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LEFT-TURN LANE DESIGN AND OPERATION

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16. Abstract

This project examined important issues related to the design and operation of left-turn lanes. It developed an analytical model for determining the queue storage lengths of left-turn lanes at signalized intersections by considering both parts of left-turn queue: (1) the vehicles that arrive during the red phase (red-phase queue), and (2) the queue of vehicles carried over from previous cycles (leftover queue). The evaluation results indicated that the developed model considerably outperforms the existing methods by providing more accurate estimates of left-turn queue lengths. In addition, a decision-making flowchart for installing multiple left-turn lanes was developed by combining the warrants in four categories: (1) capacity and volume based, (2) left-turn queue length based, (3) safety based, and (4) geometric condition based warrants. Furthermore, the safety benefits of extending the lengths of left-turn lanes were analyzed and the analysis results indicated that extending the length of left-turn lanes will significantly reduce the rear-end accident risk at intersections. Finally, this research investigated the left-turn bay taper length estimation and the impacts of signal phasing sequence on left-turn operation. It recommended using different sets of bay taper lengths for the intersections in the urban and non-urban areas and it suggested the way to select the proper signal phasing sequence for the intersections with left-turn lane overflow and/or blockage problems.

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DISCLAIMER

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NOTICE

The United States Governments and the states of Texas do not endorse products or manufacturers. Trade or manufactures' names appear herein solely because they are considered essential to the object of this report.

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SUMMARY

Left-turn lanes can improve the safety and operation of intersections by providing space for deceleration and storage of vehicles waiting to make a left turn. Insufficient length can result in the left-turn lane overflow and the blockage of left-turn lane entrance by through traffic, which seriously compromises both the operation and safety of an intersection. The left-turn lane problem is very complicated involving design, operational, as well as safety issues. Generally, field engineers face following three critical questions in the design of left-turn lanes:

1. How long should the left-turn lane be?
2. When and where should multiple left-turn lanes be provided?
3. What are the safety benefits of extending the length of existing left-turn lanes?

The exiting methods have limitations in recommending appropriate queue storage lengths for left-turn lanes. In addition, there is a lack of guidelines for installing multiple left-turn lanes. Furthermore, few studies have been conducted for quantifying the safety effectiveness of extending the length of existing left-turn lanes.

This research is to examine important issues related to the design and operation of left-turn lanes and recommend best practices that could improve both safety and efficiency of intersections. To this end, the research entails the following specific objectives: (1) synthesize national practices from other states on the design and operation of left-turn lanes, (2) identify important parameters/variables that are associated with the determination of deceleration and queue storage length requirements for left-turn lanes, (3) develop procedures and methodologies for determining queue storage lengths for both signalized and unsignalized intersections, (4) develop criteria for determining when to install multiple left-turn lanes, (5) determine safety benefit resulted from the increased queue storage length, and (6) examine other relevant elements

associated with the design and operation of left-turn lanes. To meet these objectives, a strategic work plan consisting of 12 tasks was implemented. Following are the descriptions of the work that has been conducted in the research and the key results/findings.

First, a review of the state-of- the-art and the state-of-the-practice was conducted. This literature review focuses on the studies on two topics: (1) the warrants for left-turn lanes, and (2) the queue storage length of left-turn lanes. It was found that the rule of thumb method for estimating queue storage length recommended by TxDOT Roadway Design Manual does not consider the factors that affect the departure rate of the intersection, which will cause overestimation of left-turn queue length at the intersections with high left-turn volume and high service rate. For the analytical methods, the accuracy of the existing models is affected by various facts and the existing models cannot model the queue forming process at signalized intersections very well. In addition, the selection and application of the traffic model for left-turn storage length estimation need to be investigated.

Second, to identify and prioritize the important parameters and variables that are essential to the determination of deceleration and storage length requirements for left-turn lanes, a survey was conduced to the field engineers. This survey was intended to seek information on criteria for multiple left-turn lane installation. Most of the survey respondents indicated that the guidelines provided by TxDOT Roadway Design Manual is used for determination of deceleration and storage length and there are few existing warrants for multiple left turn lanes. In addition, following critical issues regarding the left-turn lane design and operation were identified: (1) right of way issue (not enough space for installation or for future development), (2) the exiting methods yield short taper lengths and longer deceleration lengths, and (3) long left-turn lanes may block the access to driveways for the opposing left turn traffic. In addition, the following constructive suggestions were provided: (1) in the peak hour, due to relatively low traffic speed, the deceleration length could be shorter, and (2) using functional classifications of the roadway and cross street instead of future traffic volumes to determine the length of left turn lane.

Third, field data were collected from 28 selected intersections in Houston and Austin districts. The collected data can be categorized into four groups: traffic flow information, signal timing information, intersection geometric information, and historical accident data. Different methods were used for collecting these groups of data, including obtaining information from

Traffic Management Centers, field visiting, and recording traffic video. The data collection covered a wide range of intersections with different congestion levels (Volume to Capacity ratios, i.e. *v/c* ratios), left-turn signal control modes and different types of left-turn lanes. It was found that, although all of the 28 intersections were subject to undersaturated conditions, the left-turn queue carryover problem occurred frequently for the intersections with left-turn *v/c* ratios within the range of 50% to 80%. The collected data will be used to develop and validate the methodology for determining left-turn storage length, and to analyze the safety benefit of extending the length of left-turn lanes.

Fourth, a new analytical model (TSU model) for determining the queue storage lengths of left-turn lanes at signalized intersections was developed by considering both parts of left-turn queue: (1) the vehicles that arrive during the red phase (red-phase queue), and (2) the queue of vehicles carried over from previous cycles (leftover queue). The evaluation results indicated that the developed model considerably outperforms the existing methods by providing more accurate estimates of left-turning queue lengths.

Fifth, the traffic model-based procedures to determine the required deceleration and storage length requirements were examined. For left-turn storage length estimation, it found that, among the three selected traffic simulation models, i.e. *SYNCHRO*, SimTraffic and VISSIM, SimTraffic model illustrates the best performance, VISSIM demonstrates relatively poor performance and the developed analytical model (TSU model) outperforms the selected traffic simulation models. For left-turn deceleration length estimation, a simulation-based method was developed by using VISSIM 4.20. It provides better deceleration length estimates than those recommended by analytical methods.

Sixth, the safety benefits of increasing the storage lengths of existing left-turn lanes at intersection were analyzed by two methods: (1) accident data analysis, and (2) simulation-based safety analysis. It was found that (1) the average rear-end accident at the intersections with left-turn overflow problem was 35 percent higher than that at the intersections without left-turn overflow problem; and (2) after extending the lengths of the left-turn lanes to eliminate the overflow problem in the study intersections, all of the safety surrogate measures derived from the traffic simulation results, changed significantly in a direction that indicated the reduction of rear-end accident risk at those intersections. These results concluded that extending left-turn lanes to eliminate the left-turn lane overflow problem significantly improved intersection safety by decreasing the rear-end accident risk.

After that, to develop comprehensive guidelines on multiple left-turn lane installation, the operational and safety impacts of multiple left-turn lanes on left-turn operation were analyzed. As a result, two types of warrants for multiple left-turn lanes were developed: (1) the capacity and volume based warrants, and (2) the left-turn queue length based warrants. By combining the developed warrants with the existing warrants/guidelines, a decision-making flowchart for installing multiple left-turn lanes was developed.

Finally, two important issues related to left-turn lane design and operation were examined: (1) left-turn bay taper length estimation, and (2) the impacts of signal phasing sequence on left-turn operation. By comparing the existing methods and guidelines on left-turn bay taper length estimation, two different sets of bay tapers length was recommended for the intersections in urban areas and non-urban areas. Then, based on the results of traffic simulation studies, it was found that the vehicle delay caused by the overflow and blockage problem could be significantly reduced by choosing appropriate signal phasing sequence.

Based on the results of this research, following recommendations on the left-turn lane design and operation were made: (1) left-turn lane should be designed with adequate storage length, (2) multiple left-turn lanes should be provided when left-turn volume exceeds its capacity, resulting in high traffic delay and extreme long left-turn queue, (3)extend the length of left-turn lane or update the single left-turn to multiple left-turn lanes for the intersections with left-turn lane overflow problem to reduce the rear-end crash risk, (4) longer bay taper lengths should be provided for intersections in the non-urban areas, and (5) appropriate signal phasing sequence should be adopted to reduce the delay caused by left-turn lane overflow and blockage problems.

CHAPTER 1

INTRODUCTION

1.1 Background

Left-turn lanes are provided at intersections to improve the safety and operation of intersections by providing space for deceleration and storage of left-turn vehicles (see Figure 1 for the illustration of single left-turn lane). It reduces the shock wave effect caused by vehicle speed difference between through and left-turn vehicles. Shock waves occur when left-turning vehicles are forced to decelerate in the through lanes, thereby causing through traffic to decelerate. Eliminating conflicts between left turning vehicles decelerating or stopping and through traffic is an important safety consideration. The installation of left-turn lanes at intersections substantially reduces rear-end accidents. A major synthesis of research on left-turn lanes conducted by Gluck, et al. (1999) demonstrated that exclusive turn lanes reduce crashes between 18 and 77 percent (50 percent average) and reduce rear-end accidents between 60 and 88 percent. Furthermore, the flow of traffic through intersections will be improved by ensuring that left-turn lanes are designed with lengths sufficient to meet storage and deceleration requirements.

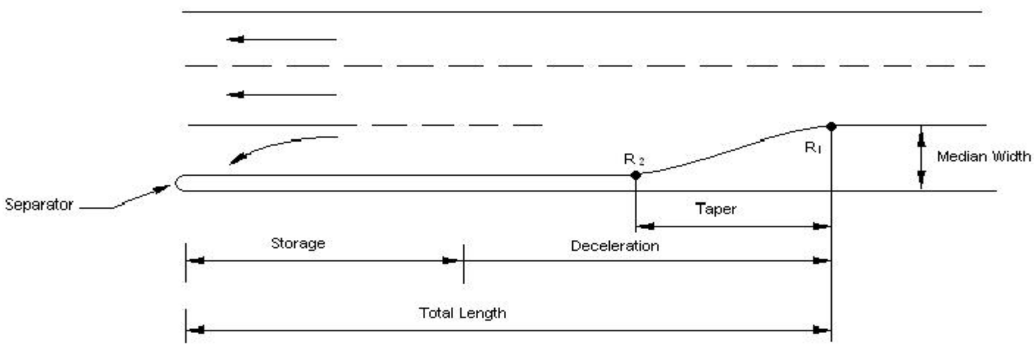


Figure 1: Illustration of Single Left-Turn Lane

On the other side, insufficient length of left-turn lane will result in the left-turn lane overflow and the blockage of left-turn lane entrance by through traffics, which were referred to as left-turn overflow and blockage problems in this study (see Figure 2). These two problems will seriously increase the traffic delay and accident risk at intersections.

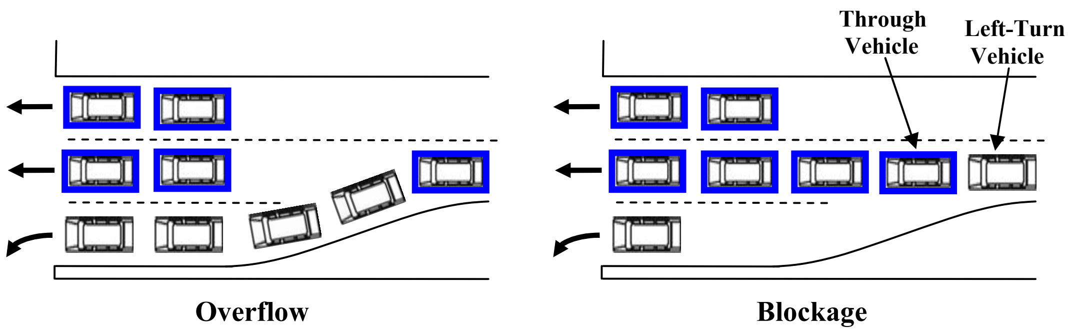


Figure 2: Left-Turn Overflow and Blockage Problems

The design and operation of left-turn lanes involve a comprehensive set of factors associated with the geometric, traffic, and control elements. These factors include, but are not limited to, the left-turn traffic volume, opposing traffic volume, annual average daily traffic of the intersection, approach grade, posted speed limit, percentage of truck/large vehicles, intersection signal control features, etc. Without understanding these essential factors, it would be impossible to design safe and efficient left-turn lanes.

The left-turn lane problem is very complicated involving design, operational, as well as safety issues. Generally, field engineers face following three critical questions in the design of left-turn lanes:

1. How long should the left-turn lane be?
2. When and where should multiple left-turn lanes be provided?
3. What are the safety benefits of extending the length of existing left-turn lanes?

Following are the existing practices for addressing these three questions.

Length of Left-Turn Lane

The length of the left-turn lane is critical in the design of left-turn lanes. The required physical length of a left-turn lane is the sum of the distance required for the driver to move laterally into the left-turn lane and decelerate to stop (deceleration length) plus the required queue storage length. The deceleration length depends on the speed of the vehicles in different locations. The storage length should be sufficient to have a high probability of storing the longest expected queue.

For the determination of queue storage length of left-turn lanes, generally there are three different types of methods: 1. Rule of thumb methods (recommended by TxDOT Roadway Design Manual), 2. Analytical methods (queuing theory based method), and 3. Traffic model based methods. These existing methods have limitations in recommending appropriate queue storage lengths for left-turn lanes. For example, the rule of thumb methods recommended by TxDOT Roadway Design Manual does not consider the factors that affect the departure rate of the intersection, which will cause overestimation of left-turn queue length at the intersections with high left-turn volume and high service rate. For the analytical methods, the accuracy of the existing models is affected by various facts and the existing models cannot model the queue forming process at signalized intersections very well. For the traffic model based method, the selection of a right traffic model for left-turn queue length estimation needs to be investigated. In addition, the network coding and model calibration usually takes ample amount of time and efforts. The detailed discussion of these existing methods will be provided in the literature review part of this report (Chapter 2).

Multiple Left-Turn Lanes

Multiple left-turn lanes (dual or triple) may be required to accommodate high left-turn volumes at the intersections. There are few guidelines on the installation of multiple left-turn lanes. The capacity and volume based warrants has been widely used for multiple left-turn lane installation. However, most of them just use a constant left-turn volume threshold as a warrant for multiple left-turn lane installation and different states choose different thresholds. There is a lack of detailed explanations for the development of these warrants and most of them were developed based on engineering judgment instead of systematic intersection performance analysis. Thus, this study is to develop criteria for installing multiple left-turn lanes based on intersection operational and safety analysis.

Safety Benefits of Extending Left-Turn Lanes

Most of the studies on left-turn lane safety analysis have focused on the safety impacts of installing the left-turn lanes, and there are relatively few studies that examine the safety impacts of extending the lengths of existing left-turn lanes. A recent FHWA study (Harwood et al., 2002) noted that no research was found that quantifies the safety effectiveness of extending the length of existing left-turn lanes to eliminate traffic overflow into through travel lanes and to allow a greater proportion of vehicle deceleration to occur in the turn lane rather than in the through travel lanes. Therefore, to fill this gap, this research will investigate the safety impacts of increasing the lengths of left-turn lanes at intersections. By ensuring the left-turn lanes designed with sufficient lengths that meet the storage and deceleration requirements, the potential accident risk caused by left-turn lane overflow problem will be reduced.

1.2 Research Goals and Objectives

Based on the context provided in the above background of the research, the proposed project is intended to achieve the following goals: *examine important issues related to the design and operation of left-turn lanes and recommend best practices that could improve both safety and efficiency of intersections*. To this end, the research involves the following specific objectives:

- Synthesize national practices from other states on the design and operation of left-turn lanes,
- Identify important parameters/variables that are associated with the determination of deceleration and queue storage length requirements for left-turn lanes,
- Develop procedures and methodologies for determining queue storage lengths for both signalized and unsignalized intersections,
- Determine criteria for determining when to install multiple left-turn lanes,
- Determine safety benefit resulted from the increased queue storage length, and
- Examine other relevant elements associated with the design and operation of left-turn lanes

1.3 Outline of This Report

This is the project report covering all tasks conducted during the research period. In the following chapters of this report, major existing methodologies proposed or adopted by different US agencies will be presented first. Then, the survey for identifying the important parameters on left-turn deign and operation will be presented and the survey results will be analyzed. Third, the data collection will be described in Chapter 4. In Chapter 5, a new methodology for the determination of storage length of left-turn lanes at signalized intersections will be proposed. For unsignalized intersections, an existing method will be recommended. In chapter 6, procedures with traffic models for determining the required deceleration and storage length requirements will be examined. Then, the safety benefits of increasing the storage lengths will be analyzed in chapter 7. In Chapter 8, the criteria for installing multiple left-turn lanes will be developed. After that, other elements related to left-turn lanes will be examined in Chapter 9. Finally, conclusions and recommendations will be given in the last chapter.

CHAPTER 2

LITERATURE REVIEW

Left-turn lanes are used to improve safety and/or operations of the intersections. A number of studies have been conduced for the improvement of the left-turn design and operation. Most of these studies involve the following two critical topics:

1. Where should left-turn bays (lanes) be provided?
2. How long should the left-turn lane be?

For the second question, it is important to find how long the queue storage length of the left-turn lanes should be. Therefore, this literature review will focus on the studies on two topics: 1) the warrants for left-turn lanes, and 2) the queue storage length of left-turn lanes.

2.1 Warrants for Left-Turn Lanes

Various studies have been conducted for developing guidelines, standards, or warrants for the design of left-turn lanes. The traffic volume-based left-turn lane warrants, proposed by Harmelink (1967) and standardized by the AASHTO Green Book (2001), is one of the first guidelines for unsignalized intersections. Later, Agent (1983) developed a set of left-turn lane warrants by considering multiple criteria, including accident rate, traffic volume (left-turn and opposing volume) and traffic conflicts. In a recent study conducted by University of Virginia (Lakkundi et al., 2004), a new set of traffic volume-based left-turn lane warrants were developed for both unsignalized and signalized intersections. In the following sections, different types of left-turn lane warrants that were developed by these three studies will be introduced in details.

2.1.1 Studies on Warrants for Left-Turn Lanes

<u>Harmelink (1967) and AASHTO Green Book (2001)</u>

Harmelink (1967) derived warrants for left-turn lanes at unsignalized intersections based on the assumption that left-turn storage lanes should be provided at the locations where the through vehicles were blocked by left-turn vehicles and the probability of this occurrence should be lower than a given critical value. The queuing theory was applied to calculate the probability that there are some vehicles waiting in the queue for making left-turn, which can be mathematically expressed as follows.

$$\textit{Prob (number of left-turn vehicles in the queue} > n) = \rho^n < a \qquad (1)$$

where:

ρ = utilization factor and $\rho = \lambda/\mu$

λ = average arrival rate (vph)

μ = average service rate (vph)

n = number of vehicles

α = a given critical value

Different numbers of vehicles (n) are selected for different types of roadways according to the minimum length of left-turn queue that will affect the movement of through vehicles. In the divided four-lane highways, n is equal to 2 because more open median space is available for storing left-turn vehicle while n is equal to 1 for undivided four-lane and two lane highways. In addition, for the probability in Equation (1), different critical values were also selected for different types of roadways with different approach speeds, which were summarized in Table 1.

