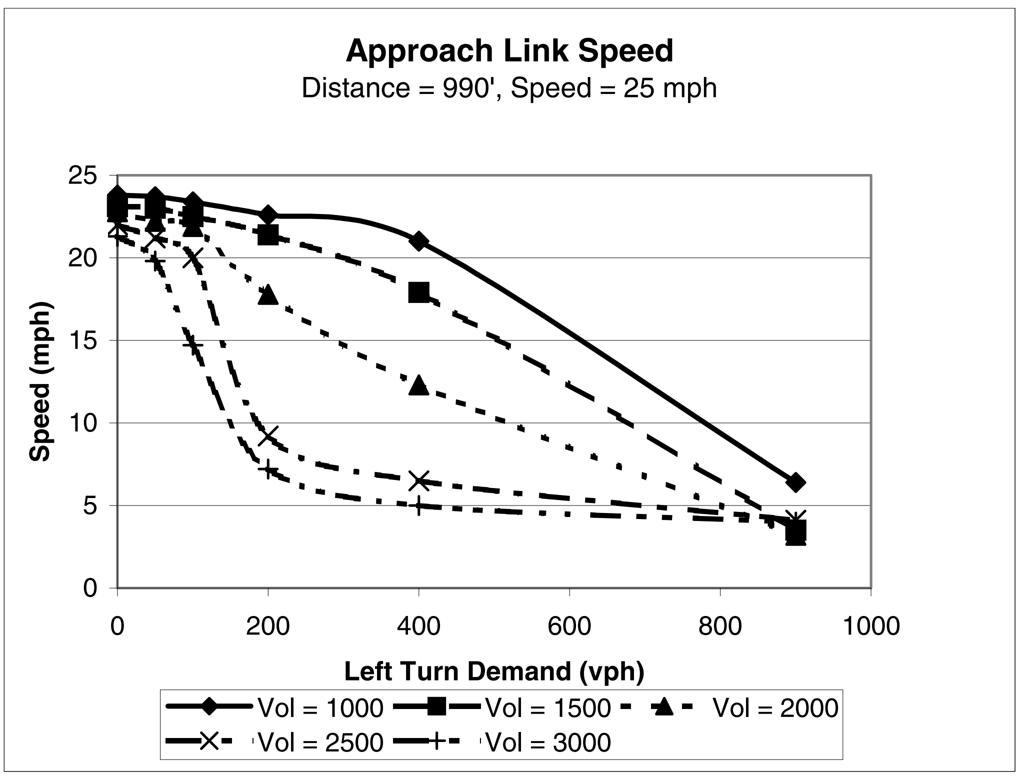


Figure 3.10 The Effects of Left-Turn Demand on Approach Link Speed

Based on the possible left-turn volume, delay experienced by left-turning vehicles, a reduction of speed along the link, and the maximum length of queue, tables were developed to identify the situations where a driveway with no median treatment is acceptable and where it breaks down. (See Tables 3.2 [A through D].)

Table 3.2-A Left-Turn Delay

Speed	25 mph	Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 3.2-B Left-Turn Delay

Speed 35 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 3.2-C Left-Turn Delay

Speed 45 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	330						
	660						
	990						
	1320						

Table 3.2-D Left-Turn Delay

Speed 55 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

The series of charts above can also be described mathematically through a linear regression equation. This is useful when conditions lie between shaded and unshaded boxes and require interpolation.

The most significant predictor of left-turn delay is the utility ratio of the driveway opening. The utility ratio is a measure of effectiveness of the driveway. If the driveway entrance were considered to be a server in a queuing theory problem, then the capacity of the driveway would be the service rate, μ, which is equivalent to Q_L. The left-turn demand at the driveway would be the arrival rate, λ. The utility ratio is computed as the arrival rate divided by the service rate. Therefore, it is important to first calculate the left-turn capacity of the driveway. This can be accomplished by referring to equations 3.9 and 3.10.

The utility ratio can be computed by dividing the arrival rate (left-turn demand) by the service rate (left-turn capacity) as shown in the following equation.

$$UR = \lambda/\mu \qquad \text{(Eq. 3.11)}$$

Where:

UR = utility ratio (λ/μ)

λ = left-turn demand (vph)

$\mu = Q_L$ = left-turn service rate (vph)

If the arrival rate is greater than the service rate, then a steady-state condition will not be achievable, an infinite queue will develop, and the system will fail. Additionally, because of randomness in the arrival and service rates, a sizable queue will also develop as UR approaches 1.0. Therefore, in order for the system to achieve a steady-state condition, a UR of less than 1 must be obtained. If UR is equivalent to or exceeds 1, then left-turn treatment is warranted.

It was also found that the effect of the utilization ratio on the left-turn delay was dependent on the opposing volume. As can be seen in Figure 3.11, left-turning vehicles will experience a higher delay when the opposing volume is higher for the same utilization ratio regardless of speed.

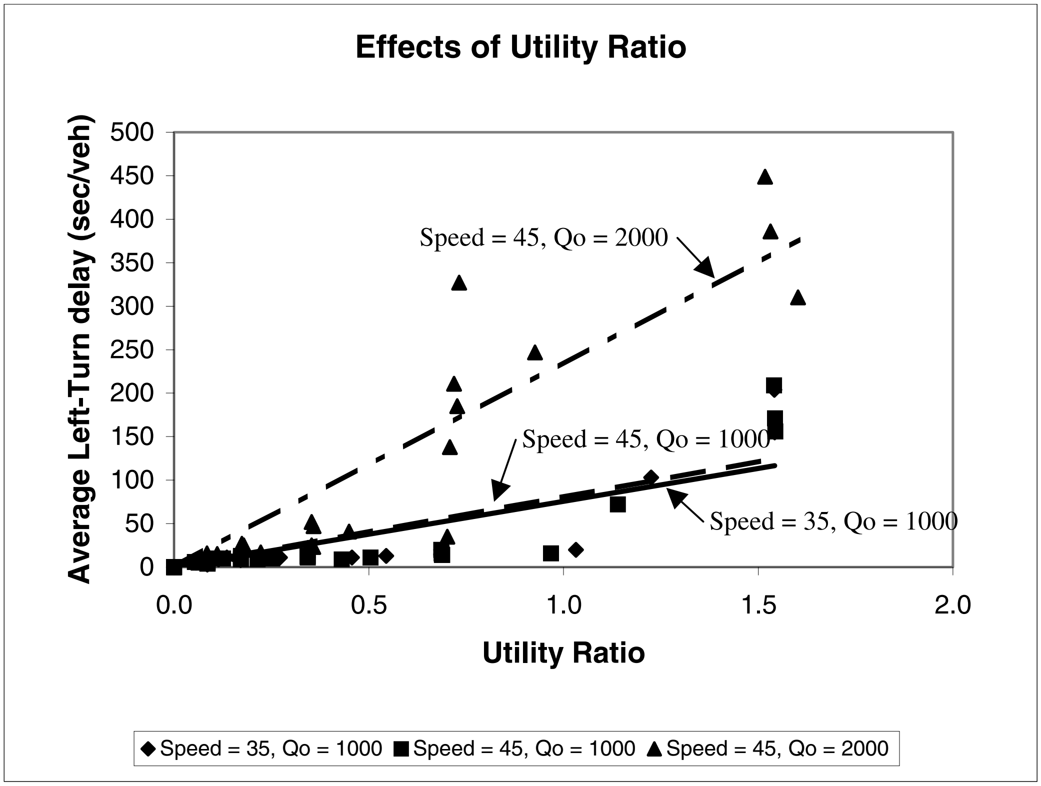


Figure 3.11 Effects of Utility Ratio on Average Left-Turn Delay

From this information, equations 3.12 and 3.13 were derived using linear regression techniques to predict the average left-turn delay that will be experienced by a vehicle under various conditions.

$$Delay_L = 0.0737 * UR * Q_o - 0.0411 * D - 0.4410 * S \quad \text{(D < 320') (Eq. 3.12)}$$

$$Delay_L = 0.0734 * UR * Q_o - 0.0219 * D + 0.0835 * \lambda \quad \text{(D > 320') (Eq. 3.13)}$$

Where:

$Delay_L$ = average delay to left-turning vehicles (sec/veh)

UR = utility ratio of the driveway opening (use equations 3.9 and 3.10 to calculate the service rate, μ)

Q_o = opposing volume (vph)

D = driveway distance from the signalized intersection (feet)

S = opposing traffic speed (mph)

λ = left-turn demand (vph)

The equations above explain 81.1% and 79.9% of the variation in average left-turn delay, respectively.

A left-turning vehicle will also have an effect on the flow of through traffic along a roadway. Similar to left-turn delay, through traffic will experience an increase in delay when left-turn and through-traffic volumes increase. Therefore, by applying the same criteria that were used to describe left-turn delay, failure of a link due to excessive through-traffic delay can also be determined. Through a series of shaded boxes, Table 3.3 (A through D) depicts the failure of a roadway under specific conditions resulting from excessive delay to the through traffic.

The same tables can also be described mathematically through a regression analysis. Equations 3.14 and 3.15 predict the increase in delay in seconds per vehicle that will be experienced by through vehicles as left-turn demand increases.

$$Delay_T = 0.0240 * UR * Q_o - 0.0591 * D + 0.0063 * Q_T \text{ (D < 320')} \quad \text{(Eq. 3.14)}$$

$$Delay_T = 0.0176 * UR * Q_o - 0.0082 * D + 0.0021 * Q_T \text{ (D > 320')} \quad \text{(Eq. 3.15)}$$

Where:

$Delay_T$ = average delay to through vehicles (sec/veh)

UR = utility ratio of the driveway opening (use equations 3.9 and 3.10 to calculate the service rate, •)

Q_o = opposing volume (vph)

D = driveway distance from the signalized intersection (feet)

Q_T = through demand volume (vph)

Both equations explain approximately 58% of the variation in increased through delay.

Table 3.3-A Increase in Through-Traffic Delay

Speed 25 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 3.3-B Increase in Through-Traffic Delay

Speed 35 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 3.3-C Increase in Through-Traffic Delay

Speed 45 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 3.3-D Increase in Through-Traffic Delay

Speed 55 mph		Left-Turn Demand (vph)					
Opposing Volume 2-lane ADT		0	50	100	200	400	900
	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

EXPERIMENT D

Can delay criteria be used to define failure of a median opening?

Another median type that was evaluated with similar techniques is the raised median design. In this experiment, left-turning vehicles were removed from the advancing traffic stream with the introduction of left-turn bays. However, it is important to recognize that this report does not include specifications for appropriate left-turn bay lengths. In actuality, a left-turn bay must be properly sized according to guidelines such as those developed by Lin (1984).

To better understand the relationships that affect this median design, a left-turn lane was constructed over the entire length of the approach link, link 100-30. A properly sized left-turn bay would significantly reduce, if not eliminate, the interaction between left-turning vehicles and same-direction through vehicles. Therefore, by allowing the left-turn lane to span the length of the approach link, the possibility of interference between the vehicles is reduced. A schematic of the experiment is shown in Figure 3.12.

This experiment is similar to experiment B in that there is no interaction between left-turning vehicles and through vehicles. The left-turn demand is adjusted according to the same criteria that were used with experiment C, as opposed to the infinite queues used earlier. The purpose of the experiment is to measure left-turn delay as opposed to the maximum left turns that can be completed, because that task was accomplished in experiment B.

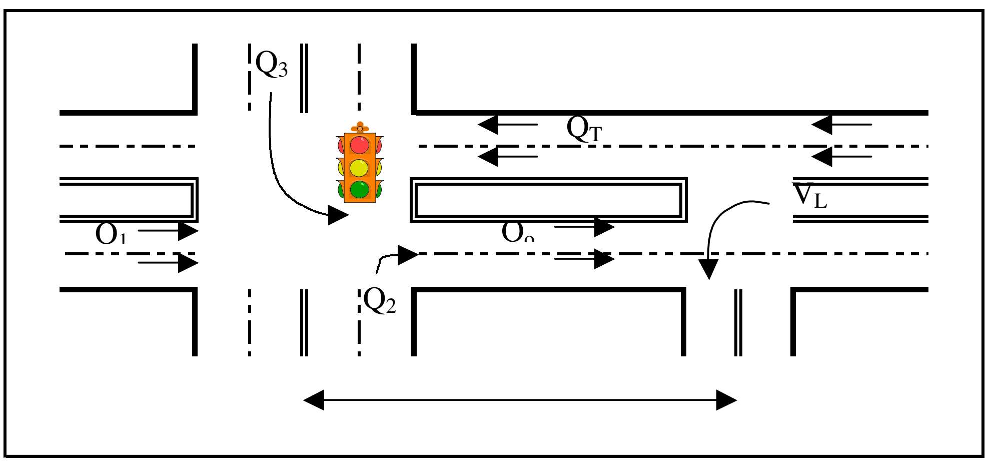


Figure 3.12 Schematic of Experiment D

In examining the results of experiment C, it appears that for low opposing volume conditions the undivided design is adequate for all speeds. Additionally, from previous studies such as Bonneson and McCoy (1998), the undivided cross section has been found to be operationally inferior to either the raised median or the TWLTL. For these reasons, the opposing volumes in this experiment were increased as indicated below.

The signal-timing plan that was used in this experiment is similar to that used in experiment C with a total cycle length of 150 sec. The cycle split for the higher opposing volume condition was computed in the same manner. A table of the signal timing is located in Appendix A.

A list of variables and experimental values is as follows:

- Opposing Volume (Q_o) = 1500, 2000, 2500, 3000, 3500 veh/hr
- Left-Turn Demand (V_L) = 0, 50, 100, 200, 400, 900 veh/hr
- Distance from intersection (D) = 110, 220, 330, 660, 990, 1320 feet
- Speed (S) = 35, 45, 55 mph

Results

As discussed earlier, the influential factor for determining failure of this typical cross section is delay incurred by the left-turning vehicles, because the left-turning vehicles are essentially removed from the advancing traffic stream. Failure of the cross section implies that a median break would be operationally inadequate at the described opening under the prescribed conditions.

Lin (1984) established a criterion for left-turn warrants at signalized intersections. He examined several left-turn warrant criteria including average left-turn delay, ninety percentile left-turn delay, average queue length, degree of saturation, and percentage of drivers incurring excessive delay.

Determining an acceptable left-turn delay threshold for midblock turns is slightly different than establishing protected left-turn signal warrants. First, the left turner is removed from the traffic stream. The delay experienced by the left turner does not affect the other users in the system. Lin also described a threshold delay of two cycles at which point a driver would become impatient and was likely to attempt a maneuver through a gap of insufficient length. Assuming that the average cycle length is 120 seconds and that it is desirable that no more than 5% of the population would experience a left-turn delay of twice that magnitude, then a reasonable threshold value for left-turn delay would be a 95th percentile left-turn delay of 240 sec/veh. If the 95th percentile value were 2.5 times the average left-turn delay, then the threshold criteria for average left-turn delay would be 96 sec/veh.

Based on the average left-turn delay criteria of 96 sec/veh, the charts in Table 3.4 (A through C) were established. A shaded box in the chart indicates that a median opening under those conditions at that location would yield a delay to the left-turning vehicles that was excessive and likely to result in an increase in accidents.

It is important to note that according to Little's formula, which is well established in the queuing theory, the average left-turn delay is inversely proportional to the left-turn volume and directly proportional to the average queue length. As the left-turn demand increases for the same average left-turn delay, the average queue length will also increase. Therefore, even though a particular combination of volumes, speed, and distance will yield an acceptable level of average left-turn delay, the average queue length may be too excessive to render it a feasible alternative.

Table 3.4-A Operationally Feasible Median Openings

Speed 35 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3500	3450	3400	3300	3100	2900
32,400	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 3.4-B Operationally Feasible Median Openings

Speed 45 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3500	3450	3400	3300	3100	2900
32,400	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 3.4-C Operationally Feasible Median Openings

Speed 55 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3500	3450	3400	3300	3100	2900
32,400	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

As with the left turns in experiment C, it is desirable to develop an equation that can be used to predict average left-turn delay with greater detail than the series of charts. A regression analysis was performed on the data obtained in experiment D to achieve the desired results.

A CORSIM program problem occurs when a vehicle wants to make a turn but is unable to enter the required lane or bay. It will stop in the nearest accessible lane and wait for an opening in the bay or turn lane. While the degree to which this behavior is realistic is in question, the action may produce inflated delay statistics. Therefore, the analysis describing the relationships between left-turn delay and other factors does not include instances where the left-turning queue reached the entrance node. When this event occurred, the next left-turning vehicle entering the system would traverse the entire link and then begin a second queue blocking a through lane.

Similar to the analysis performed in experiment C, the utility ratio, which is a measure of effectiveness of the driveway opening, was the most significant left-turn delay predictor. There is a positive relationship between the utility ratio and left-turn delay and, therefore, as the utility ratio increases there is also an increase in left-turn delay. As was discussed in the previous section, experiment C, as the utility ratio approaches 1.0 a steady-state system will be unachievable. Therefore, if the utility ratio is equivalent to or exceeds 1.0, a median opening should not be allowed at that location. See equation 3.11 to calculate the utility ratio.

It is important to note that the left-turn capacity used to calculate the utility ratio for a divided median is different than for an undivided section. Therefore, the left-turn capacity equations developed from experiment B should be used in determining driveway capacity in divided median sections.

Opposing volume was also found to be a significant predictor of left-turn delay. The relationship between opposing volume and left-turn delay is also positive, indicating that an increase in opposing volume will result in an increase in left-turn delay.