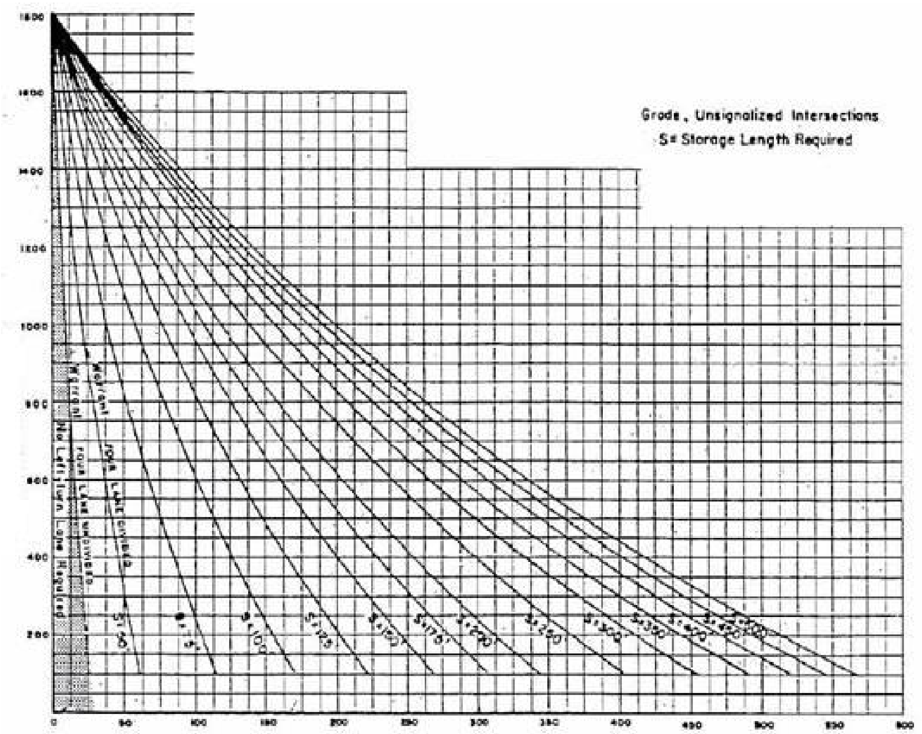
Table 1: Probability Values used in Harmelink Guidelines

Source: Harmelink (1967)

	Speed	Critical Value α
Divided four-lane highways	All range	0.005
Undivided four-lane highways	All range	0.03
Two-Lane Highways	40 mph-50mph	0.02
	50 mph-60mph	0.015
	60 mph-70mph	0.01

Thus, based on Equation (1), the relationship between average arrival rate λ and average service rate μ can be derived at the required critical probability levels. Since the average arrival rate is the function of left-turn volume/advancing volume and the average service rate is the function of opposing volume, the relationship between left-turn volume/advancing volume and opposing volume can also be derived. This was expressed by a series of design charts (see Figures 3 and 4, as examples).

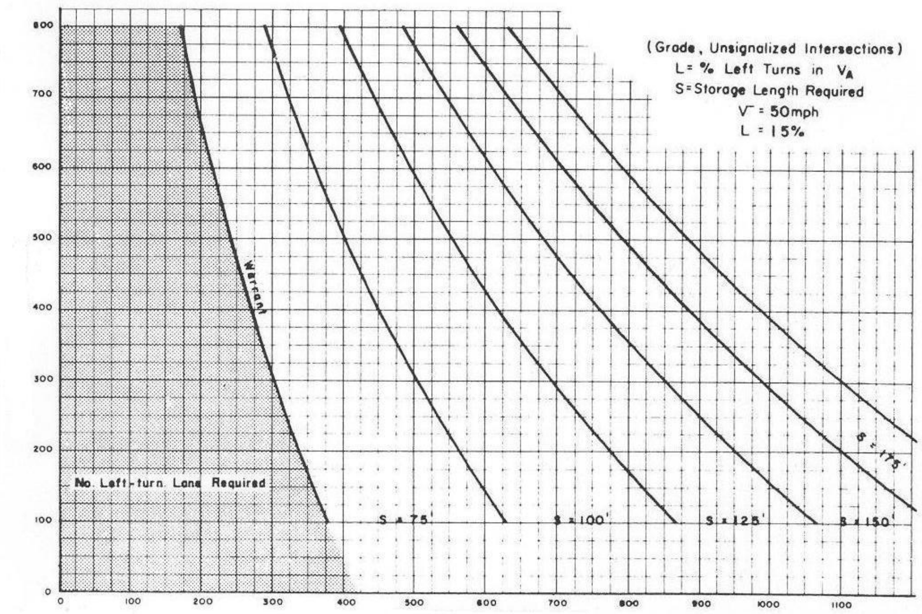
Opposing Volume (VPH)



Left-Turn Volume (VPH)

Figure 3: Warrant for Left-Turn Lanes, Four-Lane Highways
Source: Harmelink (1967)

Opposing Volume (VPH)



Advancing Volume (VPH)

Figure 4: Warrant for Left-Turn Lanes, Two-Lane Highways

Source: Harmelink (1967)

The design charts in Figures 3 and 4 present the warrants for left-turn lane. A left-turn lane with the designed shortage length (s = 60, 75… 500) will be warranted for the intersections where the advancing and opposing volumes lie above of these curves. Other design charts/warrants for two-lane highway with different approach speeds and different L_A (proportion of left turns in advancing volume) can be found in Harmelink (1967).

Based on the information presented in the design charts developed by Harmelinks (1967), AASHTO Green Book (2001) summarized the left-turn lane warrants for two-lane highways into a table as follows (see Table 2).

Table 2: Guide for Left-Turn Lanes on Two-Lane Highways
Source: AASHTO Green Book (2001)

Opposing Volume (veh/h)	Advancing Volume (veh/h)			
	5% Left Turn	10% Left Turn	20% Left Turn	30% Left Turn
40-mph Operating Speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50-mph Operating Speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
60-mph Operating Speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Table 2 provides guidelines for the installation of left-turn lanes based on the opposing traffic volume, the advancing volume, the operating speed, and the percentage of left-turning traffic. For example, at a two lane highway with 50-mph operating speed, 195 vph advancing volume, 20 percent of left-turning traffic and 600 vph opposed vehicles, the minimum warranting left-turn volumes are 195 mph. Note that the Harmelink's guidelines is only for unsignalized intersection and the safety impacts of the left-turn lane haven't been very well considered in the development of warrants.

<u>Agent (1983)</u>

Agent (1983) developed a set of warrants for left-turn lanes by considering multiple criteria: accident rate, traffic volume (left-turn and opposing volume) and traffic conflicts.

Accident Warrants

At first, to understand the safety impact of the left-turn lanes, Agent (1983) compared left-turn-related accident rates (accidents per million left-turning vehicles) at intersections with and without left-turn lanes based on 5-year historical accident data as shown in following table below.

Table 3: Accident Rates at Intersections with and without Left-Turn Lanes
Source: Agent (1983)

		Accident Rate (Left-Turn Accidents Per Million Left-Turn Vehicles)
Unsignalized	No Left-Turn Lane	5.7
	With Left-Turn Lane	1.3
Signalized	No Left-Turn Lane	7.9
	With Left-Turn Lane	3.6
	With Left-Turn Lane and Phasing	0.8

Table 3 shows that the accident rates of the intersections with left-turn lanes are significantly less than that of the intersections without left-turn lanes for both unsignalized and signalized intersections. Based on these results, he recommended installing left-turn lanes if the left-turn-related accident rates are higher than the critical accident rate (number of left-turn-related accidents per year) given by Equation (2).

$$N_c = N_a + K\sqrt{N_a} + 0.5 \qquad (2)$$

where:

N_c = critical number of accidents per year

N_a = average number of accidents (at unsignalized and signalized intersections: 0.8 and 1.2 left-turn accidents per approach per year, respectively)

K = constant, related to level of statistical significance (for P equals to 0.95 and 0.995 are 1.645 and 2.576, respectively)

Volume Warrants

Agent (1983) also developed volume warrants based on the assumption that left-turn storage lanes should be provided at the locations where left-turn traffic caused significant delay (the level of service of the intersection is less than grade C). In this study, a computer simulation method was used to find the relationships between traffic delay (or load factor) and other variables such as percentage left turns, traffic volume, cycle length, cycle split, and number of opposing lanes. The simulation was conducted for both signalized and unsignalized intersections. Based on the simulation results, charts for the relationships between approach delay and the opposing volume or percentage left turns were developed. Then, by selecting a critical delay of 30 seconds, the critical sums of peak-hour left-turn and opposing volumes for different types of intersections with different signal timing features were derived and presented in Table 4.

Table 4: Critical Sum of Left-Turn and Opposing Volumes during the Peak Hour for Creating a Left-Turn Delay Problem

Source: Agent (1983)

Signalized Intersection (Four-Lane Highway)			
	Cycle Split		
Cycle Length	70/30	60/40	50/50
120	950	800	600
90	1,000	850	700
60	1,150	1,000	850
Signalized Intersection (Two-Lane Highway)			
	Cycle Split		
Cycle Length	70/30	60/40	50/50
120	650	550	400
90	700	600	500
60	750	650	550
Unsignalized Intersection (Two-Lane Highway)			
Delay Criterion	Four-Lane highway		Two-Lane Highway
30 seconds	1,000		900
20 seconds	900		800

Traffic Conflicts Warrants

The safety of an intersection is usually examined based on its crash history. However, crashes are rare events and no crash history is available for some intersections that were newly constructed or updated. Thus, Agent (1983) also developed left-turn warrants based on the traffic conflict data collected from the field observation. Conflict analysis is one type of safety surrogate, which was defined as events such as near misses or sudden braking for vehicles on the verge of a collision hazard.

In this study, agent categorized left-turn-related accidents into five groups and compared the number of conflicts for each category at the intersections that meet and do not meet the accident warrants (see Table 5 for the details). To determine which types of conflicts were mostly related to the accident, Agent also developed regression equations for estimating the number of conflicts based on the number of accidents at intersections. With this confirmed, the number of required conflicts can also be estimated based on the critical number of accidents that warrants a left-turn lane. This result was also presented in the Table 5.

Table 5: Methods of Developing Traffic Conflict Warrants
Source: Agent (1983)

Type of Conflict		Critical Traffic Conflict Level for Given Method: Average Value at Locations Meeting Accident Warrant	Upper Level of Confidence Interval at Locations Not Meeting Accident Warrant	Determine the Critical No. of Conflicts based on the Critical No. of Accidents by using the Developed Regression Equations
Total of left-turn related conflicts	Peak hour*	45	37	38
	Average**	30	26	26
Opposing left turn	Peak hour*	8.7	5	6.0
	Average**	5.9	3	3.8
Slowed for left turn	Peak hour*	23	22	20
	Average**	15	15	14
Previous left turn	Peak hour*	14	11	12
	Average**	7.9	8	7.3
Weave (involving left-turning vehicle)	Peak hour*	4.4	3	3.4
	Average**	2.2	1.6	1.7

* the highest one-hour number of conflicts
** average number of conflicts in the three pick hours

According to the results presented in Table 5, Agent recommended that a left-turn lane needs to be installed when a conflict study shows one of the following conditions:

a) An hourly average of 30 or more total left-turn-related conflicts or 6 or more opposing-left-turn conflicts in a 3-hour study period during peak-volume conditions.
b) 45 or more total left-turn-related conflicts or 9 or more opposing-left-turn conflicts occur in 1-hour period.

<u>Lakkundi et al. (2004)</u>

Lakkundi et al. (2004) developed left-turn warrants for both unsignalized intersections and signalized intersections (with simple two phases) by using traffic simulation method. The guidelines for unsignalized intersections were developed based on the assumption that the probability of left-turn blocking through vehicles should be very low. The critical probability proposed in Harmelink (1963) was adopted in this study. Multiple simulation runs were made for each combination of opposing and advancing vehicle volumes, and left-turn percentages under varying operating speed conditions. Based on the results of simulation, warrants in the form of charts that show the relationship between the advancing and opposing traffic volume at given critical probability levels were developed (see Figure 5). Left-turn lanes were warranted for the intersections where the advancing and opposing volumes lie above the guideline lines. Figure 5 is a sample left-turn warrant developed under the condition of 3% left-turn vehicles in the advancing volume. A complete set of warrants can be found in Lakkundi et al. (2004).

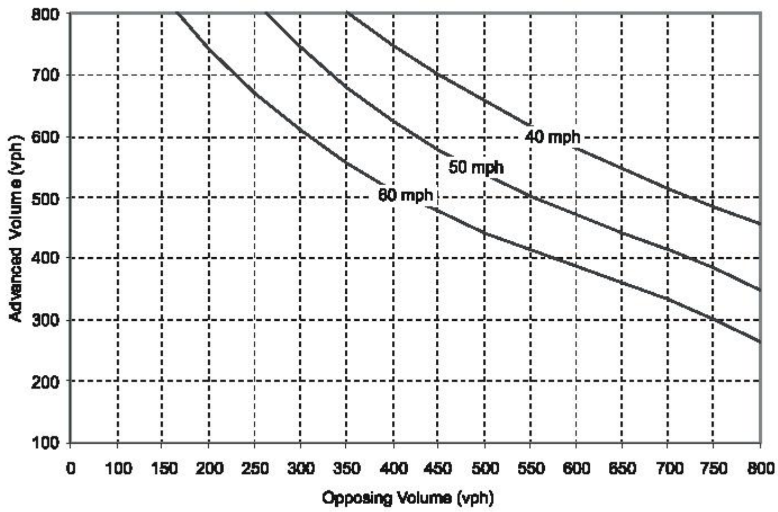


Figure 5: Left-Turn Lane Guidelines at Unsignalized Intersections for 3% Left-Turn Vehicles on Advancing Volumes
Source: Lakkundi et al. (2004)

Guidelines for the signalized intersections were developed based on the assumption that the intersection delay caused by left-turn vehicles should be lower than 55 seconds (LOS=E) and the intersection Volume to Capacity ratio (*v/c*) should be less than 85%. The input variables for running simulation program for pretimed signals include: g/C (0.1 through 0.8 in 0.1 increments), cycle length (60, 80, and 100 seconds), number of lanes (four and six lanes), percentage of left-turning vehicles (3%, 5%, 10%, 20%, and 30%). Based on the results of multiple simulation runs, the left-turn warrants in the form of tables were developed (see Table 6). It was recommended that if the advancing traffic volume is above the minimum value given in Table 6, then the left-turn lane should be installed in that location.

Table 6: Left-Turn Lane Guidelines at Pre-timed Signalized Intersections (Two-Lane Approaches, Cycle Length of 60 sec)
Source: Lakkundi et al. (2004)

Opposing Volume (vph)	Advancing Volume (vph) for 3% Left-Turn						
	G/C=0.2	0.3	0.4	0.5	0.6	0.7	0.8
100	225	400	550	705	855	1005	1155
150	155	395	545	695	845	995	1145
200	50	365	540	685	840	985	1130
250	50	295	520	675	825	975	1120
300	50	75	500	665	810	965	1110
350	50	50	425	645	800	950	1095
400	50	50	245	630	785	935	1075
450	50	50	65	540	760	915	1055
500	50	50	50	395	740	890	1035
550	50	50	50	120	650	865	1020
600	50	50	50	55	515	830	1000
650	50	50	50	50	300	755	975
700	50	50	50	50	80	660	905
750	50	50	50	50	50	475	825
800	50	50	50	50	50	130	725
850	50	50	50	50	50	75	530
900	50	50	50	50	50	50	330
950	50	50	50	50	50	50	140
1000	50	50	50	50	50	50	50
1050	50	50	50	50	50	50	50
1100	50	50	50	50	50	50	50
1150	50	50	50	50	50	50	50
1200	50	50	50	50	50	50	50

In the case of actuated signal controls, there would be no guidelines developed in the form of either tables or charts due to the complexity of the problem. It was recommended to apply the warrants for the pretimed signalized to the actuated signalized intersections based on the estimated average cycle length and green times through multiple simulation runs.

2.1.2 Summarization/Comparison of Different Types of Warrants

Table 7 summarizes and compares different types of warrants.

Table 7: Summarization/Comparison of Different Types of Warrants

Studies	Major Criteria	Basic Assumptions	Influencing Factors
Harmelink (1967) and AASHTO Green Book (2001)	Volume-based warrants for unsignalized intersections: • Opposing and left-turn traffic volume for four-lane highways • Opposing and advancing traffic volume for two-lane highways	The probability of more than one/two left-turning vehicles waiting for making a left turn should be lower than a specific level. Two vehicles are for divided four-lane highways and one vehicle is for undivided four-lane and two-lane highways.	• Traffic volume: opposing, left-turn, advancing • Speed • Number of lanes • Divided/undivided
Agent (1983)	Accident-based warrant: historical rates of the left-turn related accidents	Left-turn lanes should be installed if the critical number of left-turn-related accidents had occurred.	• Historical rates of the left-turn-related accidents
	Volume-based warrants: Sum of opposing and advancing traffic volume	• The intersection delay caused by left-turn traffic should be lower than a critical value. • Load factor of intersection should be less than 0.3, which represent the upper bound of level of service (LOS) C.	• Traffic volume: opposing, left-turn, advancing • Cycle length • Cycle split • Number of opposing lanes
	Traffic-conflict-based warrants	The rate of a left-turn-related traffic conflicts at an intersection should be controlled in a suitable low level.	• Observed left-turn-related traffic conflicts
Lakkundi et al. (2004)	Volume-based warrants: opposing and advancing traffic volume	• Unsignalized intersection: similar to the Harmelink (1967) • Signalized intersection: to maintain left-turn delay lower than 55 seconds (LOS=E) and *v/c* ratio less than 85%	• Traffic volume: opposing, left-turn, advancing • Cycle length • Speed • Number of lanes

2.1.3 TxDOT Practice

In TxDOT Roadway Design Manual, the Harmelink (1967) left-turn warrants (or AASHTO Green book guideline) were provided for the two-lane highways in the rural areas. For the roadways in the urban areas and four-way roadways in the rural areas, no uniform guidance was provided. The installation of left-turn lane was decided based on the field engineers' experiential judgments.

2.2 Determination of Left-turn Storage Length

Once the decision of installing left-turn is made, it is important to determine how long the left-turn lane should be. The overflow of left-turn lane could significantly impact the safety and the operational efficiency of an intersection. The AASHTO Green Book (2001) provides following general instructions for both unsignalized and signalized intersections:

- At unsignalized intersections, the storage length can be estimated based on the number of turning vehicles likely to arrive in an average two-minute period in the peak hour. Note that this two-minute waiting time assumption could be changed for different intersections because it depends on the average time for completing the left-turn maneuver, which is affected by the volume of opposing traffic at a particular intersection. It also suggested that space for at least two passenger cars should be provided and space should be provided for at least one car and one truck for the intersection with over 10 percent truck traffic.
- At signalized intersections, the required storage length depends on the signal cycle length, the signal phasing arrangement, and the rate of arrivals and departures of left-turning vehicles. The storage length is usually based on one and half to two times the average number of arrival vehicles per cycle, which is predicated based on the design traffic volume. This length will be sufficient to serve heavy surges that occur from time to time. As in the case of unsignalized intersections, the storage length of left-turn lane should be long enough to store at least two vehicles.