In this experiment, the additive effects of the utility ratio and the opposing volume resulted in a better model than the interactive term ($UR * Q_o$) that was found significant in experiment C. This difference may be explained by the data that were collected from CORSIM. In all experiments, the advancing volume was equivalent to the opposing volume and, therefore, separate influences of the advancing and opposing volumes on delay could not be specifically measured. In experiment C, the through traffic interacted with the left-turning vehicles and produced larger overall delays. This interaction is reflected in the term that was used in the predictive equation. When the left-turning vehicles were removed from the through-traffic stream, then the additive effects of the opposing volume on left-turn delay could be measured.

The driveway distance from the intersection also did not have any additional effect on left-turn delay that was not captured in the utility ratio. Therefore, segmenting the data by distance did not improve the overall predictability of equation 3.16, which explains 82% of the variation in left-turn delay when the vehicle was removed from through traffic.

$$Delay_L = 116.75 * UR + 0.0258 * Q_o \qquad \text{(Eq. 3.16)}$$

Where:

UR = utility ratio of the driveway opening (use equations 3.3 and 3.4 to calculate the service rate, μ)

Q_o = opposing volume (vph)

SUMMARY

Through computer simulation, we are able to describe several relationships between independent variables, the left-turn capacity, and left-turn and through-traffic delay. For example, at lower speeds a greater number of left turns can be made at a specific density. Left-turn delay increases sharply as left-turn demands increase. Inversely, approach link speed decreases as left-turn demand increases.

Linear regression analysis of the data allows us to describe in future detail the specific relationships between variables. Driveway distance from a signalized intersection is a significant determinant of the number of left turns that can be completed. However, the signal cycle length, the appropriation of green time, and the origin of the opposing traffic stream are not considerable predictors of left-turn capacity. An increase in speed of opposing traffic will result in an increase in the number of left-turn maneuvers that can be completed, whereas an increase in the opposing volume will constitute a decrease in the left-turn capacity. The utility ratio of the driveway is an important factor in the amount of delay that will be experienced by left-turning and through vehicles.

A series of tables and equations was developed with this information that determines if a left-turn lane is warranted and if a median opening will be operationally successful. The applications chapter will use these methods along with the safety criterion that was extracted from the literature to develop a procedure for determining median design.

CHAPTER 4 APPLICATIONS

The previous chapters have discussed information retrieved from the literature and developed through simulation experiments designed to establish median design relationships and criteria. The following chapter synthesizes these chapters into an application document that can be used to determine an appropriate median design. This process is applicable to four-lane, two-directional cross sections.

The application method will follow a step-by-step instructional pattern that mimics the decision process that would be executed by a designer. This process is summarized as follows:

Task 1: Determining if Left-Turn Treatment Is Required

1a: Safety Criteria

1b: Operational Criteria

1c: Calculation of Capacity and Delay

Task 2: Raised Median or Flush Median Design

2a: Safety Considerations (Raised vs Flush Median)

2b: Operational Considerations

Task 3R: Determining the Necessity of Left-Turn Bays at Intersections

Task 4R: Calculating the Length of the Intersection Left-Turn Bay

Task 5R: Assessment of Midblock Opening

5Ra: Delay to the Left-Turner

5Rb: Storage Area or Bay Length

5Rc: Distance to the Intersection or Additional Median Opening

Task 5F: Choosing One-Way or Two-Way Left-Turn Lanes

Necessary Information

Information required to complete the application process includes:

- Directional 24-hour volume (two lanes)
- Arterial speed
- Left-turn demand
- Driveway location(s) and distance(s) from the upstream intersection

This process assumes that the necessary right-of-way is available for left-turn treatment if it is required. If adequate right-of-way is not available, the designer will have to determine if left-turn treatment at the sacrifice of through-traffic lanes or parking is the optimal solution for the roadway characteristics. Such a decision is outside of the scope of this methodology.

TASK 1: DETERMINING IF LEFT-TURN TREATMENT IS REQUIRED

The first step in median design, provided that the necessary right-of-way is available, is to determine if left-turn treatment is required given the roadway and adjacent driveway characteristics. There are several ways to accomplish this task.

1a: Safety Criteria

Several studies have determined that median treatment, regardless of type, is a safer alternative to no median treatment (Stover 1994). Therefore, if a disproportionate number of accidents occur in the vicinity of the driveway location due to left-turn-related maneuvers, then left-turn treatment is warranted without regard to operational criteria.

Studies have determined that four left-turn-related accidents per year at an unsignalized intersection are justification for left-turn treatment (Oppenlander 1990). The Manual on Uniform Traffic Control Devices (MUTCD) uses five or more accidents within a 12-month period as a threshold for intersection signalization. Therefore, the four accidents per year criterion could appropriately be applied to an unsignalized intersection consisting of a driveway and a street.

> **If left-turn related accident rate is equivalent or exceeds 4/year, median treatment is warranted.** If the safety criterion is satisfied, then proceed to Task 2, otherwise continue with 1b.

1b: Operational Criteria

Three sets of acceptance grids were developed through experiment C that indicate if median treatment is required based upon operational criteria. One chart set, Table 4.1 (A through D), addresses excessive delay problems experienced by left turners. The delay threshold considered as excessive is average left-turn delays exceeding 35 seconds per vehicle (sec/veh) (Lin 1984). A second chart set, Table 4.2 (A through D), relates operational problems incurred by the through-traffic stream. These charts identify conditions causing unacceptable increases in delay to through traffic. The chart sets are located at the end of this section.

Table 4.1-A Left-Turn Delay

Speed 25 mph		Left-Turn Demand (vph)					
Opposing Volume 2-lane ADT		0	50	100	200	400	900
	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 4.1-B Left-Turn Delay

Speed 35 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 4.1-C Left-Turn Delay

Speed 45 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	330						
	660						
	990						
	1320						

Table 4.1-D Left-Turn Delay

Speed 55 mph		Left-Turn Demand (vph)					
Opposing Volume 2-lane ADT		0	50	100	200	400	900
	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 4.2-A Increase in Through-Traffic Delay

Speed 25 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 4.2-B Increase in Through-Traffic Delay

Speed 35 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 4.2-C Increase in Through-Traffic Delay

Speed 45 mph		Left-Turn Demand (vph)					
Opposing Volume 2-lane ADT		0	50	100	200	400	900
	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 4.2-D Increase in Through-Traffic Delay

Speed 55 mph		Left-Turn Demand (vph)					
Opposing Volume 2-lane ADT		0	50	100	200	400	900
	Through Volume (vph)	500	450	400	300	100	
6,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

To use the charts, the designer may choose either delay criteria. From the chosen chart set, the designer should select the chart corresponding to the roadway speed. Within the correct chart, the designer should find the grid associated with the corresponding directional 24-hour volume, driveway distance from the intersection, and left-turn demand. If the left-turn demand is unknown, Table 4.3 can be used to obtain an estimate. If the box is shaded, then left-turn treatment is required; if the box is blank, then no median treatment is required.

Table 4.3 Average Left Turns Generated by Specific Land Uses

LU Code	Land Use	Ave Generated Left-Turns (vph)
21	Commercial Airport	642
110	General Light Industrial	40
130	Industrial Park	156
140	Manufacturing	123
150	Warehousing	39
151	Mini-Warehouse	4
210	Single-Family Detached Housing	66
220	Apartment	51
230	Residential Condominium/Townhouse	36
240	Mobile Home Park	33
310	Hotel	59
320	Motel	27
520	Elementary School	57
530	High School	177
560	Church	8
565	Day Care Center	13
590	Library	30
610	Hospital	147
620	Nursing Home	23
710	General Office Building	146
720	Medical-Dental Office Building	44
732	Post Office	98
750	Office Park	266
760	Research and Development Center	162
770	Business Park	233
812	Building Materials and Lumber Store	18
814	Specialty Retail Center	89
815	Discount Store	161
817	Nursery (Garden Center)	11
820	Shopping Center (small)	106
820	Shopping Center (medium)	458
820	Shopping Center (large)	846
831	Quality Restaurant	29
832	High Turnover (Sit-Down) Restaurant	23
833	Fast Food Restaurant without Drive-Through Window	48
834	Fast Food Restaurant with Drive-Through Window	55
844	Service Station (54% am, 58% pm from passers by)	78
845	Service Station with Convenience Market	35
850	Supermarket	95
851	Convenience Market (Open 24 Hours)	29
861	Discount Club	218
890	Furniture Store	10
912	Drive-in Bank	44

If a box is shaded, median treatment is warranted. If the operational criterion is satisfied, then proceed to Task 2.

1c: Calculation of Capacity and Delay

The designer may wish, however, to obtain more detail or may be unsure of the results given by the charts if the roadway characteristics require interpolation between shaded and unshaded boxes. In this situation, the decision can be made through a series of calculations that have been developed in this research effort.

The first step is to determine the left-turn capacity of the driveway opening. The following equations, 4.1 and 4.2, which were developed through experiment C, predict the left-turn capacity of a driveway.

$$Q_L = 1190 + 6 * S - 0.4 * Q_o - 1.3 * D \quad \text{(D < 320') (Eq. 4.1)}$$

$$Q_L = 916 - 0.3 * Q_o \quad \text{(D > 320') (Eq. 4.2)}$$

Where:

Q_L = maximum allowable number of left turns (vph)

S = opposing vehicle speed (mph)

Q_o = opposing volume (vph)

D = driveway distance from the intersection (ft)

Once the capacity of the driveway has been determined, the next step is to determine the utility ratio (UR). The utility ratio is a measure of effectiveness of the driveway. If the driveway entrance were considered to be a server in a queuing theory problem, then the capacity of the driveway would be the service rate, μ, which is equivalent to Q_L. The left-turn demand at the driveway would be the arrival rate, λ. The utility ratio is computed as the arrival rate divided by the service rate.

$$UR = \lambda/\mu \quad \text{(Eq. 4.3)}$$

If the arrival rate is greater than the service rate, then a steady-state condition will be unachievable, an infinite queue will develop, and the system will fail. Additionally, because of randomness in the arrival and service rates, a sizable queue will also develop as UR approaches 1.0. Therefore, in order for the system to achieve a steady-state condition, a UR of less than 1 must be obtained.

If UR is equivalent or exceeds 1, then left-turn treatment is warranted. The designer should proceed with Task 2.

The next step is to predict the delay that will be experienced by left-turn vehicles or through traffic. This step is accomplished by two sets of equations that were developed with data extracted from experiment C. Either set of equations can be used to determine if left-turn treatment is warranted or the designer may choose to compute both delays to determine a "worst case" scenario.

Equations 4.4 and 4.5 can be used to predict the delay that will be experienced by left-turning vehicles.

$$Delay_L = 0.07 * UR * Q_o - 0.04 * D - 0.4 * S \qquad (D < 320') \text{ (Eq. 4.4)}$$

$$Delay_L = 0.07 * UR * Q_o - 0.02 * D + 0.08 * \lambda \qquad (D > 320') \text{ (Eq. 4.5)}$$

Where:

$Delay_L$ = average delay to left-turning vehicles (sec/veh)

UR = utility ratio (λ/μ)

λ = left-turn demand (vph)

$\mu = Q_L$ = left-turn service rate (vph)

The increase in delay to through vehicles can be calculated with equations 4.6 and 4.7:

$$Delay_T = 0.024 * UR * Q_o - 0.06 * D + 0.006 * Q_T \qquad (D < 320') \text{ (Eq. 4.6)}$$

$$Delay_T = 0.018 * UR * Q_o - 0.008 * D + 0.002 * Q_T \qquad (D > 320') \text{ (Eq. 4.7)}$$

Where:

$Delay_T$ = average delay to through vehicles (sec/veh)

Q_T = through demand volume (vph)

If $Delay_L$ or $Delay_T$ is equivalent or exceeds 35 sec/veh, median treatment is warranted. The designer should proceed with Task 2.

TASK 2: RAISED MEDIAN OR FLUSH MEDIAN DESIGN

There are several criteria and considerations for selecting a raised median or a flush median design. Many attempts have been made to quantify the choice of median design; however, there are a number of characteristics that are difficult to measure. Both types of designs have positive attributes and both have drawbacks (Hartman 1989).

Overwhelmingly, studies have favored raised medians over TWLTLs for safety considerations (Stover 1994, Margiotta 1995, Mukherjee 1993, Bretherton 1994, Bonneson 1998). However, all agree that some median treatment is better, both in terms of safety and operations, than the undivided cross section.

Operationally, both designs are equivalent under low driveway density, low traffic volume, and moderate speed conditions (Venigalla 1992, Bonneson 1998). The literature states that raised medians are generally preferred when through volumes and driveway densities are high. TWLTLs are preferred under lighter through-volume conditions; however, there is a discrepancy surrounding the preferred driveway spacing and left-turn volume.

Flush medians are generally a better design for:

- Access
- Strip development areas
- Construction costs

In general, raised medians are a better choice for:

- Traffic operations
- Safety
- Aesthetics
- Impact on adjacent developments
- Capacity
- Pedestrian operations

Design consistency and community involvement are also important factors in determining the median design. Raised medians are often used to regulate access along an arterial and encourage development of larger parcels of land. Businesses and residents are generally opposed to replacing TWLTLs with raised medians because of a loss of access. However, research has indicated that merchants that do not rely on "drive-by" traffic actually benefit from the raised median (Hartman 1989).

Raised medians, however, can be a maintenance hassle (Van Winkle 1988) and require a proper design so that they do not become accident hazards. They also lack the operational flexibility that flush medians offer for emergency vehicles or roadway maintenance and utility crews, and have a tendency to increase adverse travel for vehicles on the network.

Flush median designs also have drawbacks associated with them. There are an increased number of conflicting maneuvers at driveways, there is no pedestrian refuge area, and under congested conditions motorists ignore proper lane markings and intended usage. There may also be a lack of access and land-use control in roadway sections where a TWLTL is present.

The major dispute in median design choice, however, propagates down to the functional classification of the roadway. The discrepancy over TWLTLs versus raised medians is highlighted when defining the use of the roadway. TWLTLs provide access to adjacent property while raised medians are better equipped to offer a high level of service to through traffic. Conflicts arise when there are both a high volume of through vehicles and a high left-turn demand at multiple driveways.

Regardless of driveway densities, raised medians have been found to be operationally superior to TWLTLs under high traffic-volume conditions, as stated previously. Given a driveway opening in a specific location, one can assume that the TWLTL and the raised median with a left-turn bay will operate in similar manners due to the fact that in both instances the left-turner is removed from the through traffic stream and must maneuver across the same opposing volume. The driver in the TWLTL, however, encounters additional conflicts. He or she may meet a vehicle in the TWLTL moving in the opposite direction (head-on conflict). He or she may also enter the TWLTL behind a vehicle that is proceeding in the same direction but wishes to execute a left turn in a prior driveway. This action results in additional delay to the second vehicle in the TWLTL and may cause frustration leading to dangerous maneuvers.

2a: Safety Considerations (Raised vs Flush Median)

Flush median designs, continuous one- or two-way left-turn lanes (OWLTL, TWLTL), are not recommended where through-traffic speeds exceed 45 mph. A study of accident experience on continuous turn lanes found only marginally higher accident rates compared to raised median sections (Walton 1980). However, that study recommended limited continuous left-turn lane use under high-speed conditions due to the potentially catastrophic results of high-speed accidents.

If through-traffic speeds are greater than 45 mph, choose "raised median" design.

As previously mentioned, research efforts have also shown that raised medians are safer at higher traffic conditions than TWLTLs. One criterion that has been used as a threshold value for choosing median designs is a 24-hour design volume of 24,000 vehicles (Stover 1994). Therefore:

If the 24-hour design volume is equivalent or exceeds 24,000 vehicles, choose "raised median" design.

2b: Operational Considerations

Flush median designs are generally not recommended along facilities that have significant traffic congestion. Since potential flow along arterials is limited by intersection capacity, congestion usually propagates upstream and downstream from intersections. One criterion for congestion identification is queues of more than ten vehicles in all intersection approach lanes or queues that cannot be dissipated during the queue signal phase. Therefore:

If intersection queues are greater than ten vehicles or queues are not dissipated during the signal green time, choose raised median design.

If the median design is being developed for a new facility, or for any reason queues cannot be counted, congestion potential can be estimated using the ratio of demand to capacity. The Highway Capacity Manual is recommended as an easier way to estimate intersection capacity. If expected demand approaches calculated capacity, significant queues can be expected and conditions would likely exceed the threshold for significant congestion. Significant experience indicates, however, that a demand-to-capacity ratio exceeding 0.9 for a planned facility should be adequate justification for choosing a raised median design. Therefore:

If intersection demand-to-capacity ratio exceeds 0.9, choose raised median design.

For the flush median design, proceed with tasks followed by an F and for raised median designs follow tasks marked with an R.

TASK 3R: DETERMINING THE NECESSITY OF LEFT-TURN BAYS AT INTERSECTIONS

The flow of traffic on the network should take precedence over midblock turning movements. Therefore, once the general type of median design has been determined it is important to establish the necessity of a left-turn bay at the intersection because it will affect the design of upstream median openings.

This task can be accomplished by a number of means. Criteria for determining the requirement of left-turn bays have been outlined in numerous documents such as the Highway Capacity Manual, Research Report 258-1 (published by the Center for Transportation Research at The University of Texas at Austin), and many state agency design manuals. Below is the outline of the procedure that was developed from Research Report 258-1 (Lin 1984).

No Left-Turn Vehicles in Opposing Flow

When there are no left-turn vehicles in the opposing flow, the warrants for a left-turn bay at a signalized intersection can be computed by the following equation:

$$Q_W = f_c Q_C (G/C) - e_o Q_o \qquad \text{(Eq. 4.8)}$$

Where:

Q_W = warranted left-turn volume, vph

Q_C = capacity of conflict area given 1 hour of green time

G = green phase duration, seconds

C = cycle length, seconds

e_o = equivalency factor (note $e_o = 1/e_L$)

f_C = allowable utilization factor

Values of $\tilde{e}_L, \tilde{e}_o, \tilde{Q}_C, and\ \tilde{f}_C$ for the two-lane opposing flow intersection are summarized in Table 4.4.