Numerous studies have been conducted for determining the length of left-turn lanes, especially the storage length of left-turn lanes. In general, the methods for calculating the storage lengths of left-turn lanes can be categorized into three types: (1) Rule of Thumb Methods, (2) Analytical-Based Methods, and Simulation-Based Method. In the following sections, the typical methods in each category will be introduced and discussed in details.

2.2.1 Rule of Thumb Methods

The conventional rule of thumb method has been widely applied in practice due to its simplicity and easiness of implementation. It estimates the storage requirements of left-turn lanes based on the average left-turn volume per cycle for signalized intersections or per given time interval for unsignalized intersections during the peak hour. A general form of the rule of thumb methods can be expressed mathematically by following equation:

$$L = K\,(V/\,N_C)\,S \qquad \text{for signalized intersection}$$

and

$$L = K\,[V/(3600/I)]S \quad \text{for unsignalized intersection} \tag{3}$$

where:

- L = storage length (ft)
- V = left-turn flow rate during the peak hour (vph)
- K = a constant to reflect random arrival of vehicles (usually 2)
- N_C = number of cycles per hour (for signalized intersection)
- I = average vehicle waiting interval in seconds (for unsignalized intersection)
- S = average queue storage length per vehicle (average distance, front bumper-to-bumper of a car in queue)

Note that, the average storage length S in Equation (3) depends on the percentage of trucks or buses in the arriving vehicles. Usually, 25 ft is assumed as the average queue storage length when truck or bus percentage is less than 5%. An adjustment factor $K = 2$ is usually applied to account for random variations in vehicle arrivals which implies a failure rate of approximately 5 percent. However, when the variance of vehicle arrival rate decrease, for example the left volumes increase toward saturation flow or vehicle movements are controlled by coordinated traffic signal systems, the adjustment factor can decrease to 1.5.

In TxDOT Roadway Design Manual, the rule of thumb method has been recommended for calculating the storage length of left-turn lanes by assuming following values for the parameters in Equation (3):

- $K = 2$ (the probability of storing longest expected queue is greater than 0.98). A value of "1.8" may be acceptable on collector streets.
- S is determined based on the percentage of trucks as given in Table 8.

Table 8: Queue Storage Length (per vehicle) Based on Percentage of Trucks
Source: TxDOT Roadway Design Manual (2006)

% of Trucks	S (ft)	S (m)
< 5	25	7.6
5 - 9	30	9.1
10 - 14	35	10.7
15 - 19	40	12.2

- I = 120 seconds (or 2 minutes) as the average vehicle waiting interval at unsignalized intersections

As a result, the rule of thumb method recommended by TxDOT Roadway Design Manual can be written as:

$$L = (V / N_C)(2)(S) \qquad \text{for signalized intersection}$$

and

$$L = (V / 30)(2)(S) \qquad \text{for unsignalized intersection} \tag{4}$$

In addition, a minimum storage length (100 ft) is set up for the intersection with very low left-turn traffic volume. Finally, the storage length for left-turn lane is determined by following equation:

$$L^* = \max(100\,ft, L) \tag{5}$$

where L is determined by Equation (4) based on the traffic and signal control conditions in the intersections.

<u>Comments on Rule of Thumb Methods</u>

Although the rule of thumb method is simple and easy for implementation, it has its disadvantages as well. The method is too simple and does not consider factors that determine the departure rate of the intersection, such as the opposing volumes, the percentage of green phase and so on. Actually, the form of left-turn queue is a procedure determined by both the arrival rate and departure rate of an intersection. Therefore, as pointed by Kikuchi (1993), the left-turn queue length will be overestimated by this method when arrival rate is high and will be underestimated when arrival rate is low. In addition, this method uses a constant factor 2 or 1.8 to ensure that the probability of storing all vehicles is greater than 98% and it did not estimate this probability based on the probability distribution of the arrival vehicles. As a result, it easily overestimates the required storage length of the left-turn lane.

2.2.2 Analytical-Based Methods

To estimate the storage lengths of left-turn lanes, various analytical-based methods, such as regression and queuing-theory-based models, has been developed for estimating the left-turn queue lengths at both signalized and unsignalized intersection. In the following section, these methods for unsignalized and signalized intersections will be introduced individually.

2.2.2.1 Methods for Unsignalized Intersections

2.2.2.1.1 Regression-Based Methods

Basha (1992)

In order to determine the storage lengths of left-turn lanes at unsignalized intersections, Basha (1992) used regression method to establish two relationships: 1) the storage length of left-turn lane as a function of the left-turn volume and the gaps in the opposing traffic; and 2) the amount of acceptable gaps as a function of the opposing traffic. These two relationships can be expressed as

$$Q = f_2(D, G)$$

and
$$G = f_1(V) \tag{6}$$

where:

Q = maximum left-turn lane length, in vehicles

D = left-turn volume, in vehicles per interval

G = total acceptable gap times in opposing traffic in a specific interval, sec

V = opposing traffic volume, in vehicle per interval

The functions f_1 and f_2 were derived by regression analysis and the general forms of these two equations were given in Equation (7).

$$G = f_1(V) \; = \alpha_1^G V^{\beta_1^G}$$

and

$$Q = f_2(D,G)$$
$$= \alpha_1 D + \alpha_2 G^{-1} + \alpha_3 G^{-2} + \alpha_4 G^{-4} \qquad (7)$$

where $\alpha_1^G, \beta_1^G, \alpha_1.....$ are coefficients in these regression equations.

In this study, based on the 15-minute intervals of traffic data collected at two unsignalized intersections during one hour peak time, two different empirical equations with different regression coefficients were derived for these two intersections. The equations for the first intersection were:

$$G = (4{,}716.2846) \times V^{(-0.4005)} \qquad (R^2 = 0.81)$$
$$Q = 0.1369 \times D + 65{,}880 \times G^{-1} - 58{,}800{,}000 \times G^{-2} + 13{,}110{,}000{,}000 \times G^{-3} \qquad (8)$$

And the equations for the second intersections were:

$$G = (5{,}161.7861) \times V^{(-0.3864)} \qquad (R^2 = 0.98)$$
$$Q = 0.1195 \times D + 28{,}200 \times G^{-1} - 34{,}440{,}000 \times G^{-2} - 9{,}894{,}000{,}000 \times G^{-3} \qquad (9)$$

It is found that the developed equations could only produce accurate prediction for the intersection that had been calibrated and could not produce accurate results when being applied to other intersections in other jurisdictions. The low locational transferability of these regression functions suggest that left-turn queue lengths at the unsignalized intersection depend on more factors than just left-turn and opposing through traffic volumes. The author thought one variable, the distance to an adjacent signalized intersection, might need to be included in the regression model. It is because a nearby signalized intersection would create regular and lengthy gaps in the opposing traffic. This will allow more vehicles at the unsignalized intersection to turn left, thereby, resulting in shorter queues. In other words, the impacts of traffic platoon characteristics should be considered in the development of the regression models.

Gard (2001)

Gard (2001) developed a set of regression equations for estimating the maximum queue lengths at unsignalized intersections based on the data collected from the field. Different regression equations were developed for different intersection approaches with different traffic volume conditions. The derived equations were listed in Table 9. For validation purpose, the prediction accuracy of the developed regression equations was compared with four commonly used methodologies. The results showed that, in 49 out of 51 comparisons, the regression equations provided maximum queue-length estimates that were as accurate as or more accurate than the other methodologies.

Table 9: Equations for Estimating the Maximum Left-turn Queue Length
Source: Gard (2001)

Movement	Condition	Equation
Major-street left turn	Approach volume≤100 VPH/PHF	Max. Queue= - 2.042 + 1.167 ln(AppVol) + 0.975×TS
	Approach volume>100 VPH/PHF	Max. Queue=+4.252 – 1.23×Lanes + 0.07996×Speed + 1.412×TS – 374.028/AppVol + 0.00001144×AppVol×ConflVol
Minor-street left turn	Approach volume≤60 VPH/PHF	Max. Queue= + 0.958 + 0.00111×(AppVol)^2 + 0.000333×(ConflVol)
	Approach volume>60 VPH/PHF	Max. Queue=+6.174 – 2.313×TS + 0.03307× Speed - 1201.644/ConflVol + 0.00006549 (AppVol)^2

AppVol = hourly traffic volume divided by peak-hour factor (PHF) for subject movement
ConflVol = hourly traffic volume divided by PHF that conflicts with subject movement (refer to the *Highway Capacity Manual* to identify movements that conflict with subject approach)
TS = a dummy variable with a value of 1 if a traffic signal is located on the major street within one-quarter mile of the subject intersection and 0 otherwise;
Lanes = number of through lanes occupied by conflicting traffic
Speed = posted speed limit on major street (in miles per hour)

Comments on Regression-Based Methods

Regression method is to fit a curve to the data colleted from the field. Without the understanding of the impacts of the underlying influencing factors on the form of left-turn queue, this method can easily cause an “overfit” problem. One symptom of an “overfit” problem is unexplainable coefficients in some regression equations. For example, in Table 9 above, the negative coefficient of variable lanes (number of through lanes occupied by conflicting traffic) in the equation for the major street approach with volume lager than 100 vph/PHF indicates that more conflict lanes will cause less maximum left-turn queue length, which is unreasonable. In addition, the coefficient of variable TS (existing of a nearby signalized intersection within one-quarter mile) in the equation for major street approach with volume large than 100 is positive, while in the equation for minor street approach with a volume less than 60vph/phf are negative. The inconsistent sign of coefficients of TS indicate that the effects of variable TS on maximum left-turn queue length are different in these two scenarios, which is difficult to be explained. Therefore, as found in Basha (1992), the model developed by regression method could only produce accurate prediction for the intersections that are used for model calibration. It could not produce accurate results when being applied to other intersections in other jurisdictions.

2.2.2.1.2 Queuing Theory-Based Methods

Lertworawanich and Elefteriadou (2003)

An *M/G2/1* queuing model was developed by Lertworawanich et al. (2003) for determining the storage lengths of left-turn lanes at unsignalized intersections with a single through lane and a single lane for opposing traffic. The model was developed based on the assumption that the probability of left-turn lane overflow should be less than a given threshold (0.01, 0.02, or 0.05). It also assumed that the arrival of traffic follows a Poisson distribution. The storage lengths of left-turn lanes (in number of passenger cars) were estimated and summarized in three tables for different combinations of volumes and probabilities of left-turn lane overflows. The tables were developed based on the assumption of a critical gap of 4.1 seconds and a follow-up time of 2.2 seconds. Note that critical gap was defined as the minimum gap that all left-turning vehicles were assumed to accept. Follow-up time was defined as the time that elapsed from the time a left-turn vehicle accepted a gap until the next vehicle in the queue started looking for gaps. Table 10 is one of the reference tables in Lertworawanich (2003).

Table 10: Left-Turn Lane Storage Lengths (vehicle units) at Unsignalized Intersections with Single Through and Single Left-Turn Lane, based on 0.05 Probability of Overflow (No Heavy Vehicles)

Source: Lertworawanich and Elefteriadou (2003)

Left-Turn Volume (vph)	Opposing Volume (vph)							
	500	600	700	800	900	1000	1100	1200
100	1	1	1	1	1	1	1	1
200	1	1	1	2	2	2	2	2
300	2	2	2	2	3	3	4	4
400	2	3	3	4	4	5	7	9
500	3	4	4	6	7	11	18	> 50
600	4	5	7	10	18	> 50	> 50	
700	6	9	14	33	> 50	> 50		
800	10	18	> 50	> 50				
900	22	> 50	> 50					
1000	> 50							

Note that the left-turn queue lengths in this study were estimated based on the assumption that no heavy vehicles were present. In case of the presence of heavy vehicles, Lertworawanich and Elefteriadou (2003) recommended to re-apply their proposed methodology by using the following adjusted critical gap and follow-up time:

critical gap (s) = base critical gap (s)
+ [adjustment factor for heavy vehicles (s)][proportion of heavy vehicles]
+ [adjustment factor for grade (s)][percent grade divided by 100]
− adjustment factor for each part of two-stage gap acceptance process (s) −
adjustment factor for intersection geometry (s) (10)

follow-up time (s) = base follow-up time (s)
+ [adjustment factor for heavy vehicles][proportion of heavy vehicles] (11)

<u>*Comments on Queuing Theory-Based Methods*</u>

Queuing theory is a sound method for estimating the left-turn queue lengths at unsignalized intersections. However, the left-turn traffic volumes at the unsignalized intersections are usually very low. This is due to the fact that if the left-turn volume or the cross product of left-turn volume and opposing volume at an intersection is high, the traffic control at this intersection needs to be upgraded to the signalized or even a protected left-turn traffic control. Note that the warrants for the protected left-turn control recommended by Traffic Engineering book (Roess et al., 2000) are:

$$V_{LT} > 200\ veh/h$$

or, $$V_{LT} \times (V_O/N_O) > 50{,}000 \tag{12}$$

Therefore, in Table 10, only the top-left cell are useful for unsignalized intersections and the recommended left-turn storage length are very short (only 1 vehicle storage length). However, in the real world, it is not safe and cost-effective to build such short left-turn lanes. Therefore, a minimum storage length, such as two vehicles (50 ft) recommended by AASHTO Green Book (2001) or four vehicles (100 ft) recommended by TxDOT Roadway Design Manual, should be applied to the intersections where there is very low left-turn volume.

2.2.2.1.3 Based on Vehicle Arrivals in a Given Interval

NDOR Roadway Design Manual

NDOR (Nebraska Department of Roads) Roadway Design Manual states that the storage length of a turn lane should be designed so that the probability that the number of arrival left-turn vehicles during a given time interval exceed the turn lane capacity is less 5% of the time. It provides Equation (13) for determining the storage length of left-turn lane (in feet), based on the assumption that the arrival of left-turn vehicles follows a Poisson distribution.

$$L = X \times 25 \tag{13}$$

where:

L = storage length (ft)

X = the maximum number of vehicles appearing during a given time interval I at a given probability (95% suggested by the manual)

"25" = is considered as the average length of a vehicle

X is a function of average number of vehicles per interval (m) and it can be derived by Equation (12):

$$P(V_I < X) = \frac{e^{-m} m^X}{X!} = 95\% \tag{14}$$

where V_I is the number of arriving left-turn vehicles during the given interval I, which is assumed to follow a Poisson distribution with mean *m*. And *m* can be estimated by the average number of vehicles per interval as follows:

$$m = D \times I \times (1/3600) \tag{15}$$

where:

m = the average number of vehicles per interval

D = design hourly volume (DHV) of vehicles making the turn

I = interval (60 seconds in rural areas, 90 seconds in urban and suburban areas)

According to Equation (14), the following table (Table 11) was given by NDOR Roadway Design Manual for the estimating the value of X based on *m*.

Table 11: Recommended Left-turn Storage Length in Number of Vehicles
Source: NDOR Roadway Design Manual (2005)

Average Number of Vehicles per Interval (m)	95% Probable Maximum Number of Vehicles during the Same Intervals (X)
0.1 to 0.3	2
0.4 to 0.8	3
0.9 to 1.3	4
1.4 to 1.9	5
2.0 to 2.6	6
2.7 to 3.3	7
3.4 to 4.0	8
4.1 to 4.7	9
4.8 to 5.4	10
5.5 to 6.2	11
6.3 to 7.0	12
7.1 to 7.8	13
7.9 to 8.6	14
8.7 to 9.4	15
9.5 to 10.2	16
10.3 to 11.0	17
11.1 to 11.8	18
11.9 to 12.6	19
12.7 to 13.4	20
13.5 to 14.2	21
14.3 to 15.0	22

Comments on Methods Proposed in NDOR Roadway Design Manual

- The advantages: It estimates the length of the left-turn queue based on the probability distribution of the arrival vehicles.
- The disadvantages: Similar as the rule of thumb method, it only considers the traffic arrival rate and does not consider the factors that determine the departure rate of the intersections, such as the opposing volumes, the number of opposing lanes and so on. As a result, the left-turn queue length will be overestimated by this method when the arrival rate is high and will be underestimated when the arrival rate is low.

2.2.2.2 Methods for Signalized Intersections

2.2.2.2.1 Queuing Theory-Based Methods

Oppenlander et al. (1989)

Oppenlander et al. (1989) estimated the design lengths of left-turn lanes at signalized intersections with separated signal phases using queuing theory-based models. Based on the assumption of a Poisson arrival pattern (random distribution) and an exponential service distribution (exponential discharge times), left-turn queue length n (in number of vehicles) can be derived by the following equation:

$$n = (\log P_n - \log(1-\lambda/\mu))/\log(\lambda/\mu) \tag{16}$$

where:

n = number of vehicles in the queue

P_n = probability of n vehicles in the queue

λ = arrival rate, equivalent passenger cars per second (pcps)

μ = service rate, equivalent passenger cars per second (pcps)

and, λ *and* μ can be estimated by following Equations:

$$\lambda = 1.1 \times V/3600 \tag{17}$$

$$\mu = S \times (G/C)/3600 \tag{18}$$

where:

"1.1" = adjustment factor for the equivalence of left-turn vehicles with a separate phase

V = left- turn volume, equivalent passenger cars per hour (pcph)

S = lane saturation flow, equivalent passenger cars per hour of green (pcphg)

G/C = ratio of green time to cycle length (cycle split) for the turning-lane phase

Final results were summarized in a set of reference tables for the 50th-, 85th- and 95th-percentile left-turn queue lengths in feet under different conditions such as turning volumes, ratio of green time to cycle length, and saturation flows. Table 12 is a part of Oppenlander's results.

Table 12: 50th-, 85th- and 95th- Percentile Left-Turn Queue Lengths (feet), with Separate Signal Phase (Saturation Flow of 1500 vph)
Source: Oppenlander et al. (1989)

Left- or Right-Turn Volume	Green Time/Cycle Length Ratio (G/C)																				
	0.05			0.10			0.15			0.20			0.25			0.30			0.35		
	50	85	95	50	85	95	50	85	95	50	85	95	50	85	95	50	85	95	50	85	95
25	-	25	50	-	-	25	-	-	25	-	-	25	-	-	-	-	-	-	-	-	-
50	25	125	225	-	25	50	-	25	25	-	-	25	-	-	25	-	-	25	-	-	25
75				-	50	100	-	25	50	-	25	25	-	-	25	-	-	25	-	-	25
100				25	125	225	-	50	75	-	25	50	-	25	50	-	25	25	-	-	25
125				175	525	825	-	75	125	-	50	75	-	25	50	-	25	50	-	25	25
150							25	125	225	-	50	100	-	25	75	-	25	50	-	25	50
175							75	275	450	25	75	150	-	50	100	-	25	75	-	25	50
200							750	>	>	25	125	225	-	75	125	-	50	75	-	25	75
225										75	225	375	25	100	150	-	50	100	-	50	75
250										175	525	825	25	125	225	-	75	125	-	50	100
275													50	200	325	25	100	175	-	75	125
300													100	350	550	25	125	225	25	75	150
325													350	975	>	50	175	300	25	100	175
350																75	275	450	25	125	225
375																175	525	825	50	175	275
400																750	>	>	75	250	400
425																			125	375	625
450																			275	775	1250
475																			>	>	>

Comments on the Method proposed by Oppenlander et al. (1989)

This queuing model assumes a continuously serving queue and thus cannot properly represent the stop-and-go nature of the operation at signalized intersection. It will significantly underestimate the queue length of left-turn vehicles at signalized intersections because the major part of the queue is built up during the red phase.

2.2.2.2.2 Discreet Time Markov Chain (DTMC)-Based Method

Kikuchi et al. (1993)

Kikuchi et al. (1993) analyzed the required storage lengths of left-turn lanes at signalized intersections by considering two aspects: 1) the probability of left-turn lane overflow and 2) the probability of blockage at the entrance to the turning lane by the queue of vehicles in the adjacent through lane (see Figure 6).

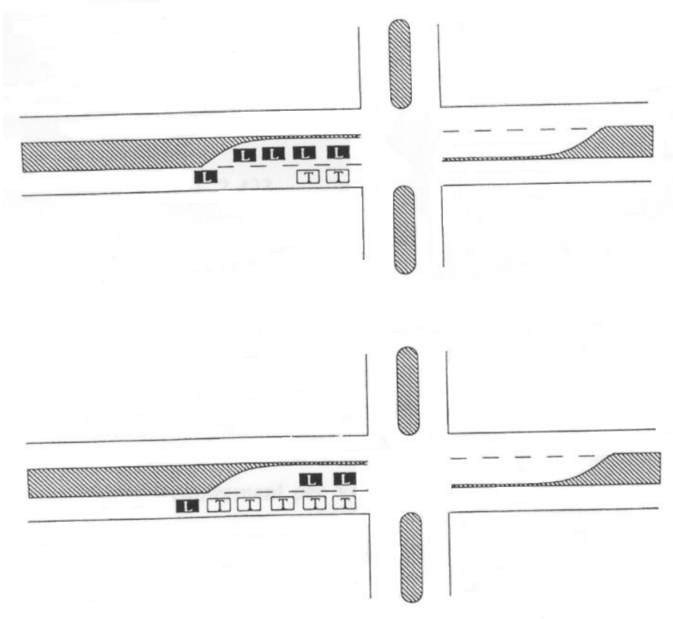


Figure 6: Lane Overflow and Blockage of Lane Entrance at a Signalized Intersection
Source: Kikuchi et al. (1993)

1) From the left-turn lane overflow standpoint, given the threshold probability for overflow *(τ_1=0.02)*, a Markov Chain based model was developed to estimate the maximum left-turn queue length, i.e. *N**. The estimated maximum left-turn queue lengths *N** (in number of vehicles) has been listed in a set of tables (see Table 13 as an example).

Table 13: Recommended Lane Length at Signalized Intersections, Overflow Consideration: Probability of Overflow < 0.02; Number of Vehicles during Permitted Phase = 0/cycle
Source: Kikuchi et al. (1993)

Left-Turn Volume (vph)	Cycle Time (in seconds)															
	90				120				150				180			
	Green Time (sec)				Green Time (sec)				Green Time (sec)				Green Time (sec)			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
50	4	4	3	3	5	4	4	4	7	5	5	5	13	6	6	6
70	5	4	4	4	10	6	5	5	38	7	6	6	-	9	7	7
90	9	5	5	5	-	7	6	6	-	10	8	7	-	22	9	8
110	24	6	6	5	-	10	7	7	-	25	9	8	-	-	13	10
130	-	8	6	6	-	17	8	8	-	-	12	10	-	-	30	12
150	-	10	7	7	-	-	10	9	-	-	22	11	-	-	-	17
170	-	15	8	7	-	-	14	10	-	-	-	14	-	-	-	35
190	-	34	9	8	-	-	24	11	-	-	-	21	-	-	-	-
210	-	-	11	9	-	-	-	13	-	-	-	-	-	-	-	-
230	-	-	14	9	-	-	-	18	-	-	-	-	-	-	-	-
250	-	-	21	10	-	-	-	30	-	-	-	-	-	-	-	-

Note: « - » indicates that the required pocket becomes infinitely long for the combination of parameters.

2) From the left-turn lane blockage standpoint, given the threshold probability for blockage ($\tau_2=0.1$), the required left-turn storage length (in vehicles) was calculated by following equations:

$P_B(N)$ = *Prob {number of through vehicle* $\geq N$ *,*

and the number of left-turning vehicles already in the lane $< N$ *,*

and a left-turn vehicle arrives}

$$N^{**} = \min \{N \mid P_B(N) \leq \tau_2\} \qquad (19)$$

The recommended left-turn lane storage length N^{**} (in number of vehicles) is listed in Table 14 below.

Table 14: Recommended Left-Turn Lane Length in Number of Vehicles, Blockage Consideration: Probability of Blockage < 0.01

Source: Kikuchi et al. (1993)

Left-Turn Volume (vph)	Duration of Through Red = 45 seconds								Duration of Through Red = 60 seconds							
	Through Volume (in vphpl)								Through Volume (in vphpl)							
	500	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**	**500**	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**
50	6	7	8	9	10	11	13	14	9	10	12	13	14	16	17	19
75	7	8	9	10	12	13	14	15	9	11	13	14	16	17	19	20
100	8	9	10	11	12	13	14	16	10	11	13	15	16	18	19	*
125	8	9	10	11	13	14	15	16	10	12	13	15	17	18	20	*
150	8	9	10	12	13	14	15	16	10	12	14	15	17	19	20	*
175	8	9	11	12	13	14	16	17	10	12	14	15	17	19	20	*
200	8	9	11	12	13	14	16	17	10	12	14	15	17	19	*	*
225	8	9	11	12	13	15	16	17	10	12	14	15	17	19	*	*
250	8	9	11	12	13	15	16	17	10	12	14	15	17	19	*	*

Left-Turn Volume (vph)	Duration of Through Red = 75 seconds								Duration of Through Red = 90 seconds							
	Through Volume (in vphpl)								Through Volume (in vphpl)							
	500	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**	**500**	**600**	**700**	**800**	**900**	**1000**	**1100**	**1200**
50	11	13	15	17	18	20	*	*	13	16	18	20	*	*	*	*
75	12	14	16	18	20	*	*	*	14	16	19	*	*	*	*	*
100	12	14	16	18	20	*	*	*	14	17	19	*	*	*	*	*
125	12	14	17	19	20	*	*	*	15	17	20	*	*	*	*	*
150	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*
175	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*
200	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*
225	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*
250	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*

Note: « * » indicates that the required lane length is large. A better way of dealing with the blockage problem may be changing the signal time. In most of these cases the value from this table will not be critical, since required lane length from overflow consideration will be greater.

Finally, by comparing the left-turn lane storage length N* and N** obtained in (1) and (2), the maximum of N* and N** is the final recommended storage length of left-turn lane.

Comments on the Methods Proposed by Kikuchi et al. (1993)

- The advantages: DTMC-based method is an innovative approach to model the left-turn queue length. This method can consider both the random fluctuations in traffic arrival patterns and the effects of signal control on left-turn queue lengths.
- The disadvantages: This model did not separate the red-phase queue (the queue build up during the red signal phase) and the leftover queue (the queue carried over from previous cycles), and thus the effects of the red phase on queue length cannot be well considered.

The evaluation results of this study showed that the model resulted in a relatively shorter queue length in comparison with other methods.

2.2.2.2.3 Based on Vehicle Arrivals in the Red Phase

Kikuchi et al. (2004)

Kikuchi et al. (2004) developed a method to determine the length of left-turn lanes at signalized intersections with dual left-turn lanes (DLTL) and a single through lane. Similar to Kikuchi et al. (1993), the lengths of DLTL are analyzed based on two conditions: 1) high probability of no left-turn overflow (=95%/99%), and 2) high probability of no blockage of the entrance of the left-turn lane by the queue of through vehicles (=95%/99%). According to these two conditions, probability formulation was developed for estimating the required queue lengths based on two assumptions: (1) all queues (both in left-turn lanes and through lanes) are built up during red-phase (red-phase queue), and (2) all vehicles will clear the intersection by the end of green phase (no leftover queue). A sample result of this study is given in Table 15.

Table 15: Computed Length of left-turn lane for 16 Cases of Left-turn (LT) Volume and Through (TH) Volume Combinations (α = 0.95/0.99)

Source: Kikuchi et al. (2004)

		LT per Red Phase			
		8	**14**	**19**	**25**
TH per Red Phase	**8**	13/15	13/16	14/17	17/19
	14	20/23	21/24	21/24	21/24
	19	25/29	26/30	26/30	27/30
	25	32/36	33/37	33/37	33/37

Comments on the Method Proposed by Kikuchi et al. (2004)

- The advantages: It is constructive to estimate the length of left-turn lanes by considering both the left-turn lane overflow and blockage problems
- The disadvantages: (1) it assumes that the entire left-turn queue is built up during the red phase, and that all vehicles can clear the intersection by the end of the green phase (thus, no queue carryover). However, the results of our field survey show that, even for the

intersections with left-turn *v/c* ratios of less than 1 (between 60% and 80%), the left-turn queue carryover frequently occurred (in about 20% of the cycles observed). Therefore, this assumption of no queue carried over is unreasonable and it will cause left-turn queue lengths to be underestimated. (2) In this paper, the left-turn lane overflow and blockage problem are treated equally. Actually, left-turn overflow problem has more serious impacts on both the efficiency and safety of left-turn operation. Therefore, the overflow problem should be controlled with lower tolerable probability as in Kikuchi et al. (1993).

2.2.3 Simulation-Based Methods

Because of the complexity in modeling the operation of a signalized intersection, traffic simulation is a practical means to estimate the queue length of left-turn vehicles at a signalized intersection. In literature, simulation-based methods have been used by several studies for estimating the storage length of left-turn lanes.

Oppenlander et al. (1994, 1996, 1999 and 2002)

Oppenlander et al. (1994, 1996, 1999 and 2002) conducted a series of studies on determining the storage length of left-turn lanes at signalized intersections using stochastic (Monte Carlo) Simulation models. In the Oppenlander et al. (1994, 1996), a simulation model was developed based on the assumption that the arrivals of vehicles at an intersection follow a Poisson distribution and the departures of vehicles from intersections follow a triangle distribution. Later, in Oppenlander et al. (1999), they conducted another simulation-based study to estimate the left-turn queue length at the intersections with uniform arrival left-turning vehicles. Since both uniform and random arrival patterns represent the ideal/extreme arrival conditions, interpolation method was recommended for the intersection with intermediate conditions in the real world. In the Oppenlander et al. (2002), they conducted a simulation based study for signalized intersections without a separate signal phase (permitted left-turn operation with a single lane for opposing traffic). This study was not only for determining the storage length of left-turn lanes for permitted left-turn movements, but also led to calculate the storage of left-turn lanes at the intersections with protected-permitted left-turn controls in conjunction with the results from Oppenlander et al. (1994, 1996, 1999). In addition to the parameters used in their previous studies, the variable of opposing volume was also required in the simulation. The opposing volume was defined as any movement or combination of movements that conflicted with vehicles in the left-turn lane. For both left-turn and opposing traffics, Poisson arrival pattern and triangular distribution departure patterns were assumed.

In the Oppenlander et al. (1994, 1996, 1999 and 2002), the results were summarized in a series of reference tables for the 50th-, 85th- and 95th-percentile left-turn queue lengths (in vehicle units) under different conditions (turning volumes, cycle length, left-turn green time, and opposing volumes). Following are the sample tables of Oppenlander's results. Table 16 shows storage length at signalized intersections with separate signal phase (cycle length of 60 sec) and random Poisson arrival. Table 17 represents storage length at signalized intersections without separate signal phase (cycle length=60 sec, green time=30 sec) and random Poisson arrival. In the Oppenlander et al.'s studies, it also suggested that, in case of actuated signal operations, maximum green time by approaches should be used for these reference tables.

Table 16: 50th-, 85th- and 95th- Percentile Storage Lengths (vehicle units), with Separate Signal Phase (Cycle Length=60 sec) and different Effective Green Times

Source: Oppenlander et al. (1996)

Lane Volume (vph)	Percentile Value	Effective Green Time (sec)						
		10	15	20	25	30	35	40
	50th	1	0	0	0	0	0	0
50	85th	2	1	1	1	1	1	1
	95th	2	2	2	2	2	2	1
	50th	1	1	1	1	1	1	0
100	85th	3	2	2	2	2	2	1
	95th	4	3	3	3	3	2	2
	50th	2	2	2	1	1	1	1
150	85th	4	3	3	3	2	2	2
	95th	6	4	4	4	3	3	3
	50th	4	2	2	2	2	1	1
200	85th	9	4	4	4	3	3	2
	95th	13	5	5	4	4	3	3
	50th	∞	3	3	2	2	2	1
250	85th	∞	6	5	4	4	3	3
	95th	∞	8	6	5	5	4	4
	50th		5	3	3	2	2	2
300	85th		10	6	5	4	4	3
	95th		14	7	6	5	5	4
	50th		32	4	3	3	2	2
350	85th		∞	7	5	5	4	3
	95th		∞	9	7	6	5	5
	50th		∞	5	4	3	3	2
400	85th		∞	9	6	5	5	4
	95th		∞	12	8	7	6	5
	50th			11	5	4	3	2
450	85th			21	7	6	5	4
	95th			27	10	8	6	5
	50th			∞	6	4	3	3
500	85th			∞	10	7	6	5
	95th			∞	13	9	7	6
	50th				9	5	4	3
550	85th				16	8	6	5
	95th				23	10	8	6
	50th				∞	6	4	3
600	85th				∞	10	7	6
	95th				∞	13	9	7
	50th					8	5	4
650	85th					15	8	6
	95th					19	10	7
	50th					19	6	4
700	85th					43	9	6
	95th					55	12	8
	50th					∞	7	4
750	85th					∞	13	7
	95th					∞	19	10
	50th						12	5
800	85th						25	9
	95th						33	12

Table 17: Left-Turn Lane 50th-, 85th- and 95th- Percentile Storage Lengths (vehicle units), without Separate Signal Phase (Cycle Length=60 sec, Green Time=30 sec)

Source: Oppenlander et al. (2002)

Opposing Volume (vph)	Percentile Value	Left-Turn Volume (vph)									
		50	100	150	200	250	300	350	400	450	500
100	50th	0	1	1	2	2	3	3	4	5	8
	85th	1	2	2	3	4	5	5	7	10	18
	95th	2	3	3	4	5	6	7	9	17	25
200	50th	0	1	1	2	2	3	5	12	∞	∞
	85th	1	2	2	3	4	6	10	23	∞	∞
	95th	2	3	4	4	6	9	14	30	∞	∞
300	50th	0	1	1	2	3	7	∞	∞		
	85th	1	2	3	4	7	19	∞	∞		
	95th	2	3	4	6	11	26	∞	∞		
400	50th	0	1	2	4	38	∞				
	85th	1	2	4	12	∞	∞				
	95th	2	4	7	17	∞	∞				
500	50th	1	2	15	∞	∞					
	85th	2	5	29	∞	∞					
	95th	3	11	35	∞	∞					
600	50th	1	∞	∞							
	85th	4	∞	∞							
	95th	6	∞	∞							
700	50th	∞									
	85th	∞									
	95th	∞									

Lakkundi et al. (2004)

In Lakkundi et al. (2004), in addition to the study for left-turn lane warrants, a preliminary study for left-turn lane lengths estimation was also conducted. Lakkundi et al. (2004) investigated the probability of left-turn lane overflows for varying left-turn bay lengths using an event-based simulation program (LTGAP) given that the purpose of installing a left-turn lane was to prevent left-turn lane overflows. Recommended left-turn lane lengths were provided in the form of a graph at a given traffic volume, geometry, and intersection traffic control conditions. In other words, they plotted the probability of left-turn bay overflow against left-turn bay length. At each left-turn bay length, 100 simulation runs were made to obtain an average left-turn bay overflow probability. Left-turn bay length was evaluated from 0 to 1,200 feet in every 50 feet for signalized intersections and was varied from 0 to 500 feet in every 50 feet for unsignalized intersections. The developed simulation based method for left-turn lane length analysis was applied to both signalized and unsignalized intersections. Two sample analysis results were presented in Figures 7 and 8. Note that the measure of effectiveness (MOE) used for analyzing the signalized intersection was the “percentage of time the left-turn lane overflow occurred” and the MOE for unsignalized intersection was the “percentage of time the through vehicles were blocked by left-turn vehicles.” This type of analysis could be done for any volume and speed combinations desired by the user.

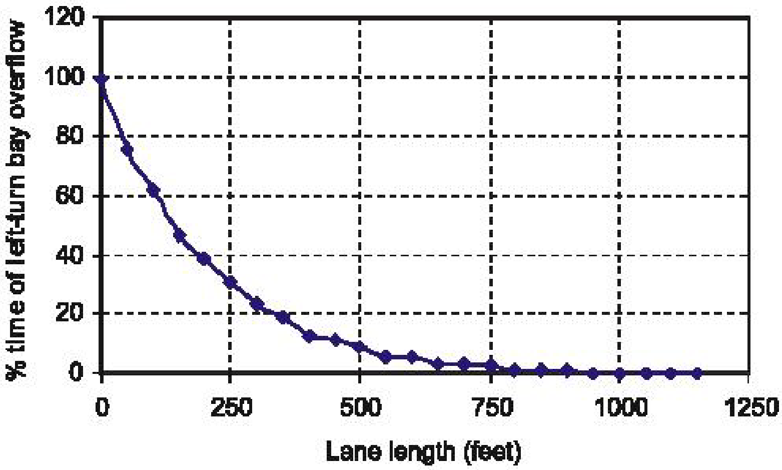


Figure 7: Left-Turn Lane Length Analysis at Signalized Intersectionfor Approach and Opposing Volumes of 500 vph, Cycle Length of 60 seconds, G/C Ratio of 0.5, and 30% Left-Turn Vehicles
Source: Lakkundi et al. (2004)

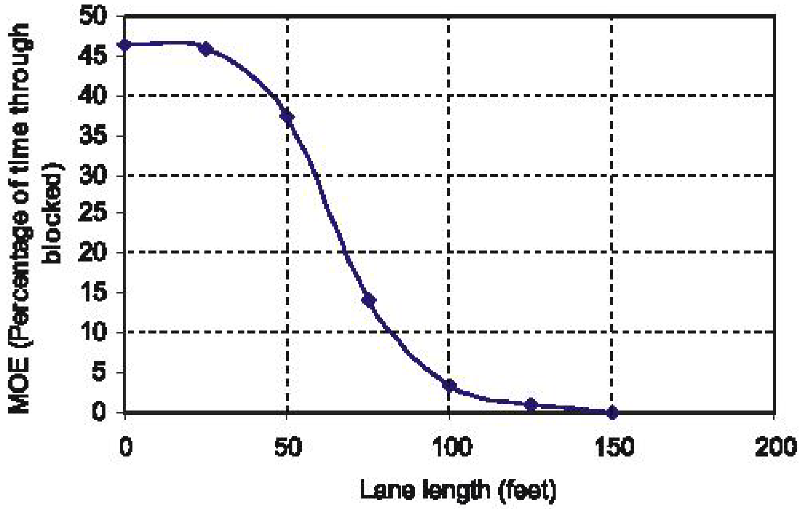


Figure 8: Left-Turn Lane Length Analysis at Unsignalized Intersection for Approach Volume of 800 vph, Opposing Volume of 50 vph, and 20% Left-Turn Vehicles

Source: Lakkundi et al. (2004)

<u>*Comments on Simulation-Based Methods*</u>

The major limitations of the simulation approach are that the simulation model must be carefully calibrated to be able to duplicate the real-world traffic conditions, and that it is valid only for a specific set of roadway-traffic conditions.