Table 4.4 Values for Two-Lane Opposing Flows (Lin 1984)

Opposing Volume Q_o (vph)	Through Volume in Median Lane (vph)	$\tilde{e}_L$	$\tilde{e}_o$	$\tilde{Q}_C$	$\tilde{f}_C$
$0 < Q_oC/G < 1000$	100	2.0	0.507	910	0.86 - 0.92
	200	2.1	0.483	840	0.86 - 0.92
	300	2.3	0.443	740	0.86 - 0.92
	400	2.6	0.380	615	0.86 - 0.92
$0 < Q_oC/G < 800$	500	3.3	0.305	455	0.86 - 0.92
$1000 < Q_oC/G < 1600$	100	2.7	0.370	770	0.82 - 0.87
	200	2.9	0.340	695	0.82 - 0.87
	300	3.4	0.290	590	0.82 - 0.87
	400	4.4	0.230	465	0.82 - 0.87
$800 < Q_oC/G < 1600$	500	5.3	0.188	365	0.82 - 0.87
$1600 < Q_oC/G < 2000$	100	6.3	0.160	435	0.79 - 0.84
	200	7.1	0.140	375	0.79 - 0.84
	300	8.7	0.115	310	0.79 - 0.84
	400	11.1	0.090	240	0.79 - 0.84
	500	16.7	0.060	160	0.79 - 0.84

Left-Turn Vehicles in Opposing Flow

If there are left-turning vehicles in the opposing flow, the warrant for left-turns must be adjusted by the following equations:

$$\hat{Q}_W = Q_W - aQ_o \qquad \text{(Eq. 4.9)}$$

$$a = 0.317(p_c - 1/N) \qquad \text{(Eq. 4.10)}$$

Where:

a = a correction factor

p_c = percentage of the total opposing traffic (excluding left turns) on the lane with the heaviest opposing volume

N = number of opposing lanes

If left-turn demand is greater than the warranted left-turn volume Q_W, a left-turn bay is required at the intersection. The designer should proceed to the next task. Otherwise skip to task 5R.

TASK 4R: CALCULATING THE LENGTH OF THE INTERSECTION LEFT-TURN BAY

If a left-turn bay is necessary at an adjacent intersection, then it is important to size the bay before proceeding with median design as this will directly impact driveway openings and placement along the roadway.

Once again, this procedure has been well documented in other research efforts. Below is the procedure that was developed in Research Report 258-1 from the Center for Transportation Research at The University of Texas at Austin.

The maximum left-turn queue that will develop under various conditions at a four-lane cross section with a G/C ratio of 0.5 can be determined from the graph in Figure 4-1. If the G/C ratio for the intersection is other than 0.5, then an adjusted opposing volume, Q'_o, can be calculated using equation 4.11.

$$Q_o^{'} = \frac{Q_o}{2(G/C)} \qquad \text{(Eq. 4.11)}$$

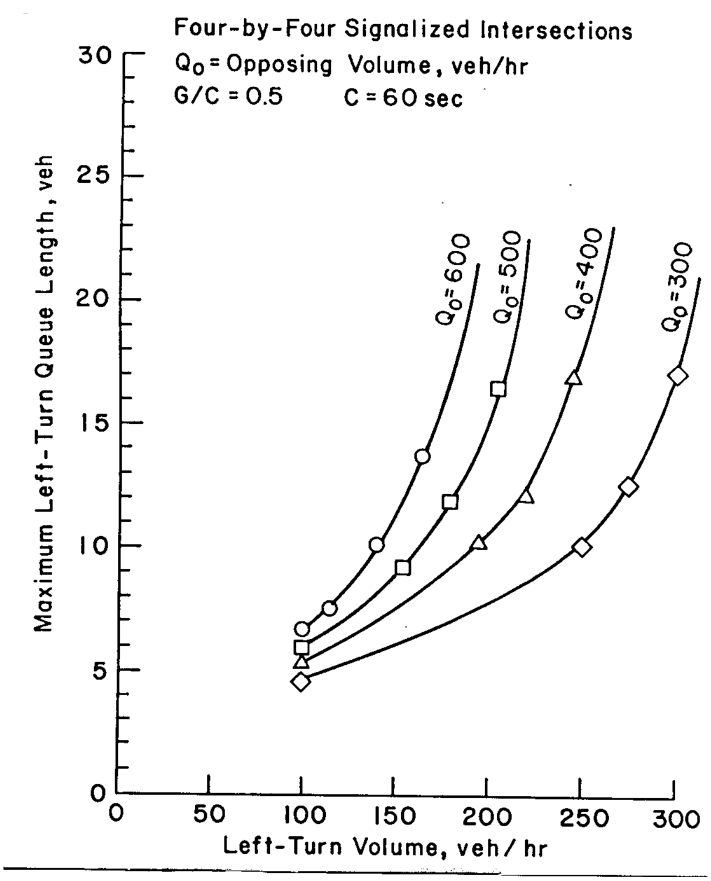


Figure 4.1 Maximum Left-Turn Queues under Various Traffic Conditions (Lin 1984)

TASK 5R: ASSESSMENT OF MIDBLOCK OPENING

In determining the location of a midblock opening, the designer must first ensure that the proposed opening will not infringe on the left-turn bay that has been established for the intersection. If there is no left-turn bay, this is not an issue. The placement of a median opening is infeasible if the proposed median location encroaches upon the intersection left-turn bay. Provided that the median opening is viable, the operational characteristics of the driveway can be examined.

There are three criteria to consider when assessing the feasibility of a median opening: delay incurred by the left-turning vehicle, storage area, and distance between the intersection and other median openings. These are discussed in the following section.

Task 5Ra: Delay to the Left-Turner

Theoretically, if a left-turner waits for a traffic-stream gap in a bay or storage lane, then operationally there is no reduction in level of service to the network through traffic if the vehicle waits indefinitely to complete his maneuver. Realistically, however, the driver will become impatient after a period of time and risk an accident by choosing a gap of insufficient size.

A series of charts was developed through experiment D, Table 4.5 (A through C), based on delay incurred by the left turner. These charts describe conditions where unacceptable levels of delay are experienced. Median openings in locations that fall into the shaded boxes should not be provided.

Table 4.5-A Left-Turn Delay

Speed 35 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3500	3450	3400	3300	3100	2900
32,400	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 4.5-B Left-Turn Delay

Speed 45 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3500	3450	3400	3300	3100	2900
32,400	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

Table 4.5-C Left-Turn Delay

Speed 55 mph		Left-Turn Demand (vph)					
Opposing Volume		0	50	100	200	400	900
2-lane ADT	Through Volume (vph)	1000	950	900	800	600	100
12,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	1500	1450	1400	1300	1100	600
18,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2000	1950	1900	1800	1600	1100
24,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	2500	2450	2400	2300	2100	1600
27,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3000	2950	2900	2800	2600	2100
30,000	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						
	Through Volume (vph)	3500	3450	3400	3300	3100	2900
32,400	110						
	220						
	Distance (feet) 330						
	660						
	990						
	1320						

If a box is shaded, do not provide a median opening; left-turn delays will likely exceed 96 seconds.

If the designer is unsatisfied with the results of the charts because roadway conditions require interpolation between shaded and unshaded boxes, then he or she may calculate the left-turn delay with equations that were developed through this research effort.

Where left-turn vehicles have been removed from the through traffic stream, the left-turn capacity of a driveway can be computed using equations 4.12 and 4.13.

$$Q_L = 1354 - D + 4 * S - 0.4 * Q_o \qquad \text{(D < 320') (Eq. 4.12)}$$

$$Q_L = 949 + 2.6 * S - 0.3 * Q_o \qquad \text{(D > 320') (Eq. 4.13)}$$

Where:

Q_L = left-turn capacity (vph)

D = driveway distance from the signalized intersection (feet)

S = opposing traffic speed (mph)

Q_o = opposing volume (vph)

As discussed in Step 1c, once the driveway capacity has been computed, the left-turn delay can be predicted by determining the utility ratio, which is the left-turn demand (•) divided by the left-turn capacity (•), and inserting it into equation 4.14.

$$Delay_L = 117 * UR + 0.026 * Q_o \qquad \text{(for all D) (Eq. 4.14)}$$

Where:

$Delay_L$ = average delay to left-turning vehicles (sec/veh)

UR = utility ratio (•/•)

λ = left-turn demand (vph)

$\mu = Q_L$ = left-turn capacity (vph)

If $Delay_L$ is equivalent or exceeds 96 sec/veh, do not provide a median opening.