2.2.4 Summarization of Different Methods for Determination of Left-Turn Storage Length

Based on the discussion above, Table 18 summarizes and compares different types of methods for the determination of left-turn storage length.

Table 18: Summarization of Different Methods for Determination of Left-Turn Storage Length

<table>
<tr><th colspan="3">Existing Methods by Categories</th><th>Reference</th><th>Major Results</th></tr>
<tr><td colspan="3">Rule of Thumb Methods</td><td>• TxDOT Roadway Design Manual
• NCHRP Report 279
• NCHRP Report 348</td><td>• Equations (4) & (5)</td></tr>
<tr><td rowspan="6">Analytical-Based Methods</td><td rowspan="3">Unsignalized Intersections</td><td>Regression based</td><td>• Basha (1992)
• Gard (2001)</td><td>• Equations (8) and (9)
• Table 9</td></tr>
<tr><td>Queuing theory based</td><td>• Lertworawanich et al. (2003)</td><td>• Table 10</td></tr>
<tr><td>Vehicle arrivals in a given interval</td><td>• NDOR Roadway Design Manual (2005)</td><td>• Equations (13) to (15)
• Table 11</td></tr>
<tr><td rowspan="3">Signalized Intersections</td><td>Queuing theory based</td><td>• Oppenlander at al (1989)</td><td>• Equations (16) to (18)
• Table 12</td></tr>
<tr><td>DTMC based</td><td>• Kikuchi et al.(1993)</td><td>• Tables 13 and 13</td></tr>
<tr><td>Vehicle arrivals in the red phase</td><td>• Kikuchi et al.(2004)</td><td>• Table 14</td></tr>
<tr><td colspan="3">Simulation-Based Methods</td><td>• Oppenlander et al. (1994, 1996, 1999 and 2002)
• Lakkundi et al. (2004)</td><td>• Tables 15 and 16
• Figures 7 and 8</td></tr>
</table>

CHAPTER 3

SURVEY TO IDENTIFY MAJOR PARAMETERS

A survey is conduced to the field engineers to identify and prioritize the important parameters and variables that are essential to the determination of deceleration and storage length requirements for left-turn lanes. This survey will also seek information on criteria for multiple left-turn lane installation. According to these purposes, the research team develops a survey instrument, which is attached in Appendix A.

3.1 Survey Design

The survey includes two parts. The first part is to identify the priorities of the parameters in the determination of left-turn lane deceleration and storage lengths, and the development of warrants on multiple left-turn lanes. These parameters include the following five main categories:

- Traffic condition
    - Left-turn volume
    - Opposing traffic volume
    - Through traffic volume
    - Vehicle types
    - Intersection congestion level
- Geometric condition
    - Grade
    - Number of left-turn lanes
    - Number of shared lanes for left turn

   - Number of through lanes
- Driving behavior
   - Average speed at the entrance of left-turn lane
   - Average speed on through lane
   - Deceleration and acceleration rate on left-turn lane
- Traffic control
   - Signalized and unsignalized
   - Pretimed and actuated
   - Permitted and protected
   - Signal cycle length
   - Phase structure and length
- Traffic safety
   - Historical accident rate
   - Historical rate of left-turn accident

At the end of this part of the survey, it is requested to identify other parameters, which are considered important by respondents, and to prioritize them.

The second part of the survey consists of some general questions on left-turn lane design and operation.

3.2 Survey Results

The survey was conducted via email in January 2006. The survey mailing list was provided by TxDOT Project Director. This list included TxDOT traffic engineers, district engineers and Austin area chapter of TexITE. Finally, 26 completed survey responses were received. Most of the responses were received by e-mail and some by fax. Based on the received survey responses, the research team analyzed the survey results, which are summarized as follows.

3.2.1 Priority of Parameters

The candidate parameters are prioritized based on their average scores. Each parameter listed in the survey is given numbers from “1” to “5” with “5” indicating the highest priority and

"1" indicating the lowest priority. The respondents are advised to circle a number that represented the importance of the parameters in left-turn lane design according to their judgments.

3.2.1.1 Left-Turn Lane Deceleration and Storage Lengths

By reviewing the responses to the survey, the priorities of the parameters in the determination of left-turn lane deceleration and storage lengths in the five categories are compared.

Traffic Condition Category

Traffic condition category includes five parameters: left-turn volume, opposing traffic volume, through traffic volume, vehicle type, and intersection congestion level. "Left-turn volume" was recognized by the respondents as the most important parameter in this category. Based on the survey results, the parameters in this category are ranked according to their scores as follows (see Figure 9 for detailed scores):

1. Left-turn volume
2. Opposing traffic volume, Intersection congestion level
3. Vehicle types
4. Through traffic volume

Geometric Condition Category

Grade, number of left-turn lanes, number of shared lanes for left turn, and number of through lanes are the four parameters in the geometric condition category. Respondents identified "number of left-turn lanes" as the most important parameter in this category. Based on the survey results, the parameters in this category are ranked according to their priority levels as follows (see Figure 9 for detailed scores):

1. Number of left-turn lanes
2. Number of shared lanes for left turn
3. Number of through lanes
4. Grade

Driving Behavior Category

The scores of the parameters in the driving behavior category are shown in Figure 9. The most important parameter is known as “average speed at the entrance of left-turn lane.” The ranks of the parameters according to the survey results are listed as follows:

1. Average speed at the entrance of left-turn lane
2. Deceleration and acceleration rate on left-turn lane
3. Average speed on through lane

Traffic Control Category

The scores of the parameters in traffic control category are shown in Figure 9. According to this result, “Signalized and unsignalized” and “permitted and protected” are the two most important parameters. Based on the survey results, the parameters in this category are ranked according to their priority levels as follows:

1. Signalized and unsignalized
2. Permitted and protected
3. Pretimed and actuated, Signal cycle length
4. Phase structure and length

Traffic Safety Category

Historical accident rate and historical rate of left-turn accident are two parameters in the traffic safety category. “Historical rate of left-turn accident” was identified as more important than the other parameter (see Figure 9).

Other Parameters

The respondents were asked to identify other parameters related to left-turn deceleration and storage length. These parameters and their scores are listed in Table 19.

Table 19: Score of Other Parameters

Parameters Name	Score
Public feedback	5
Posted speed limit	5
Deceleration rates	4
Driveways locations next to the intersect	1
Appropriate signing	1

Figure 9 shows the results of the survey for all the parameters regarding left-turn lane deceleration and storage lengths.

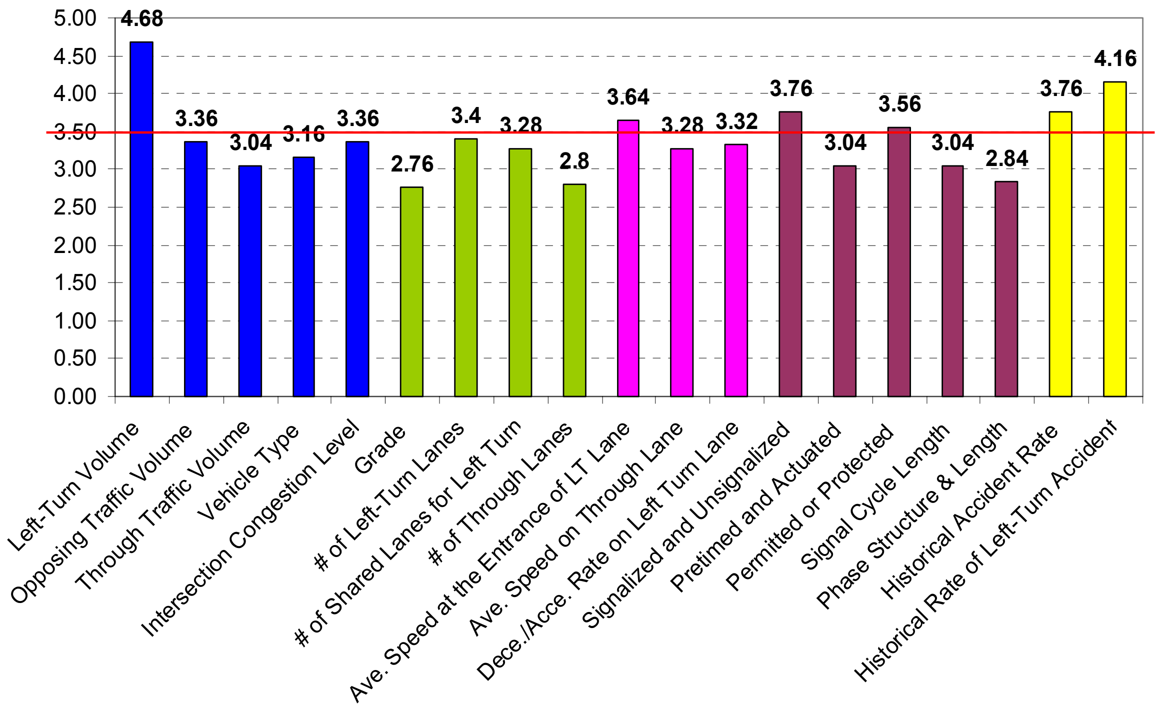


Figure 9: Parameters for Left-Turn Deceleration and Storage Lengths

By using homogeneous subset Tukey statistic test, these parameters are grouped into three subsets according to their statistical ranks (see Table 20).

Table 20: Statistical Ranks of Parameters for Left-Turn Deceleration and Storage Lengths

Homogeneous Subset Tukey Test	Factors	Average Score
1st Rank	Left-turn volume	4.68
	Historical rate of left-turn accident	4.16
	Signalized and unsignalized	3.76
	Historical accident rate	3.76
	Average speed at the entrance of left-turn lane	3.64
	Permitted or protected	3.56
2nd Rank	Number of left-turn lanes	3.40
	Opposing traffic volume	3.36
	Intersection congestion level	3.36
	Deceleration/acceleration rate on left-turn lane	3.32
	Number of shared lanes for left turn	3.28
	Ave. speed on through lane	3.28
	Vehicle types	3.16
	Through traffic volume	3.04
	Pre-timed and actuated	3.04
	Signal cycle length	3.04
3rd Rank	Phase structure and length	2.84
	Number of through lanes	2.80
	Grade	2.76

Conclusion

The analysis of the data revealed that the priorities of all 19 parameters are very close (2.76 ~ 4.68). Thus, it had better to consider as many parameters as possible in the model development. Also, the survey identified that the "left-turn traffic volume" and "left-turn related accident rate" are the highest priority level parameters for left-turn lane deceleration and storage length. Therefore, more weight will be given to these parameters in the data collection and model development tasks later on.

3.2.1.2 Warrants for Multiple Left-Turn Lanes

In this survey, the same sets of parameters are evaluated for their priorities in the development of warrants for multiple left-turn lanes. The following is the survey results of these parameters in the five categories.

Traffic Condition Category

Traffic condition category included five parameters: left-turn volume, opposing traffic volume, through traffic volume, vehicle type, and intersection congestion level. "Left-turn volume" was recognized by the respondents as the most important parameter in this category. Based on the survey results, the parameters in this category are ranked according to their priority levels as follows (see detail scores in Figure 10):

1. Left-turn volume
2. Opposing traffic volume
3. Intersection congestion level
4. Through traffic volume, Vehicle types

Geometric Condition Category

Grade, number of left-turn lanes, number of shared lanes for left turn, and number of through lanes are the four parameters in the geometric condition category. Respondents identified "number of left-turn lanes" and "number of shared lanes for left turn" as the two most important parameters in this category. Based on the survey results, the parameters in this category are ranked according to their priority levels as follows (see Figure 10 for detail scores):

1. Number of left-turn lanes
2. Number of shared lanes for left turn
3. Number of through lanes
4. Grade

Driving Behavior Category

The scores of the parameters in the driving behavior category are shown in Figure 10. The most important parameter is "average speed at the entrance of left-turn lane". The ranks of the parameters based on the survey results are given as follows.

1. Average speed at the entrance of left-turn lane
2. Average speed on through lane, and Deceleration and acceleration rate on left-turn lane

Traffic Control Category

The scores of the parameters in traffic control category are shown in Figure 10. “Signalized and unsignalized” and “permitted and protected” are the two most important parameters among other parameters. The ranks of the parameters based on the survey results are given as follows:

1. Signalized and unsignalized
2. Permitted and protected
3. Signal cycle length
4. Phase structure and length
5. Pretimed and actuated

Traffic Safety Category

Historical accident rate and historical rate of left-turn accident are two parameters in the traffic safety category. “Historical rate of left-turn accident” are more important than the other parameter according to the survey results (see Figure 10).

Other Parameters

Other parameters related to warrants for multiple left-turn lanes have been identified in the survey forms. Those parameters and their scores are listed in Table 21.

Table 21: Other Parameters on Warrants for Multiple Left-Turn Lane

Parameters Name	Score
Public Feedback	5
Posted Speed Limit	5
Deceleration Rates	4

Figure 10 shows the results of the survey for all the parameters related to multiple left-turn lane warrants.

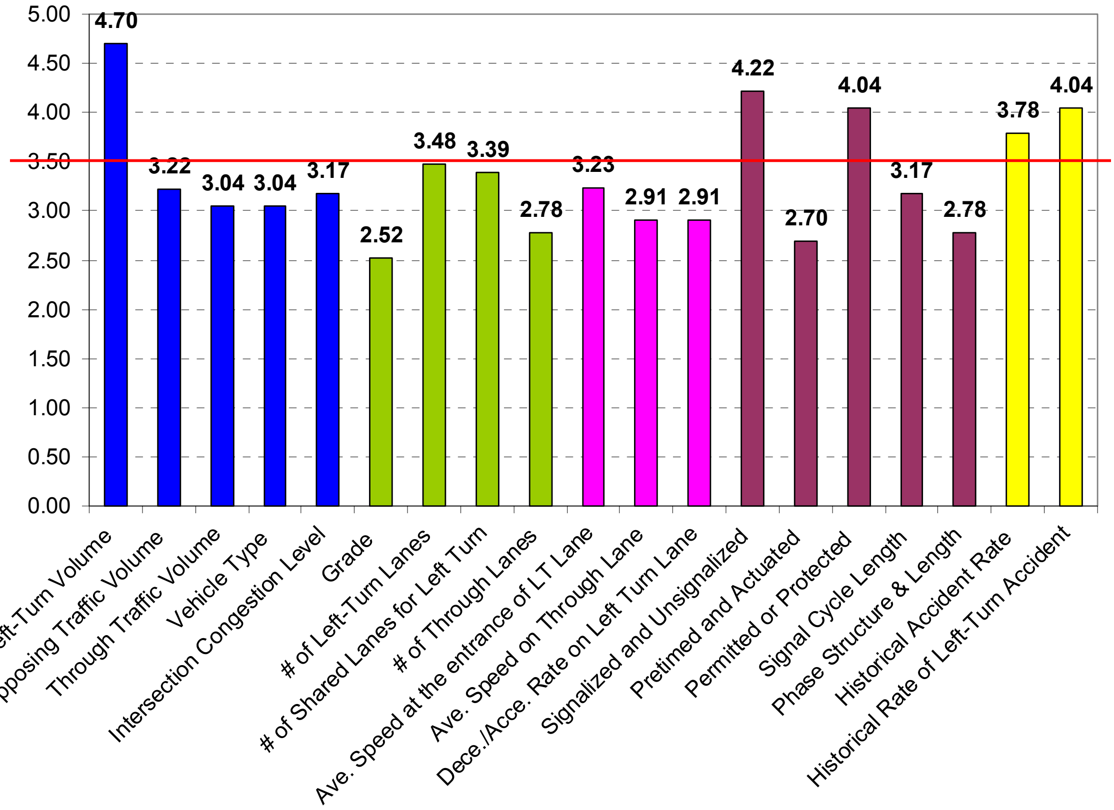


Figure 10: Parameters for Multiple Left-Turn Lane Warrant

Statistical ranks of these parameters are derived by using homogeneous subset Tukey test. Table 22 shows the five different ranks of these parameters according to their average scores.

Table 22: Statistical Ranks of Parameters for Multiple Left-Turn Lane Warrant

Homogeneous Subset Tukey Test	Factors	Average Score
1st Rank	Left-turn volume	4.70
	Signalized and unsignalized	4.22
	Permitted or protected	4.04
	Historical rate of left-turn accident	4.04
	Historical accident rate	3.78
	Number of left-turn lanes	3.48
2nd Rank	Number of shared lanes for left turn	3.39
	Average speed at the entrance of left-turn lane	3.23
	Opposing traffic volume	3.22
	Intersection congestion level	3.17
	Signal cycle length	3.17
	Through traffic volume	3.04
	Vehicle types	3.04
3rd Rank	Average speed on through lane	2.91
	Deceleration/acceleration rate on left-turn lane	2.91
4th Rank	Number of through lanes	2.78
	Phase structure and length	2.78
	Pre-timed and actuated	2.70
5th Rank	Grade	2.52

Conclusion

Based on the above survey results, the priorities of all 19 parameters are very close (2.52 ~ 4.70). Therefore, it's better to consider as many parameters as possible in the development of warrants for multiple left-turn lanes. Among these parameters, the survey identified the "left-turn related accident rate" and "intersection signal control types" as the highest priority parameters.

3.2.2 General Questions about Left-Turn Lane Design

In the second part of the survey form, the following general questions were asked about the left-turn lane design and operation:

Question 1- What are the most critical issues in the design and operation of left-turn lanes?

Question 2- What are the most important criteria for evaluating the design of a left-turn lane?

Question 3- What is the existing practice on the determination of deceleration and storage length requirements in your agency?

Question 4- What are the existing warrants for multiple left-turn lanes in your agency?

Question 5- Are there any good methods/experiences on the determination of deceleration and storage length requirements that can be shared with us?

Question 6- Are there any good methods/experiences on developing the warrants for multiple left-turn lanes that can be shared with us?

Question 7- Additional Comments

The answers to these questions are reviewed and summarized in sections 3.2.2.1 to 3.2.2.7.

3.2.2.1 Critical Issues in Design and Operation of Left-Turn Lanes

The issues identified by the respondents are listed in Table 23. Among all issues, "volume," "space for installing left-turn lanes," and "storage length" are identified as the most critical issues.

Table 23: Critical Issues in Design and Operation of Left-Turn Lanes

Issues		Percentage
Traffic Flow	Volume (left-Turn and through)	34.62%
	Speed	26.98%
	Enough gap	11.45%
Traffic Control	Protected or permitted	19.90%
	Signal phasing	15.26%
Geometric Conditions	Right of way (enough space for installation)	34.62%
	Storage length	34.62%
	Deceleration length	15.26%
	Taper length	15.26%
	Sight distance and visibility	15.26%
	Number of left-turn lanes	11.45%
Safety, Accident		11.45%
Intersection Capacity		7.63%
Vehicle Type		3.82%
Future Development		3.82%
Funding		3.82%

The following are some important comments from the respondents:

- The historical deceleration rates are probably too liberal for most agencies.
- During the peak periods when traffic speeds are typically lower and the traffic volume are heavier than non-peak periods, the deceleration distances could be shorter, while at the same time a longer queue storage length is required.
- It is recommended that roadway functional values (roadway and cross street classifications), instead of future traffic volumes, be used for determining the length of left turn lane.

3.2.2.2 Important Criteria for Evaluating the Design of Left-Turn Lanes

Half of the respondents recognized “volume” as an important criteria for evaluating the design of left-turn lanes. Other identified criteria are listed in Table 24.

Table 24: Important Criteria for Evaluating the Design of Left-Turn Lanes

Criteria	Percentage
Volume (left-turn, through and opposing)	50.00%
Speed	34.62%
Enough storage length	26.92%
Safety (accident rates)	23.08%
Vehicle types	11.54%
Queue length, left-turn vehicle waiting time and intersection delay	8%
Right of way (enough space for installation turn lane in the future)	8%

Following are some important comments from the respondents:

- On the high speed highways (speed more than 45 mph), sufficient left-turn length that prevent vehicles from being queued out of the bay is critical under any circumstances due to the possibility for rear-end collisions. But in other locations (speed less than 45 mph), the procedure for determining the length of left-turn lane should be the following:
  a) Focus on signal phasing design (cycle length, phasing, phase, etc.).
  b) Based on signal deign, figure out how much storage is required.
  c) Try to get as much deceleration distance to go with the storage as practically allowed.

3.2.2.3 Existing Practices on Determination of Deceleration and Storage Length

Most of the survey respondents indicated that TxDOT Roadway Design Manual provides guidelines on the determination of deceleration and storage length. Also AASHTO Green Book and “Future Estimated Storage Requirements" were used as the existing guidelines.

3.2.2.4 Existing Warrants for Multiple Left-Turn Lanes

The respondents were asked to give any existing warrants for multiple left-turn lanes. The results confirmed that there are few existing warrants as shown in Table 25.

Table 25: Existing Warrants for Multiple Left-Turn Lanes

Warrants	Frequency
No warrants at all	6
Warrants from TxDOT Roadway Design Manual	2
Warrants from AASHTO	1
Use CORSIM or SYNCHRO to compare delay with and without extra lane(s)	1
Rule of thumb: Left-turn volume is over 200 vph	1

“Left-turn volume” is the criterion used by most of the respondents in the determination of multiple left-turn lanes installation. Other criteria are listed in Table 26.

Table 26: Criteria Using in Warrants for Multiple Left-Turn Lanes

Parameters	Frequencies
Left-turn volume	7
Total approach volume	1
Peak hour left-turn volume at signalized intersections	2
Design volume	1
Right of way	3
Geometry	4

3.2.2.5 Other Methods/Experiences on Determination of Deceleration and Storage Length

The respondents gave some comments and experiences on the determination of left-turn deceleration and storage length:

- Existing taper guidelines provided by TxDOT Roadway Design Manual seems to yield lengths that are too short in the field.
- Existing required deceleration lengths recommended by TxDOT Roadway Design Manual seems too long.
- Not using short tapers for high speed locations.
- Using a rule of thumb (500 ft) for new construction, longer turn lanes.

- Using advance signing.
- Using two-way left-turn lane "TWLTL" (since longer turn lanes may deny access to driveways from opposing left turn traffic).
- We should not worry too much about setting the storage requirements precisely on the numeric projected queue lengths since the volume projections will not be 100% accurate.

3.2.2.6 Other Methods/Experiences on Developing Warrants for Multiple Left-Turn Lanes

The respondents suggested some other methods or experiences on developing warrants for multiple left-turn lanes:

- Using Colorado DOT and TTI guidelines
- Adding multiple left turn bays wherever the room exists to build
- Incorporating AASHTO deceleration rates
- One of the respondents believed that the 300 vph left-turn volume criterion works well.

3.2.2.7 Other Comments

At the end of the survey forms, respondents made following comments:

- Signal timing and phasing can be changed to accommodate a left turn lane/lanes.
- Guidelines on determining the length of the broken stripe at the end of solid stripe of the storage length are needed.

3.2.2.8 Summary for the General Questions Results

Most of the survey respondents indicated that the guidelines provided by TxDOT Roadway Design Manual were used for the determination of deceleration and storage length and there were few existing warrants for multiple left turn lanes.

Critical issues in the design and operation of left-turn lanes were identified as follows:

- Right of way (not enough space for installation or for future development).
- The exiting methods yield short taper lengths and longer deceleration lengths.
- Long left-turn lanes may block the access to driveways for the opposing left turn traffic.

In addition, the following constructive suggestions were made:

- During the peak hour, due to relatively low traffic speed, the deceleration length could be shorter.

- Using functional classifications of the roadway and cross street instead of future traffic volumes to determine the length of left turn lane

CHAPTER 4

DATA COLLECTION

Data collection is one of the most important tasks of the study. The results of this task will be used to develop and validate the methodology for determining left-turn storage length. In addition, the collected information will be used for analyzing the safety benefit of extending the length of left-turn lanes in Chapter 7. Before initiating data collection, a field data collection plan was developed and the candidate intersections for this study were selected.

4.1 Data Collection Plan

The design of data collection plan is to make sure that all of the data needed to develop the model would be collected, and the requirements for collecting the field data would be satisfied. Based on the parameters identified in the literature review and the results of survey, a detailed field data collection plan was developed. The data collection plan specifies the following:

- selected intersections,
- types and quantities of the data needed for each intersection,
- time periods of the day and duration for data collection,
- labors and equipments,
- methods of data collection and the data collection devices to be used , and
- detailed schedule of the data collection activities.

Basically, the data to be collected for each intersection can be categorized into two types: dynamic data and static data. The dynamic data are traffic parameters associated with traffic information. The static data are those associated with geometric design, signal timing information, and accident histories at the selected intersections. Table 27 shows the detailed list of data in each category.

Table 27: Detailed List of Data to Be Collected

<table>
<tr><th>Category</th><th colspan="2">Parameters</th></tr>
<tr><td>Dynamic</td><td>Traffic</td><td>• Approaching speed of vehicles
• Left-turn volume
• Through traffic volume
• Percentage of heavy vehicles
• Queue length in subject left-turn lane, in each cycle
• Queue length in adjacent through lane, in each cycle
• Headway
• Start-up time
• Cycle failure (left-turn queue carryover problem)</td></tr>
<tr><td rowspan="3">Static</td><td>Traffic and Geometry</td><td>• Posted speed limits on each street
• Intersection layout
• Number of lanes in all approaches (left-turn lanes, through traffic lanes, right-turn lanes, shared lanes)
• Type of subject left-turn lane (exclusive single, exclusive double, two-way left-turn lane, one lane exclusive and one lane shared with through traffic)
• Location of installed camera in intersection
• Length of existing left-turn lane in the subject approach
• Distance between driveways (distances from upstream and also downstream intersections)</td></tr>
<tr><td>Signal Timing</td><td>• Signal planning (schedule)
• Cycle length
• Splits
• Left-turn phase type</td></tr>
<tr><td>Historical</td><td>• Accident information</td></tr>
</table>

Selection of Cities/Intersections

To investigate the impacts of the different influencing factors on left turn lane design, the selected intersections should cover a broad range of areas, including intersections with different traffic flow, traffic control and geometric conditions. Specifically, the following factors are considered in the study sites selection:

- Traffic control types: unsignalized and signalized (protected, permitted, and protected-permitted left turn),
- Environmental settings: urban and rural,
- Geometric conditions: number of left turn lanes and number of through lanes, and
- Traffic conditions: low/high volume and low/high speed.

According to the contact information provided by the project monitoring committee, different traffic management centers in the city of McAllen, city of Austin, Laredo district, El Paso district, and Harris County were contacted. For each agency, the following questions were asked to obtain the basic information about the intersections equipped with traffic monitoring cameras:

- How many intersections are installed with traffic monitoring cameras?
- How many cameras are installed in each intersection?
- Are there any other surveillance systems (such as loop detectors) installed at those intersections?
- Is there historical accident information available?
- What is the traffic control type for those intersections (unsignalized or signalized (protected, permitted, or protected-permitted left turn).
- What is the environmental setting for those intersections (urban, rural, etc.)?
- What is the geometric condition for those intersections (number of left-turn lanes and number of through lanes)?
- What is the traffic condition in those intersections (low/high volume (traffic flow rates) and low/high speed)?

Based on the information received from those districts, Austin and Houston districts were selected for data collection since there are more intersections with traffic monitoring cameras in those two districts than in other districts. For example, McAllen and Laredo only have two or three intersections equipped with cameras.

To cover a wide range of traffic flow, traffic control, environmental setting and intersection geometric conditions, six categories of intersections were selected. Table 28 shows those six categories with their characteristics.

Table 28: Intersection Selection Categories

Category	Characteristics
1	• High volume • Urban area • Number of left-turn lanes = 1 • Signal control: protected left turn
2	• Low volume • Urban area • Number of left-turn lanes = 1 • Signal control: protected left turn
3	• Urban area • Signal control: permitted left turn
4	• Traffic control: unsignalized
5	• Number of left-turn lanes ≥ 2
6	• Rural area • Signal control: protected left turn

According to the results of the survey conducted earlier, intersections with high historical accident rate are highly preferred for the study. In addition, the selected intersections need to be equipped with traffic monitoring cameras because video taping is the major method for collecting the left-turn queue length information at the study intersections. For the city of Houston, after making contact with the traffic operation manager of Harris County, a list of 22 intersections with traffic monitoring cameras was received. To collect more information about those 22 intersections, a field visit was conducted. The information collected during the field visit included traffic condition, intersection layout, and the locations of cameras. After conducting the field visit, 15 intersections were selected in Houston. All of the intersections were actuated controlled signalized intersections. For Austin, 13 intersections were selected according to the recommendations of the engineers in the Austin Traffic Management Center (TMC) and the accident rates at these intersections. The selected intersections are under different types of traffic controls, including actuated, pretimed and no signal controls. Finally, 28 intersections were selected as the candidate study sites. The intersection selection results are presented in Table 29.

Table 29: Intersection Selection Results

Intersection Categories	Study Sites			
	Intersection Type Description	Final Selected Intersections		
		Houston	Austin	Subtotal
Category 1	• High volume • Urban area • Number of left-turn lanes = 1 • Signal control: protected left turn	9	10	**19**
Category 2	• Low volume • Urban area • Number of left-turn lanes = 1 • Signal control: protected left turn	-	2	**2**
Category 3	• Urban area • Signal control: permitted left turn	2	-	**2**
Category 4	• Traffic Control: unsignalized	1	-	**1**
Category 5	• Number of left-turn lanes ≥ 2	2	1	**3**
Category 6	• Rural area • Signal control: protected left turn	1	-	**1**
Subtotal		**15**	**13**	**28**

4.2 Data Collection Methods

As mentioned in the data collection plan, the required data can be categorized into four groups: traffic flow information, signal timing information, intersection geometric information, and historical accident data. Different methods were used to collect these groups of data, including obtaining information from Traffic Management Centers, field visiting, and recording traffic video.

4.2.1 Obtain Information from Traffic Management Centers

The following data was directly collected through contacting traffic management centers:

- Existing traffic signal timing information, including signal planning (schedule), cycle length, split, and left-turn phase type, and
- Accident data for the period of 18 to 36 months.

4.2.2 Field Visiting

To collect more information about the candidate intersections, a site visit form was designed before conducting the field visits (see Appendix B for a sample intersection). During the field visit, the information that was collected includes intersection layout (the lengths of the existing left-turn lanes, the number of lanes in all approaches, etc.), type of signal controls, locations of cameras, posted speed limits, traffic conditions (low/high volumes), and other observed information.

4.2.3 Video Recording

The traffic video data at the 28 selected intersections was collected through the traffic surveillance cameras controlled by the TMCs in Houston and Austin districts according to the developed data collection plan. For each intersection, 2 to 6 hours of traffic video data was collected (average of 3.5 hours per intersection). Data collection was conducted during the morning and/or evening peak hours (AM or PM peaks) of the weekdays, due to the fact that left-turn overflows were most likely to occur during those periods. The collected traffic video data first were stored in tapes and later retrieved in the laboratory. The typical equipment setup and the coverage of traffic cameras are illustrated in Figure 11. Camera No. 1 is the existing traffic surveillance camera installed in the intersection and it targets at the subject direction. This camera records the traffic in the subject direction and covers the longest queue length in the left-turn lane and adjacent through lane. Traffic cones were setup at fixed distance as the landmarks in the video for processing the collected traffic video by Video Image Vehicle Detections (VIVD) systems. Camera No. 2 is the portable camcorder that targets at the opposing traffic.

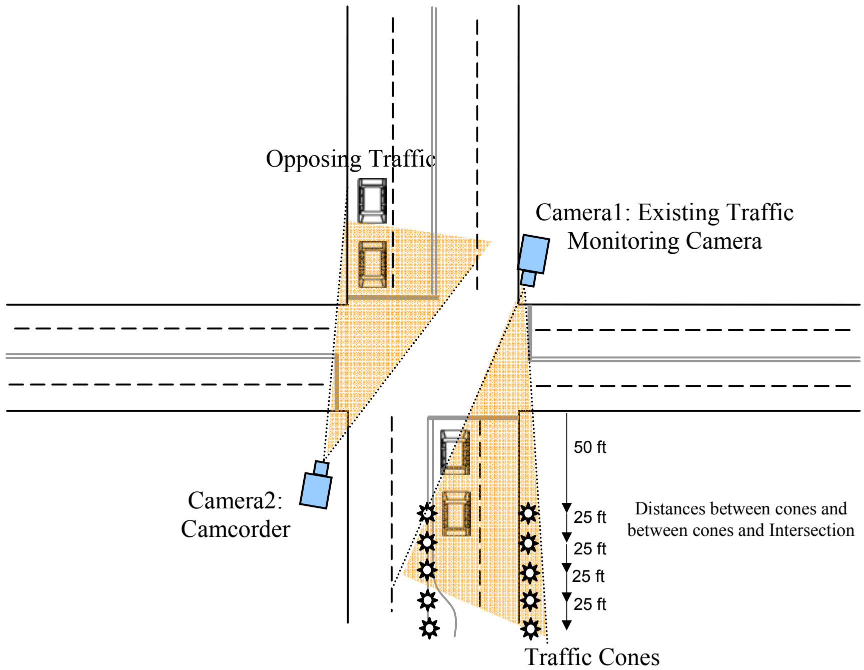


Figure 11: Equipment Setup in the Field

4.3 Data Retrieval

The data collected from each intersection underwent a preliminary analysis and examination to identify any problems during the data collection. As a result, the intersections with low quality video images were dropped. The recorded videos were processed cycle by cycle in the university laboratory by manually counting, by using a developed excel program, or by using Video Image Vehicle Detection system (VIVD) to retrieve the required traffic flow information. This information includes the following: left-turn volume, through traffic volume, queue length in the subject left-turn lane, percentage of heavy vehicles in the subject approach, queue length in the through lane (adjacent to subject left-turn lane), cycle failure percentages (left-turn queue carryover percentage), head-way, start-up time, and approaching speed of vehicles. For all of the studied intersections, left-turn lane *v/c* (volume to capacity) ratios and percentages of left-turn queue carryover were calculated.

4.4 Data Collection Results

Table 30 lists the study intersections in Austin and Figure 12 marks their locations on the map.

Table 30: Study Intersections in Austin

Intersection ID	Name	Subject Direction	Type of Left-Turn Lane	Left-Turn Signal
-	Anderson & Burnet	Northbound	TWLTL*	Protected
78	Braker & Metric	Westbound	Exclusive Double	Protected
456	Braker & Burnet	Eastbound	Exclusive Single	Protected
462	Lamar & 5th	Southbound	TWLTL*	Protected
119	Brodie & Slaughter	Northbound	Exclusive Double	Protected
432	Manchaca & Slaughter	Westbound	Exclusive Single	Protected
197	Turtle Creek & 1st	Southbound	Exclusive Single	Permitted
399	Burnet & Justin	Northbound	TWLTL*	Protected-Permitted
81	Lamar & 45th	Westbound	TWLTL*	Protected-Permitted
102	Lamar & 38th	Eastbound	TWLTL*	Protected-Permitted
103	Lamar & 6th	Northbound	TWLTL*	Protected-Permitted
118	Airport & M.L.K.	Westbound	TWLTL*	Protected-Permitted
164	Pleasant Valley & 7th	Westbound	Exclusive Single	Protected-Permitted
355	Congress & Slaughter	Eastbound	Exclusive Single	Protected-Permitted
778	Lamar & Toomey	Northbound	Exclusive Single	Unsignalized

* Two-Way Left-Turn Lane

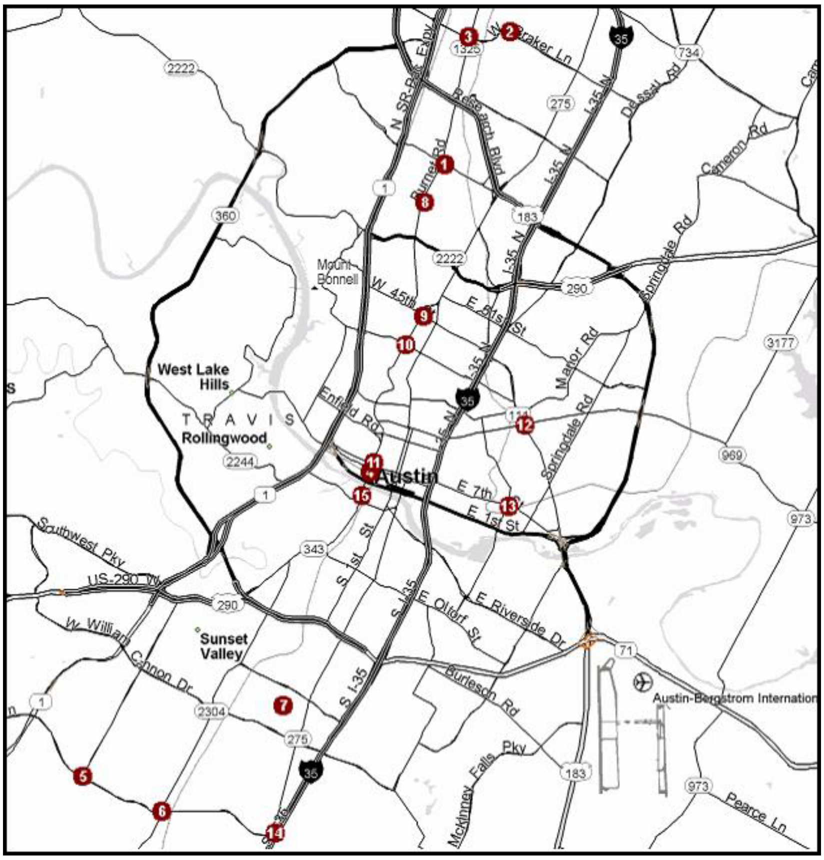


Figure 12: Map of Study Intersections in Austin

Study intersections in Houston are listed in Table 31 and their locations are marked on the map in Figure 13.