Task 5Rb: Storage Area or Bay Length

Adequate procedures for determining the length of storage for the medians are similar to those used in determining the left-turn bay length at the intersection. It is important for the entry speed of the left-turning vehicle into the left-turn pocket to be no more than 10 mph different than that of the through-volume traffic. The pocket length should be sized according to the entrance speed and the ability of a vehicle to come to a stop before reaching the end of the queue. As the speed differential between the left-turning vehicle and the through-vehicle increases, the potential for accidents also increases (Florida 1997).

If the left-turn demand is unknown, Table 4.3, which is located at the end of this section, can be used to determine an estimate. See Task 4R for instructions on proper left-turn bay sizing.

Task 5Rc: Distance to the Intersection or Additional Median Opening

No median opening should be allowed to interfere with the functional area of another median opening or intersection left-turn bay. The functional area is defined as the distance required for channelization markings, queuing, and storage of vehicles wishing to complete a left-turn maneuver. Additionally, median openings should be prohibited in locations where a queue from an adjacent intersection would habitually form across the opening (Florida 1997).

Once the left-turn bays at the intersections along the roadway have been sized, then all median openings must lie within the distance defined by the functional areas of those left-turn bays. If uniform land uses between intersections exist, then the designer can use the average generated left-turn demands given in Table 4.3 to calculate a left-turn bay of sufficient length at any median opening along the roadway section. The median openings can then be strategically placed between the intersections.

If the land use along the roadway is irregular, then the designer may wish to identify critical median openings between intersections. Once the functional areas of these median openings have been described, then intermittent median openings can be planned between the intersections and the critical median openings as distance permits.

The Florida Department of Transportation (DOT) has defined a classification system of their roadways based on function. From these access classes, they have set the following minimum median opening spacing criteria for arterials with both directional and full movements.

Table 4.6 Florida DOT Median Opening Standards

Access Class	Minimum Median Opening Spacing (Directional)	Minimum Median Opening Spacing (Full)	Minimum Signal Spacing
3	1320 ft	2640 ft	2640 ft
5	660 ft	2640 ft (over 45 mph) 1320 ft (≤ 45 mph)	2640 ft (over 45 mph) 1320 ft (≤ 45 mph)
7	330 ft	660 ft	1320 ft

The above function classes pertain to arterial roadways. A higher access class number indicates that the function of the arterial is to provide access to adjacent property rather than accommodate through-traffic movement. A lower number is representative of an arterial whose primary function is the movement of through traffic.

TASK 5F: (OWLTL OR TWLTL) Choosing One-Way Or Two-Way Left-Turn Lanes

Few studies have been done concerning the choice between OWTLT and TWLTL. A TWLTL is generally chosen in areas of strip commercial development. An OWLTL is more beneficial at major intersections with high left-turn demand or where there are driveways on only one side of the street.

SUMMARY

This chapter describes a process that can be used by the practitioner to design median treatment for a four-lane, bidirectional arterial roadway. The tasks required to complete this process are summarized below. An example application is provided as Appendix B.

Task 1: Left-Turn Treatment Is Required if:

1a: Safety Criteria

- Accident rate ≥ 4/year

1b: Operational Criteria

- Shaded box in Table 4.1 (A through D)
- Shaded box in Table 4.2 (A through D)

1c: Calculation of Capacity and Delay

1. $Q_L = 1190 + 6 * S - 0.4 * Q_o - 1.3 * D$ (D < 320’)

2. $Q_L = 916 - 0.3 * Q_o$ (D > 320’)

3. UR = λ/μ

- UR ≥ 1

D < 320’

4. $Delay_L = 0.07 * UR * Q_o - 0.04 * D - 0.4 * S$

5. $Delay_T = 0.024 * UR * Q_o - 0.06 * D + 0.006 * Q_T$

D > 320’

6. $Delay_L = 0.07 * UR * Q_o - 0.02 * D + 0.08 * \lambda$

7. $Delay_T = 0.018 * UR * Q_o - 0.008 * D + 0.002 * Q_T$

- $Delay_L$ or $Delay_T \geq 35$ sec/veh

Task 2: Raised Median or Flush Median Design

Raised Median Design

- Traffic operations
- Safety
- Aesthetics
- Impact on adjacent developments
- Capacity
- Pedestrian operations

2a: Safety Considerations

- Speed > 45 mph
- 24-hour design volume ≥ 24,000 vehicles

2b: Operational Considerations

- Intersection queues > ten vehicles or less are not dissipated during the signal green time
- Intersection demand/capacity ratio > 0.9

Flush Median Design

- Access
- Strip development areas
- Construction costs

Task 3R: Left-Turn Bays at Intersections Required if:

No Left-Turn Vehicles in Opposing Flow

1. $Q_W = f_c Q_C (G/C) - e_o Q_o$

   see Table 4.4 for values of $\tilde{e}_L, \tilde{e}_o, \tilde{Q}_C, and\ \tilde{f}_C$

Left-Turn Vehicles in Opposing Flow

2. $\hat{Q}_W = Q_W - aQ_o$
3. $a = 0.317(p_c - 1/N)$

- Left-Turn Demand $> Q_W$ or $\hat{Q}_W$

Task 4R: Length of the Intersection Left-Turn Bay

Adjust opposing volume if G/C ratio • 0.5

1. $Q_o^{'} = \dfrac{Q_o}{2(G/C)}$

- See Figure 4.1 for maximum left-turn queue

Task 5R: Assessment of Midblock Opening

5Ra: Do not provide median opening if:

- Shaded box in Table 4.5 (A through C)

1. $Q_L = 1354 - D + 4 * S - 0.4 * Q_o$ (D < 320')
2. $Q_L = 949 + 2.6 * S - 0.3 * Q_o$ (D > 320')
3. UR = λ/μ
4. $Delay_L = 117 * UR + 0.026 * Q_o$

- $Delay_L$ • 96 sec/veh

5Rb: Storage area or bay length

See Task 4R to size a left-turn bay

5Rc: Distance to the intersection or additional median opening

Place median openings based on intersection left-turn functional areas and critical median opening functional areas

Task 3F: Choosing One-Way or Two-Way Left-Turn Lanes

OWLTL

- Major intersections with high left-turn demand
- Driveways on only one side of the roadway

TWLTL

- Strip commercial development
- Driveways on both sides of the roadway

CHAPTER 5 CONCLUSIONS

Principal arterial class streets must move large traffic volumes while providing limited property access. Guidelines for median design and other characteristics that will maintain traffic flow potential are needed. Without such guidelines, principal arterials tend to lose traffic flow potential at the expense of property access functions.

This research effort has developed a series of guidelines to aid the designer in median selection. The criteria consider flush medians with no left-turn lanes, flush medians with left-turn lanes, and raised medians with limited median openings.

The thorough literature review and computer simulations research team have found that:

- Driveway distance from a signalized intersection is a significant factor in the capacity of the driveway opening.
- Cycle length and split are not important predictors of the capacity of a driveway opening.
- The origin of the vehicles in the opposing traffic stream is not a significant predictor of the capacity of an intersection.
- At lower speeds, a greater number of left turns can be made through equivalent opposing densities.
- Speed has a positive relationship with the capacity of the driveway opening.
- The volume of the opposing traffic is inversely related to the capacity of the driveway opening.
- The capacity of the driveway opening decreases when left-turning vehicles do not have an exclusive left-turn bay/lane.
- Left-turn delay increases sharply as left-turn demand increases. The congestion break point is dependent on the opposing volume.
- The utility ratio of the driveway opening is a significant predictor of left-turn and through-traffic delay.
- A larger opposing volume will produce a greater amount of left-turn delay for the same driveway-opening utility ratio.
- Some median treatment is superior both operationally and for safety purposes.

- Raised medians are beneficial for flow, safety, aesthetics, access control, pedestrian operations, speeds greater than 45 mph, when 24-hour design volume meets or exceeds 24,000 vehicles, when intersection queues are great or cannot be fully dissipated, or when the intersection demand/capacity ratio exceeds 0.9.
- Flush medians are suitable for access to adjacent properties in strip development areas from a construction cost standpoint.

By using the guidelines developed through this research effort, the designer can provide viable median selections that operate well with the functionality of the roadway, while adhering to the congruity of the community.

REFERENCES

Ballard, J.L., and P.T. McCoy. Computer Simulation Study of the Operational Effects of Two-Way Left-Turn Lanes on Urban Four-Lane Roadways. In *Transportation Research Record 1195*. Transportation Research Board, National Research Council, Washington, D.C., 1988, pp. 1-10.

Ballard, J.L., and P.T. McCoy. Operational Effects of Two-Way Left-Turn Lanes on Two-Way, Four-Lane Streets. In *Transportation Research Record 923*. Transportation Research Board, National Research Council, Washington, D.C., 1983, pp. 54-57.

Balke, Kevin N., Kay Fitzpatrick, and Torsten Lienau. *Factors Affecting Traffic Operations on Seven-Lane Cross Sections.* Research Report FHWA/TX-94/1293-2F, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 1993.

Bonneson, J.A., and P.T. McCoy. Median Treatment Selection for Existing Arterial Streets. *ITE Journal*, March 1998, pp. 26-34.

Bonneson, J.A., and P.T. McCoy. *NCHRP Report 395: Capacity and Operational Effects of Midblock Left-Turn Lanes*. Washington, D.C., USA: Transportation Research Board, National Research Council, October 1997.

Bretherton, W.M. Jr. Are Raised Medians Safer Than Two-Way Left-Turn Lanes? *ITE Journal*, December 1994, pp. 20-25.

Florida Department of Transportation.

Hartman, J.P., and D.B. Szplett. Median Design Alternative: Raised Median vs. Two-Way Left-Turn Lane. In *ITE 1989 Compendium of Technical Papers.* Institute of Transportation Engineers, Washington, D.C., 1989, pp. 59-62.

Harwood, D.W. *Effective Utilization of Street Width on Urban Arterials.* NCHRP Report 330, Transportation Research Board, National Research Council, Washington, D.C., August 1990.

Harwood, D.W. *Multilane Design Alternative for Improving Suburban Highways*, NCHRP Report 282, Transportation Research Board, National Research Council, Washington, D.C., March 1986.

Harwood, D.W. and J.C. Glennon. Selection of Median Treatments for Existing Arterial Highways. In *Transportation Research Record 681.* Transportation Research Board, National Research Council, Washington, D.C., 1978, pp. 70-77.

Highway Capacity Manual. Transportation Research Board, National Research Council, 1997.

Institute of Transportation Engineers Committee 6Y-19, *Planning Urban Arterial and Freeway Systems*, Institute of Transportation Engineers, Washington, D.C., 1988.

Institute of Transportation Engineers Trip Generation Manual.

Joshua, S.C., and A.A. Saka. Mitigation of Sight-Distance Problem for Unprotected Left-Turning Traffic at Intersections. In *Transportation Research Record 1356,* Transportation Research Board, National Research Council, Washington, D.C., 1992, pp. 73-79.

Lin, H., R.B. Machemehl, C.E. Lee, and R. Herman. *Guidelines for Use of Left-Turn Lanes and Signal Phases*. Research Report 258-1. Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, January 1984.

Manual on Uniform Traffic Control Devices. U.S. Department of Transportation, Federal Highway Administration, 1988.

Margiotta, R., and A. Chatterjee. Accidents on Suburban Highways – Tennessee's Experience. In *Journal of Transportation Engineering*, May/June 1995, pp. 255-261.

McCormick, D.P. and E.M. Wilson. Comparing Operational Effects of Continuous Two-Way Left-Turn Lanes. In *Transportation Research Record 923.* Transportation Research Board, National Research Council, Washington, D.C., 1983, pp. 86-90.

McCoy, P.T., J.L. Ballard, D.S. Eitel, and W.E. Witt. Cost-Effectiveness Methodology for Two-Way Left-Turn Lanes on Urban Four-Lane Roadways. In *Transportation Research Record 1197,* Transportation Research Board, National Research Council, Washington, D.C., 1988, pp. 19-33.

McCoy, P.T., J.L. Ballard, D.S. Eitel, and W.E. Witt. Two-Way Left-Turn Guidelines for Urban Four-Lane Roadways. In *Transportation Research Record 1195.* Transportation Research Board, National Research Council, Washington, D.C., 1988, pp. 11-19.

McCoy, P.T., J.L. Ballard, and Y.H. Wijaya. Operational Effects of Two-Way Left-Turn Lanes on Two-Way, Two-Lane Streets. In *Transportation Research Record 869*. Transportation Research Board, National Research Council, Washington, D.C., 1982, pp. 49-54.

Median Handbook. Florida Department of Transportation, January 1997.

Meyer, K.G., and Kruger, G.E. Moving Forward By Scaling Back an Arterial. *ITE International Conference*, 1996, pp. 65-69.

Modur S., R.B. Machemehl, and C.E. Lee. *Criteria for the Selection of a Left-Turn Median Design*. Research Report FHWA/TX-90+1138-1F. U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 1990.

Mukherjee, D., A. Chatterjee, and R. Margiotta. Choosing Between a Median and a TWLTL for Suburban Arterials. *ITE Journal*, July 1993, pp. 25-30.

Nemeth, Z.A. Two-Way Left-turn Lanes: State-of-the-Art Overview and Implementation Guide. In *Transportation Research Record 901.* Transportation Research Board, National Research Council, Washington, D.C., 1978, pp. 62-69.

Oppenlander, J.C., and J.C. Bianchi. Guidelines for Left-Turn Lanes. In *ITE 1990 Compendium of Technical Papers.* Institute of Transportation Engineers, Washington, D.C., 1990, pp. 191-196.

Parker, M.R. *Simplified Guidelines for Selecting an Urban Median Treatment –Urban Median Information*. Virginia Department of Transportation, Richmond, VA, 1991.

Parsonson, P.S. *After Median Construction, Dekalb County SR 10 Memorial Drive*, Georgia Institute of Technology, Atlanta, Georgia, 1991.

Squires, C.A. and P.S. Parsonson. Accident Comparison of Raised Median and Two-Way Left-Turn Lane for Median Treatments. In *Transportation Research Record 1239.*

Transportation Research Board, National Research Council, Washington, D.C., 1989, pp. 30-40.

Stover, V.G. *Medians, A Discussion Paper*, Florida Department of Transportation, December 1994.

Thakkar, J.S. Study of the Effect of Two-Way Left-Turn Lanes on Traffic Accidents. In *Transportation Research Record 960,* Transportation Research Board, National Research Council, Washington, D.C., 1984, pp. 27-33.

Van Winkle, S.N. Raised Medians vs. Flush Medians. *ITE Journal*, April 1988, pp. 25-26.

Venigalla, M., R. Margiotta, A. Chatterjee, A.K. Rathi, and D.B. Clarke. Operational Effects of Nontraversable Medians and Two-Way Left-Turn Lanes: A Comparison. In *Transportation Research Record 1356.* Transportation Research Board, National Research Council, Washington, D.C., 1992, pp. 37-46.

Walton, C.M., R.B. Machemehl. Accident and Operational Guidelines for Continuous Two-Way Left-Turn Median Lanes. In *Transportation Research Record 737.* Transportation Research Board, National Research Council, Washington, D.C., 1980, pp. 43-52.

Walton, C.M., T.W. Horne, and W.K. Fung. *Design Criteria for Median Turn Lanes*. Research Report FHWA/TX-78106-212-1F, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 1978.

APPENDIX A

Table A-1 Average Left-Turns Generated by Specific Land Uses

LU Code	Land Use	Avg Generated Left-Turns (vph)
21	Commercial Airport	642
110	General Light Industrial	40
130	Industrial Park	156
140	Manufacturing	123
150	Warehousing	39
151	Mini-Warehouse	4
210	Single-Family Detached Housing	66
220	Apartment	51
230	Residential Condominium/Townhouse	36
240	Mobile Home Park	33
310	Hotel	59
320	Motel	27
520	Elementary School	57
530	High School	177
565	Day Care Center	13
590	Library	30
610	Hospital	147
620	Nursing Home	23
710	General Office Building	146
720	Medical-Dental Office Building	44
732	Post Office	98
750	Office Park	266
760	Research and Development Center	162
770	Business Park	233
812	Building Materials and Lumber Store	18
814	Specialty Retail Center	89
815	Discount Store	161
817	Nursery (Garden Center)	11
820	Shopping Center (small)	106
820	Shopping Center (medium)	458
820	Shopping Center (large)	846
831	Quality Restaurant	29
832	High Turnover (Sit-Down) Restaurant	23
833	Fast Food Restaurant without Drive-Through Window	48
834	Fast Food Restaurant with Drive-Through Window	55
844	Service Station (54% am, 58% pm from passersby)	78
845	Service Station with Convenience Market	35
850	Supermarket	95
851	Convenience Market (Open 24 Hours)	29
861	Discount Club	218
890	Furniture Store	10
912	Drive-In Bank	44