Table 31: Study Intersections in Houston

Intersection ID	Name	Subject Direction	Type of Left-Turn Lane	Left-Turn Signal
3213	Eldridge & West	Westbound	Exclusive Single	Protected
3404	Kuykendahl & Cypreswood	Southbound	Exclusive Single	Protected
3405	Atoscocita & Wilson	Westbound	Exclusive Single	Protected
3102	Atoscicita & Will Clayton	Westbound	Exclusive Single	Protected
3106	Mason & Kingsland	Northbound	Exclusive Single	Protected
3317	Westgreen & Kingsland	Southbound	Exclusive Single	Protected
3302	Louetta & Jones	Eastbound	Exclusive-Shared*	Protected
3217	Louetta & Kuykendahl	Eastbound	Exclusive Single	Protected
3221	TX-6 & Little York	Eastbound	Exclusive-Shared*	Protected
3206	TX-6 & Clay	Eastbound	Exclusive Double	Protected
3304	Clay & Barker Cypress	Southbound	Exclusive Single	Protected
3212	FM-529 & Eldridge	Westbound	Exclusive Single	Protected
3209	Barker Cypress & Little York	Westbound	Exclusive Single	Protected

* One lane is exclusive and the other is shared with through traffic.

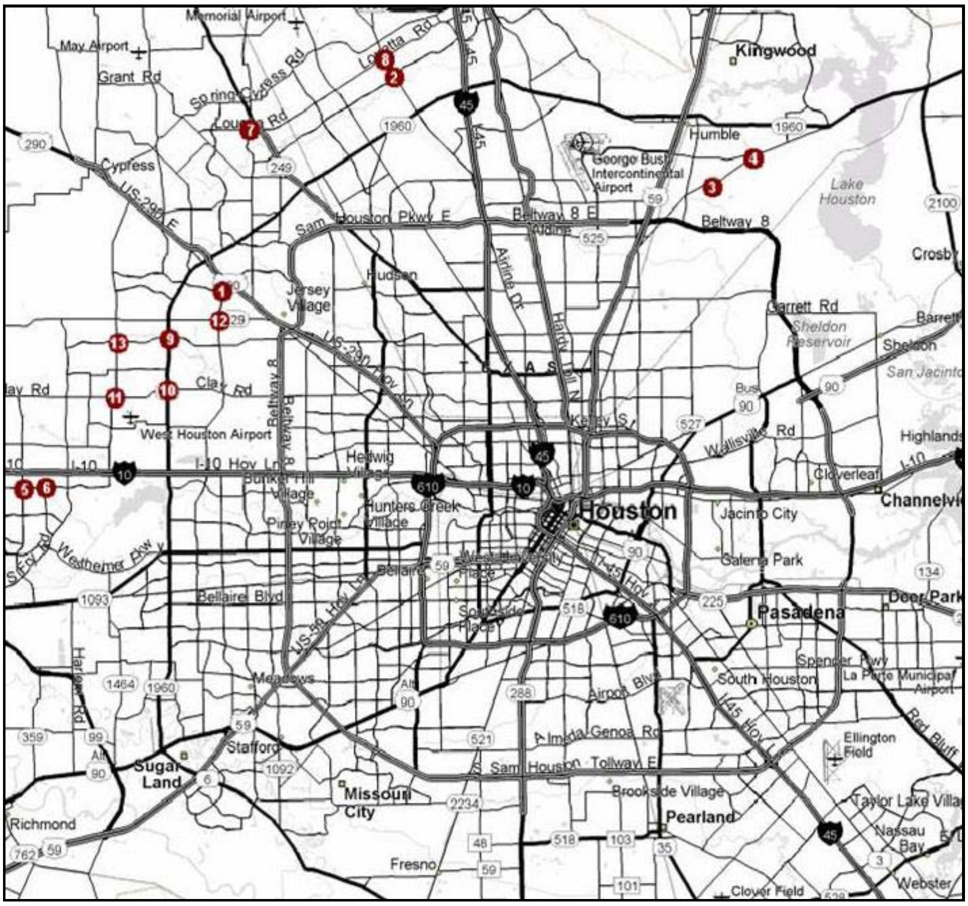


Figure 13: Map of Study Intersections in Houston

Table 32 lists the left-turn lane *v/c* ratios (Volume to Capacity ratios) and the left-turn queue carryover percentages (the percentage of cycles in that the left-turn queue cannot be cleared in one cycle and would have to be carried over to the next cycle) for all the intersections. This table also includes the left-turn overflow rates. When a left-turn lane is too short to accommodate all of the turning vehicles, the left-turn vehicle will overflow to the adjacent through lane. This will cause rear-end accidents between through and left-turn vehicles. For the 28 intersections that were studied in this research, the recorded traffic videos were carefully examined to identify the cycles with left-turn overflow problems. The percentages of cycles with left-turn overflow problem, which is referred to as left-turn overflow rates, were calculated.

Table 32: The Left-Turn Lane *v/c* Ratio and Left-Turn Queue Carryover Percentage of the Studied Intersections

Intersection ID	Name	Location	Left-Turn Lane *v/c* Ratio	Left-Turn Queue Carryover (%)	Left-Turn Overflow Rate (%)
-	Lamar & Toomey	Austin	0.05	0%	0%
78	Anderson & Burnet	Austin	0.49	0%	0%
456	Braker & Metric	Austin	0.23	0%	0%
462	Braker & Burnet	Austin	0.42	0%	0%
119	Lamar & 5^{th}	Austin	0.67	13.86%	25%
432	Brodie & Slaughter	Austin	0.66	2.5%	0%
197	Manchaca & Slaughter	Austin	0.77	41.38%	62%
399	Turtle Creek & 1st	Austin	0.06	0%	0%
81	Burnet & Justin	Austin	0.1	0%	0%
102	Lamar & 45^{th}	Austin	0.49	0%	0%
103	Lamar & 38^{th}	Austin	0.51	2.56%	0%
118	Lamar & 6^{th}	Austin	0.7	9.9%	31.25%
164	Airport & M.L.K.	Austin	0.36	0%	0%
355	Pleasant Valley & 7^{th}	Austin	0.12	0%	0%
778	Congress & Slaughter	Austin	0.56	10.53%	0%
3213	Eldridge & West	Houston	0.63	0%	23.5%
3404	Atoscocita & Wilson	Houston	0.66	0%	0%
3405	Atoscicita & Will Clayton	Houston	0.03	0%	0%
3102	Mason & Kingsland	Houston	0.58	4.5%	30.3%
3106	Westgreen & Kingsland	Houston	0.23	0%	0%
3317	Louetta & Jones	Houston	0.41	0%	0%
3302	Louetta & Kuykendahl	Houston	0.72	5.6%	0%
3217	TX-6 & Little York	Houston	0.74	14.9%	0%
3221	TX-6 & Clay	Houston	0.75	23.7%	0%
3206	Clay & Barker Cypress	Houston	0.71	1.7%	0%
3304	Kuykendahl & Cypreswood	Houston	0.21	0%	0%
3212	FM-529 & Eldridge	Houston	0.75	18.6%	0%
3209	Barker Cypress & Little York	Houston	0.62	3.1%	12.3%

From Tables 30, 31, and 32, it can be found that the data collection covered a wide range of intersections with different congestion levels (Volume to Capacity ratios), left-turn signal control modes (protected, permitted, and protected-permitted), and types of left-turn lanes. The left-turn *v/c* ratios were all significantly less than 1. We obtained an interesting finding from the collected data, which is that, although all of the 28 intersections were subject to undersaturated conditions, the left-turn queue carryover problem occurred frequently for the intersections with left-turn *v/c* ratios within the range of 50% to 80% (see Table 6). For example, at the intersection of Manchaca and Slaughter, where the left-turn *v/c* ratio is 77%, the queue carryover problem was observed in more than 40% of the cycles during the data collection time period.

CHAPTER 5
METHODOLOGY

Left-turn storage lengths should be sufficiently long to store the longest expected queue with a high probability. In this chapter, a new method for estimating the storage lengths of left-turn lanes at signalized intersections will be presented. Then, the determination of storage length of left-turn lanes at unsignalized intersections will be discussed.

5.1 Determination of Storage Length of Left-Turn Lanes at Signalized Intersections

The left-turn queue formed in a signalized intersection consists of two parts: (1) the vehicles that arrive during the red phase (red-phase queue), and (2) the queue carried over from previous cycles (leftover queue). However, existing methods have limitations in estimating these two parts of a queue. In addition, most of the existing methods neglect the queue carried over from previous cycles. As it was discussed in Chapter 4, although all of the studied intersections in this research have undersaturated conditions (all the left-turn *v/c* ratios are significantly less than 1), the left-turn queue carryover occurred frequently for those intersections with left-turn *v/c* ratios between 50% and 80%. In other words, even for intersections with left-turn *v/c* ratios of less than 1, queue carryover occurred frequently. Thus, the leftover queue cannot be neglected for those intersections. Figure 14 shows a queuing diagram of the patterns of left-turning vehicle arrivals and departures that allowed the analysis of queue formation at a signalized intersection. The dotted line represents cumulative arrivals and the solid dark line represents cumulative departures. The queue length is represented by the vertical distance between the arrival line and the departure line. The change of signal phases by cycles is indicated in the time axis. Note that the left-turn phase in Figure 14 represents a general protected-permitted left-turn phase. This method can also be applied to protected-only left turn phase (if g_2 =0) or permitted-only left turn phase (if g_1 =0). Therefore, it covers all three left-turn signal control modes: protected, permitted, and protected-permitted.

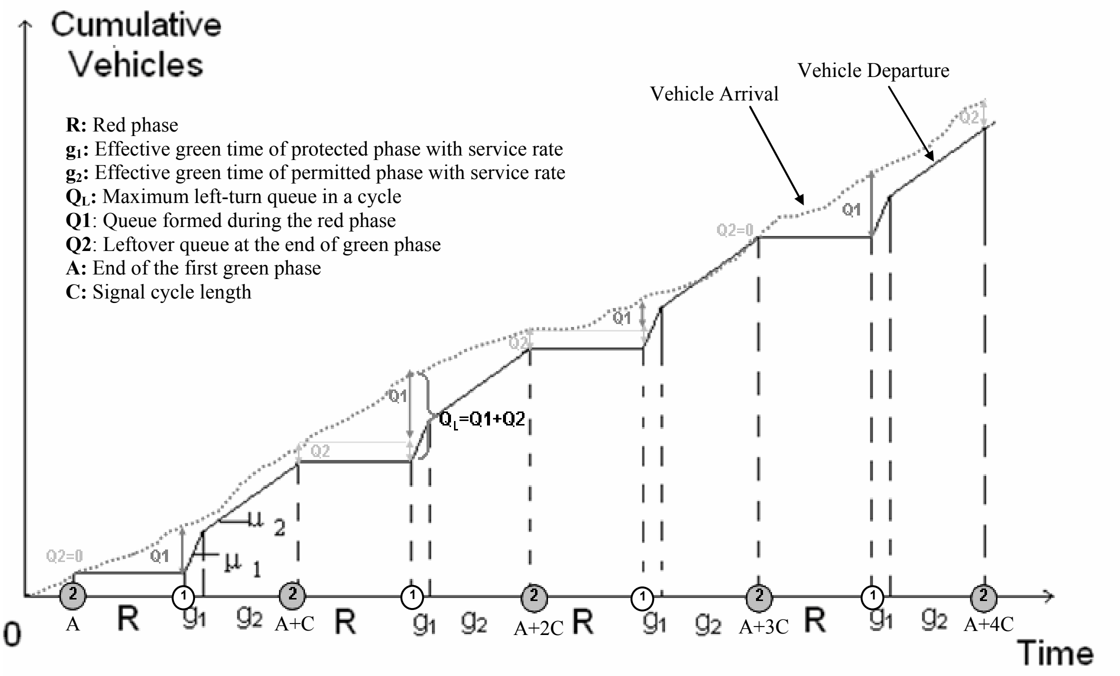


Figure 14: Cumulative Vehicle Arrival and Departure Processes on a Left-Turn Lane

Figure 14 shows that in each cycle, the left queue is most likely to reach its maximum length Q_L , at the end of the red phase (point 1). The maximum queue in each cycle Q_L consists of two parts: (1) Q_1, the queue formed during the red phase, and (2) Q_2, the leftover queue at the end of the green phase. According to the field observation, the leftover queue (Q_2) cannot be neglected for many intersections and both parts of the queue length must be added together to derive accurate estimates of the left-turn queue length at a signalized intersection. Therefore, in this study, two individual models were developed to estimate these two parts of the left-turn queue at a signalized intersection, i.e., Q_1 and Q_2. Some assumptions were made in developing the two models:

1. The arrivals of left-turn vehicles are random and follow a Poisson distribution.
2. The left-turn green time and cycle length are constant in the model. For an actuated intersection, it suggests using average cycle lengths and green times. In this study, the 28 study intersections are all actuated controls with fixed cycle lengths (for signal

coordination purposes). The signal phase splits during peak hour periods are very close to the programmed splits (or nominal splits) in most cycles. This may be due to the fact that the left-turn green phase is hardly to be “gapped out” (excessive amounts of time between cars) under the heavy traffic conditions that occur during peak hour periods. Therefore, using programmed splits (or nominal splits) for actuated intersections having fixed cycle lengths is suggested.

3. The intersection is a stable system. The average number of arrivals during a signal cycle is less than the maximum number of vehicles that can be discharged during the green phase (intersection service rate). In other words, the left-turn *v/c* ratio for the intersection is less than 1.

The frame work of the developed model for left-turn lane storage estimation is presented in Figure 15. The detailed description of the model development can be found in Appendix C.

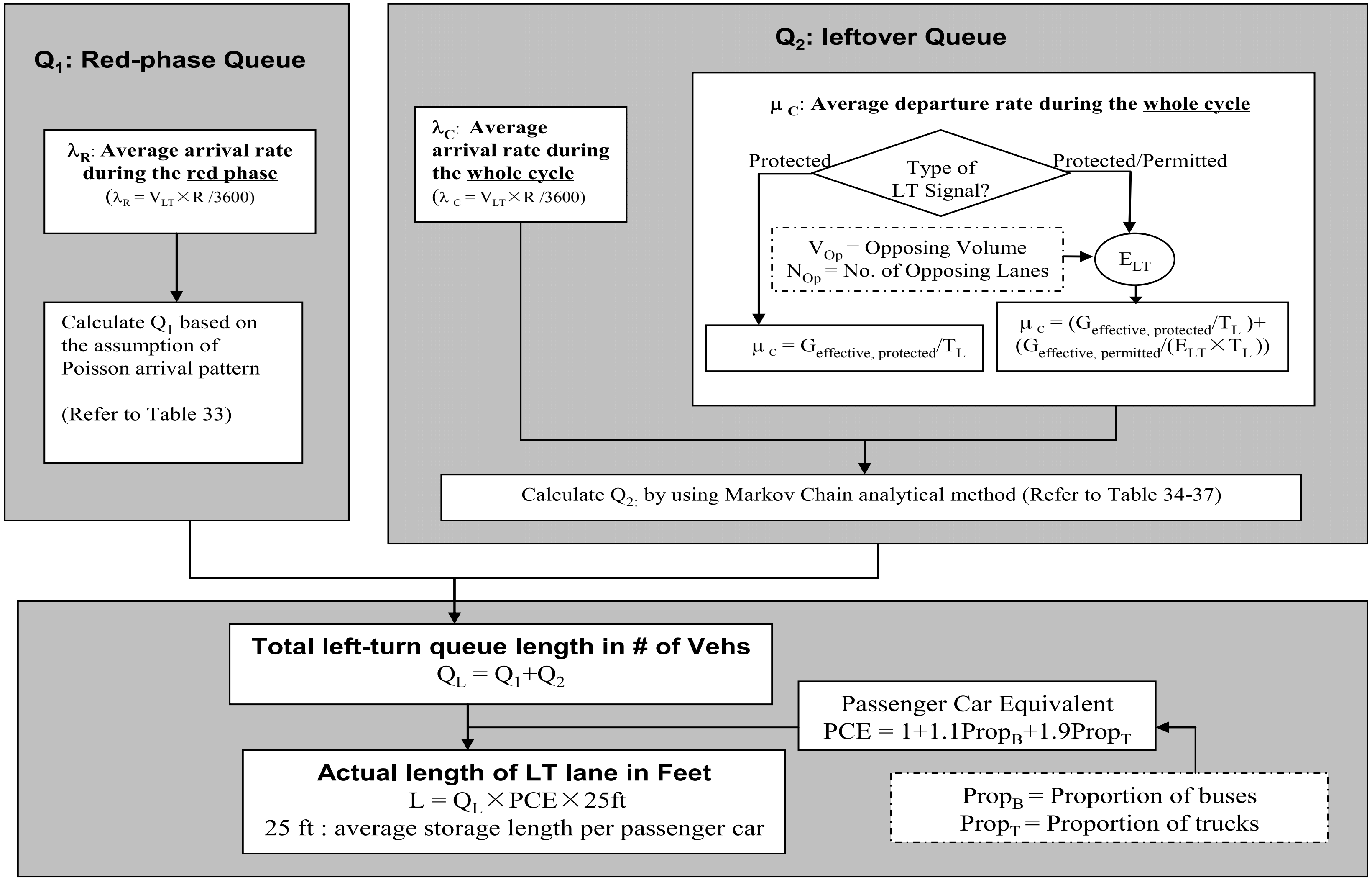


Figure 15: Model Framework

The brief introduction of every sub-model in Figure 15 is presented in the following:

5.1.1 Model 1: Estimation of Queue Formed During Red Phase in Number of Vehicles (Q_1)

According to assumption 1, the vehicles that arrive in a left-turn lane follow a Poisson distribution (see Appendix C). A reference table (Table 33) was developed to estimate the maximum queue length, i.e. Q_1, formed during the red phase based on the observed average number of arrivals during a red phase (i.e. $\lambda_t R$, where λ_t is the average left-turn arrival rate in vehicles per second and R is the length of the red phase in seconds) at different probability levels (95%, 97.5%, 99%, and 99.5%). Note that, for the intersection with a protected-permitted left-turn phase, if the service rate μ_2 during the permitted phase is significantly less than the arrival rate λ_t ($\lambda_t / \mu_2 > 2$), the red phase should also include the permitted phase, and the average number of vehicles that can be discharged during the permitted phase should be subtracted from Q_1.