Table A-2 Signal-Timing Splits for Experiments C and D

Q_1 (vph)	Q_2 & Q_3	Q_T (vph)	Split	Phase I (sec)			Phase II (sec)			Total (sec)
				Green	Yellow	Red	Green	Yellow	Red	
250	125	500	0.80	115	4	1	25	4	1	150
		450	0.78	112	4	1	28	4	1	150
		400	0.76	109	4	1	31	4	1	150
		300	0.71	101	4	1	39	4	1	150
		100	0.67	95	4	1	45	4	1	150
500	250	1000	0.80	115	4	1	25	4	1	150
		950	0.79	114	4	1	26	4	1	150
		900	0.78	112	4	1	28	4	1	150
		800	0.76	109	4	1	31	4	1	150
		600	0.71	101	4	1	39	4	1	150
		100	0.67	95	4	1	45	4	1	150
750	375	1500	0.80	115	4	1	25	4	1	150
		1450	0.79	114	4	1	26	4	1	150
		1400	0.79	113	4	1	27	4	1	150
		1300	0.78	111	4	1	29	4	1	150
		1100	0.75	107	4	1	33	4	1	150
		600	0.67	95	4	1	45	4	1	150
1000	500	2000	0.80	115	4	1	25	4	1	150
		1950	0.80	114	4	1	26	4	1	150
		1900	0.79	114	4	1	26	4	1	150
		1800	0.78	112	4	1	28	4	1	150
		1600	0.76	109	4	1	31	4	1	150
		1100	0.69	98	4	1	42	4	1	150
1250	625	2500	0.80	115	4	1	25	4	1	150
		2450	0.80	115	4	1	25	4	1	150
		2400	0.79	114	4	1	26	4	1	150
		2300	0.79	113	4	1	27	4	1	150
		2100	0.77	111	4	1	29	4	1	150
		1600	0.72	103	4	1	37	4	1	150
1500	750	3000	0.80	115	4	1	25	4	1	150
		2950	0.80	115	4	1	25	4	1	150
		2900	0.79	114	4	1	26	4	1	150
		2800	0.79	113	4	1	27	4	1	150
		2600	0.78	111	4	1	29	4	1	150
		2100	0.74	106	4	1	34	4	1	150
1750	875	3500	0.80	115	4	1	25	4	1	150
		3450	0.80	115	4	1	25	4	1	150
		3400	0.80	114	4	1	26	4	1	150
		3300	0.79	114	4	1	26	4	1	150
		3100	0.78	112	4	1	28	4	1	150
		2600	0.75	107	4	1	33	4	1	150

APPENDIX B

Example Application

The following example application is provided to illustrate use of the procedure. Assume that an existing four lane arterial street is being re-designed and this procedure is to be used to configure the median.

Task 1: Determine whether any left turn treatment should be provided between intersections.

1a: Safety criteria
If the existing median permitted left turns into driveways, historical left-turn driveway associated accident rates should be determined. Four or more annual left-turn driveway oriented accidents for any driveway indicates safety criteria would justify left turn treatment. If safety criteria demand median treatment, steps 1b and 1c can be skipped. However, for example purposes, assume historical accident data are not available so safety criteria cannot be checked. Therefore go to step 1b.

1b: Operational criteria
Use chart set Table 4.1 to determine if any driveways would be expected to cause unacceptable left-turn delay. Assume: opposing 2-lane ADT is 24,000, through volume is 400 vph, and speed is 45 mph. Based upon these assumptions, use Table 4.1-C and note that all driveways 330 feet or more from an intersection and having left turn demands of 100 or more vehicles per hour would be expected to have 35 or more seconds of left turn delay. One or more driveways meeting these criteria would justify left turn treatment. Then check for unacceptable delay to through traffic caused by left-turns into driveways using chart set Table 4.2. Note that any driveway with hourly left turn demand greater than 400 vph would justify left turn treatment based on Table 4.2-C.

1c: Calculation of capacity and delay is optional and not normally expected

For example purposes, assume that at least one of the criteria in 1a or 1b has been satisfied and some median treatment is justified. Therefore, go to Task 2.

Task 2: Choose Raised or Flush median design

2a: Based upon safety criteria, choose raised median if 85th percentile speeds exceed 45 mph or 24 hour design two-directional traffic volume exceeds 24,000.

2b: Raised medians are recommended if design hour congestion is anticipated. Three rules of thumb can be used to identify likely congestion conditions. These include anticipated intersection queues exceeding 10 vehicles in any lane, queues not completely dissipated during signal green or demand-capacity ratio for any intersection lane group exceeding 0.9.

For example purposes, assume at least one of the criteria in 2a or 2b has been met and raised medians are recommended. Therefore, continue with Task 3R. Note if flush medians had been selected the process would continue with Tasks identified with the suffix F (flush medians).

Task 3R: Determine the need for intersection left-turn bays

Given that a raised median design has been selected, right-of-way is probably available for intersection turn bays and they would usually be provided. However, a procedure has been included for determining the need for intersection left-turn bays and it would be applied as follows:

Compute the threshold left-turn demand that would justify a left-turn bay in two steps. First, Q_w the threshold volume assuming no left-turn vehicles in the opposing flow is computed using equation 4.8 $Q_w = f_cQ_c(G/C) - e_oQ_o$. For the example, assume opposing flow $Q_o = 800$ vph, cycle length C = 80 seconds, green time serving the left-turn flow = 50 seconds, and through traffic volume in the median (left) lane = 300 vph. Then using Table 4.4, Q_o C/G = 800(80/50) = 1280 which is between 1000 and 1600 and with the median lane volume of 300, values of $e_L = 3.4$, $e_o = 0.290$, $Q_c = 590$ and $f_c = 0.82$ to 0.87 are read from the eighth line. Therefore, Qw = .82(590)(50/80) – 0.290(800) = 70. And this threshold left-turn volume is corrected for the presence of left-turn vehicles in the opposing flow, using equations 4.9 and 4.10. For the example, assume the opposing lane with the largest percentage of the opposing flow is the right lane and it carries 60 percent of the opposing flow, and remembering there are two lanes each direction, N = 2, then, a = 0.317(0.6 – ½) = 0.0317, and the corrected threshold volume is Qw = 70 – 0.0317(800) = 45 vph

For example purposes, assume the left-turn demand is 125 vph so the demand exceeds the threshold and a left-turn bay should be provided. Then, the next step is estimation of the required left-turn bay length.

Task 4R Determine the minimum length of intersection left-turn bay

Figure 4.1 can be used directly so one would read the maximum left-turn queue length if the green time per cycle ration is 0.5, otherwise, an adjusted opposing volume is computed using equation 4.11 and the adjusted volume is used in the Figure. Since the G/C ratio for the example is 50/80, equation 4.11 issued. Therefore, Qo = Qo / (2(G/C)) = 800 / (2(50/80) = 640 vph. Entering the chart with the left-turn demand of 125 vph and interpolating the maximum left-turn queue would be about 8 or 9 vehicles. A queue longer than 8 or 9 vehicles would very rarely occur (less than 5 percent likelihood).

This value can be multiplied by an average vehicle length plus clear space such as 18 feet, producing a left-turn bay length of 144 feet. The last step is assessment of potential midblock median openings.

Task 5R Assessment of midblock median openings

Any proposed median opening should be tested using three criteria: potentially excessive left-turn delay that could lead to impatient drivers unsafely crossing opposing traffic streams to enter a driveway, ability to provide adequate storage or bay length, and distance to intersections or other median openings. For example purposes, assume three median opening requests are to be examined and these are, respectively, 200, 300, and 450 feet from the upstream intersection and the left-turn demand for entrance into the potential openings would be 150, 210, and 75 vph respectively. If left-turn demand for driveway entrance is unknown, Table 4.3 can provide estimates of such values based upon the land use served by the driveway.

5Ra: Checking for excessive left turn delay
Chart series Table 4.5 can be used to identify potential excessive delay values. Using Table 4.5-B for the 45 mph condition of the example, the 24,000 ADT of the arterial the following is determined:

1. The proposed opening 200 feet from the upstream intersection has a left-turn volume of 100 vph so it will not have excessive delay.
2. The proposed opening at 300 feet from the intersection with a 210 vph left-turn demand will have excessive delay.
3. The proposed opening at 450 feet from the intersection having a 75 vph demand will not have excessive delay.

Therefore, the proposed openings at 200 and 450 feet from the upstream intersection are okay while the one at 300 feet should not be provided and drops out of the analysis.

5Rb: Determining the minimum left-turn bay lengths for proposed median openings at 200 and 450 feet from the upstream intersection

Using the procedures of Task 4R, and Figure 4.1, both openings have left-turn demands exceeding the 45 vph threshold for bay provision maximum left-turn queue lengths of 7 and 5 would occur for the 200 and 450 feet openings respectively. Multiplying by 18 feet per vehicle plus space produces values of 126 and 90 feet.

If these storage areas cannot be provided because of utility, street hardware or other requirements, the proposed openings for which adequate bay lengths cannot be provided should be deleted from further analysis.

5Rc: Checking minimum distances to other median openings and intersections.

Using Table 4.6 and assuming the arterial in question has been designated an Access Class 5 facility, which is to have property access as well as, serve through traffic, the following is determined.

The minimum median opening spacing is 660 feet so both the 200 and 450 feet openings cannot be provided. Usually the opening with the higher traffic demand would be chosen so the opening at 200 feet would be approved and the one at 450 feet would be denied. Note that this conclusion is based upon the concept that this opening would provide access to a driveway on only one side of the street. If the opening was to serve both sides, the criteria would call it a "full" opening and minimum spacing would be 1320 feet.

Therefore, only one of the three proposed openings would be approved, namely the one at 200 feet from the upstream intersection. The one at 300 feet was denied due to potentially excessive delay which would likely lead to a signal request in the future. Note also that if the first opening is approved, the one at 300 feet would also be denied based on the spacing criteria. The opening at 450 feet would be denied based on the spacing criteria.