Table 33: Queue Formed During the Red Phase in Number of Vehicles (Q_1)

Average Number of Arrivals During Red Phase	Poisson Arrival Pattern at Different Probability Levels			
	95%	97.5%	99%	99.5%
1	2	3	4	4
2	4	5	6	6
3	6	6	7	8
4	7	8	9	10
5	8	9	10	11
6	10	11	12	13
7	11	12	13	14
8	12	14	15	16
9	14	15	16	17
10	15	16	18	19
11	16	17	19	20
12	18	19	20	21
13	19	20	22	23
14	20	21	23	24
15	21	23	24	25
16	22	24	26	27
17	24	25	27	28
18	25	26	28	29
19	26	28	29	31
20	27	29	31	32
21	28	30	32	33
22	30	31	33	35
23	31	32	34	36
24	32	34	36	37
25	33	35	37	38
26	34	36	38	40
27	35	37	39	41
28	37	38	41	42
29	38	40	42	43
30	39	41	43	45
31	40	42	44	46
32	41	43	45	47
33	42	44	47	48
34	43	45	48	49
35	45	47	49	51
36	46	48	50	52
37	47	49	51	53
38	48	50	53	54
39	49	51	54	56
40	50	52	55	57

5.1.2 Model 2: Estimation of Leftover Queue at the End of the Green Phase in Number of Vehicles (Q_2)

The time points A, A+C, A+2C….A+nC in Figure 14 are the ends of the green phases. At these time points, if all the left-turning vehicles cannot be cleared at the intersection, the remaining vehicles are carried over to the next cycle. The leftover queue lengths at these time points A, A+C, A+2C….A+nC form a Discrete-Time Markov Chain (DTMC) (see Appendix C). A series of reference tables (Tables 34, 35, 36, and 37) were developed to estimate the maximum leftover queue length Q_2, based on the observed average number of arrivals during a whole cycle, ($\lambda_t C$, where C is the cycle length) and intersection service rate in vehicles per cycle (m, see appendix C for the estimation of m) at different probability levels (95%, 97.5%, 99%, and 99.5%).

Table 34: Leftover Queue at the End of Green Phase in Number of Vehicles (Q_2) at 95% Probability Level

Service Rate / Arrivals*	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1		1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2			2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3				4	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4					5	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5						6	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6							8	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7								9	4	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8									11	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9										12	5	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10											13	6	3	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
11												15	6	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12													16	7	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0
13														17	8	4	3	1	0	0	0	0	0	0	0	0	0	0	0	0
14															19	8	5	3	1	0	0	0	0	0	0	0	0	0	0	0
15																20	9	5	3	2	0	0	0	0	0	0	0	0	0	0
16																	21	10	6	4	2	1	0	0	0	0	0	0	0	0
17																		22	11	6	4	2	1	0	0	0	0	0	0	0
18																			23	11	7	4	3	1	0	0	0	0	0	0
19																				24	12	7	5	3	1	0	0	0	0	0
20																					25	13	8	5	3	2	0	0	0	0
21																						26	13	8	5	3	2	1	0	0
22																							27	14	8	6	4	2	1	0

***** The units of both service and arrival rates are number of vehicles per cycle (vpc).

Table 35: Leftover Queue at the End of Green Phase in Number of Vehicles (Q_2) at 97.5% Probability Level

Service Rate / Arrivals*	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1		2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2			3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3				5	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4					7	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5						8	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6							10	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7								12	5	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8									14	6	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9										15	7	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10											17	8	5	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
11												19	9	5	3	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0
12													20	9	6	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0
13														22	10	6	4	3	1	0	0	0	0	0	0	0	0	0	0	0
14															23	11	7	5	3	2	0	0	0	0	0	0	0	0	0	0
15																25	12	7	5	3	2	1	0	0	0	0	0	0	0	0
16																	26	13	8	5	4	2	1	0	0	0	0	0	0	0
17																		27	14	8	6	4	3	1	0	0	0	0	0	0
18																			28	15	9	6	4	3	2	0	0	0	0	0
19																				29	15	10	7	5	3	2	1	0	0	0
20																					30	16	10	7	5	3	2	1	0	0
21																						31	17	11	7	5	4	2	1	0
22																							32	18	11	8	6	4	3	1

* The units of both service and arrival rates are number of vehicles per cycle (vpc).

Table 36: Leftover Queue at the End of Green Phase in Number of Vehicles (Q_2) at 99% Probability Level

Service Rate / Arrivals*	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1		2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2			4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3				7	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4					9	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5						11	5	3	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6							13	6	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7								15	7	4	3	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8									17	8	5	3	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9										20	9	6	4	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10											22	10	6	4	3	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0
11												24	11	7	5	3	2	1	0	0	0	0	0	0	0	0	0	0	0	0
12													26	12	8	5	4	3	1	0	0	0	0	0	0	0	0	0	0	0
13														28	14	9	6	4	3	2	1	0	0	0	0	0	0	0	0	0
14															29	15	9	6	5	3	2	1	0	0	0	0	0	0	0	0
15																31	16	10	7	5	4	2	1	0	0	0	0	0	0	0
16																	32	17	11	8	6	4	3	2	1	0	0	0	0	0
17																		33	18	11	8	6	4	3	2	1	0	0	0	0
18																			34	19	12	9	6	5	3	2	1	0	0	0
19																				35	20	13	9	7	5	4	3	2	0	0
20																					35	21	13	10	7	6	4	3	2	1
21																						36	22	14	10	8	6	4	3	2
22																							37	23	15	11	8	6	5	4

* The units of both service and arrival rates are number of vehicles per cycle (vpc).

Table 37: Leftover Queue at the End of Green Phase in Number of Vehicles (Q_2) at 99.5% Probability Level

Service Rate / Arrivals*	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1		3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2			5	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3				8	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4					10	5	3	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5						13	6	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6							15	7	5	3	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7								18	9	5	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8									20	10	6	4	3	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9										23	11	7	5	4	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10											25	12	8	6	4	3	2	1	0	0	0	0	0	0	0	0	0	0	0	0
11												28	14	9	6	5	3	2	1	0	0	0	0	0	0	0	0	0	0	0
12													30	15	9	7	5	4	3	1	0	0	0	0	0	0	0	0	0	0
13														31	16	10	7	6	4	3	2	1	0	0	0	0	0	0	0	0
14															33	17	11	8	6	5	3	2	1	0	0	0	0	0	0	0
15																34	19	12	9	6	5	4	3	1	0	0	0	0	0	0
16																	35	20	13	9	7	5	4	3	2	1	0	0	0	0
17																		36	21	14	10	7	6	4	3	2	1	0	0	0
18																			37	22	14	10	8	6	5	4	2	1	0	0
19																				38	23	15	11	8	7	5	4	3	2	1
20																					38	25	16	12	9	7	6	4	3	2
21																						38	26	17	12	9	7	6	5	3
22																							38	27	18	13	10	8	6	5

***** The units of both service and arrival rates are number of vehicles per cycle (vpc).

5.1.3 Estimation of Maximum Left-Turn Queue Length in Number of Vehicles (Q_L)

Finally, the maximum queue length, Q_L , at a signalized intersection in number of vehicles can be estimated by adding both estimated queues together.

$$Q_L = Q_1 + Q_2 \tag{20}$$

where Q_1 and Q_2 are estimated by using Table 33 and Tables 34 to 37, respectively, and the probability that the queue length is less than Q_L will be greater than the multiplier of the probabilities for Q_1 and Q_2 estimation (i.e. $\alpha_1 \times \alpha_2$) because:

$$\begin{aligned} \Pr ob(queue < Q_L) &= \Pr ob(leftover\ queue + queue\ in\ red\ phase < Q_1 + Q_2) \\ &\geq \Pr \mathrm{ob}(\text{leftover queue} < Q_1)\ \Pr \mathrm{ob}(\text{queueinredphase} < Q_2) \\ &= \alpha_1 \times \alpha_2 . \end{aligned}$$

Therefore, by using Q_1 and Q_2 at different probability levels of 95%, 97.5%, 99%, 99.5% (introduced in Tables 33, 34, 35, 36, and 37), the maximum queue length can be calculated at the probability levels of 90%, 95%, 98%, and 99%. For example, if Q_1 and Q_2 are estimated by using Table 33 and Table 35 with probability levels of $\alpha_1 = 97.5\%$ and $\alpha_2 = 97.5\%$, respectively, the probability that the left-turn queue length is less than Q_L will be greater than $\alpha_1 \times \alpha_2 = 95\%$.

5.1.4 Storage Length of Left-Turn Lane in Actual Distance

The estimated queue length in vehicles must be converted to the actual left-turn lane length in feet (or meters). Since large vehicles such as trucks and recreational vehicles require more storage space, vehicle mix must be considered in this conversion. In this study, the actual storage length, L, is estimated by the method recommended by Kikuchi et al. (1993) as follows:

$$L = Q_L \times PCE \times L_P \tag{21}$$

where:

Q_L : queue length in number of vehicles, given in Equation (20)

L_P : average storage length of a passenger car. The recommended value is 25 (Messer (1977))

PCE: passenger car equivalent factor, which is calculated as follows:

$$PCE = 1 + (E_B\text{-}1)Prop_B + (E_T\text{-}1)Prop_T \tag{22}$$

where:

$Prop_B$: proportion of buses or recreational vehicles

$Prob_T$: proportion of trucks

E_B: PCE of a bus or recreational vehicle (the recommend value is 2.1)

E_T: PCE of a truck (the recommend value is 2.9)

5.1.5 Intersections with Exclusive and Shared Left-Turn Lanes

According to the field observations of this study, at the intersections with exclusive and one shared left-turn lanes (Figure 16), approximately 60% of the left-turn volume is assigned to exclusive left-turn lane. Therefore, it is recommended to use this portion of left-turn volume in the estimation of the queue storage length of exclusive left-turn lane for the intersections with exclusive and shared left-turn lanes.

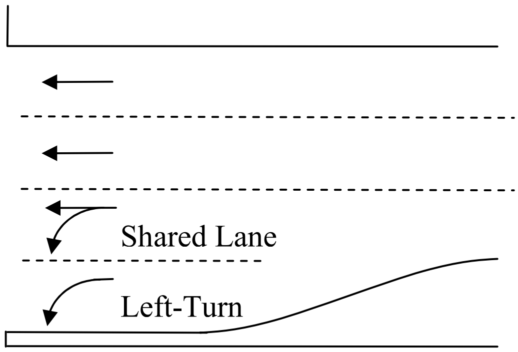


Figure 16: Intersections with Exclusive and Shared Left-Turn Lanes

5.1.6 Case Study

The intersection of Lamar and 5th Street in Austin was chosen to demonstrate the application of the proposed model. Traffic data was collected at this study site over a period of about 2.4 hours (54 cycles). The hourly volume of the left-turn lane was 210 vehicles per hour (vph). The queue length was counted for each cycle and the maximum queue length among all the cycles was 18 vehicles. For signal timing, the cycle length was 150 seconds: a 125-second red phase and a 25-second protected phase. According to the field observation, the average left-turn headway was 2.02 seconds. Based on this information, the left-turn queue length in this location can be calculated based on the following three steps.

Step 1. Estimation of the queue formed during the red phase in number of vehicles (Q_1): The average number of arrivals during the red phase, $\lambda_t R$, is

$$\lambda_t R = \frac{210}{3600} \times 125 = 7.3$$

According to Table 33, Q_1 is equal to 12 when the average number of arrivals during the red phase, $\lambda_t R$, is 7.

Step 2. Estimation of the leftover queue at the end of the green phase in number of vehicles (Q_2): The average number of arrivals during one cycle, $\lambda_t C$, is

$$\lambda_t C = \frac{210}{3600} \times 150 = 8.75$$

The intersection service rate per cycle, m, is

$$m = m_1 = \text{Nearest Integer to}\left(\frac{g_p - \ell_1 + e}{T_L}\right) = \text{Nearest Integer to}\left(\frac{25-2+2}{2.02}\right) = 12$$

According to Table 35, Q_2 is equal to 4 when the arrival rate is 9 vehicles per cycle (vpc) and service rate is 12 vpc.

Step 3. Estimation of the maximum left-turn queue length in number of vehicles (Q_L):

$$Q_L = Q_1 + Q_2 = 12+4=16$$

Checking with the queue length observed from the field (Q_o) over the 54 cycles, we found that the maximum queue length was 18 (it is the only observed Q_o greater than the estimated queue length Q_L = 16). The 95th-percentile observed queue length in this location is 14 and the percentage of ($Q_o > Q_L$ = 16) is 1.8%. Therefore, the proposed model produced an accurate left-turn queue length estimate for this location.

5.1.7 Model Evaluation

To evaluate the model developed for this study, we applied the model to all of the selected study intersections to estimate left-turn queue lengths. The model results were compared with the queue lengths observed in the field (Q_o) and estimates from other models. The 95th percentile of the observed queue length served as the baseline for comparison. Three existing methods for left-turn queue length estimation were selected:

1. The rule of thumb method that suggests the length of the left-turn lane to be two times the total length of the average arrivals during one signal cycle (TxDOT Roadway Design Manual).
2. The model that only considers the queue formed during the red phase (Kikuchi (1993)).
3. The model that uses the M/M/1 queuing system to estimate left-turn queue lengths (Oppenlander and Oppenlander (1989)).

The second model is referred to as the red-phase-only model and its queue length estimation is given by Equation (C-2) (see Appendix C). The third model is referred to as an M/M/1 model and its queue length is estimated by the following equation:

$$n = (\log \alpha - \log (1-\lambda_t/\mu_t))/\log (\lambda_t/\mu_t) \tag{23}$$

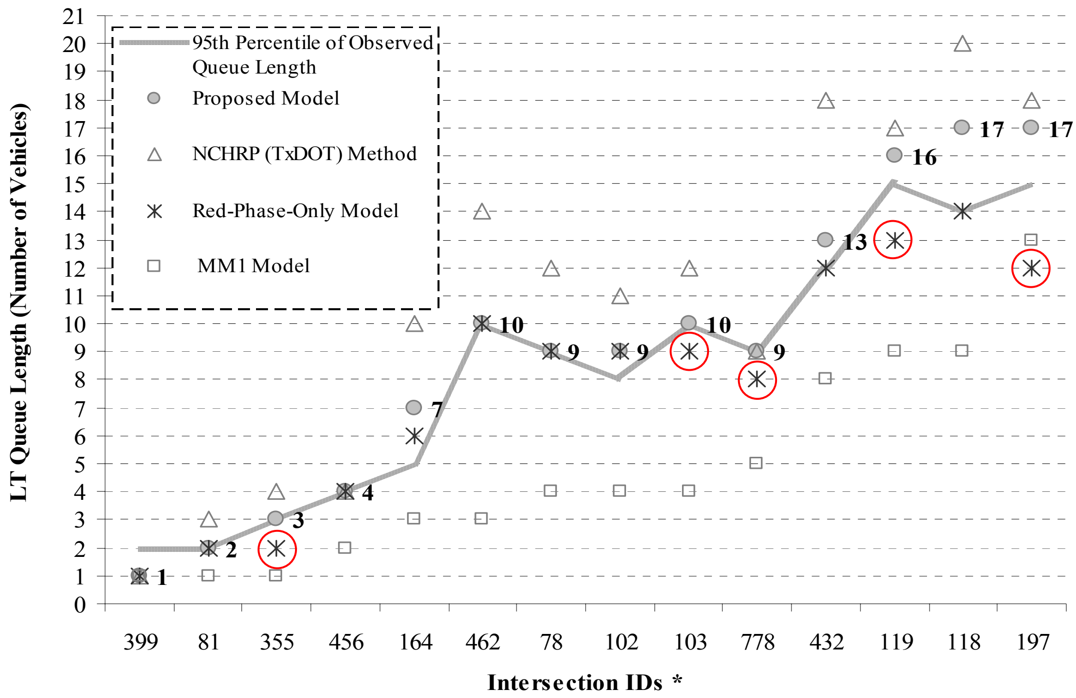
where

λ_t : average number of arrivals of left-turning vehicles in vehicles per second

μ_t : average number of departures of left-turning vehicles in vehicles per second

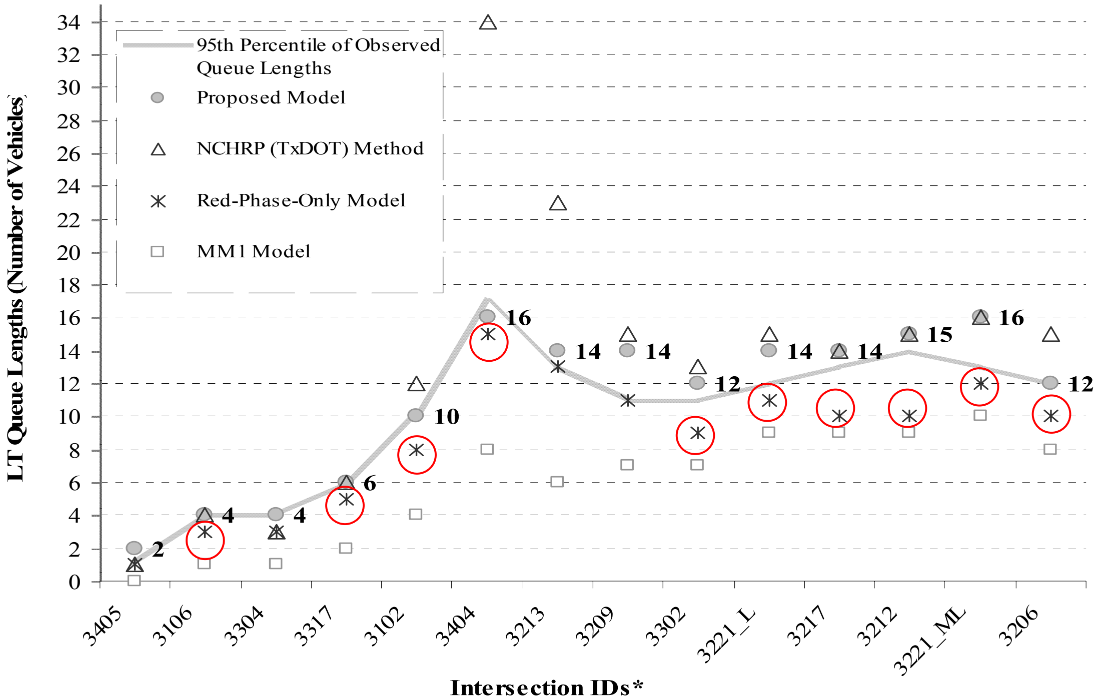
α : given probability level, which is 95% in this model evaluation

The models were also applied to the intersections selected for this study, and the estimated maximum left-turn queue lengths from the models were compared with the results from the proposed model. Figures 17 and 18 compare all four models at intersections in Austin and Houston. The intersections in the figures are arranged by their *v/c* ratios. From this comparison, it is shown that most of the queue lengths estimated by the proposed model are slightly above the line of the 95th percentile of the observed queue length. In this situation, this indicates the high accuracy of these estimates. The figure also shows some of the problems with estimates from the other three models. The red-phase-only model (the second model) underestimates the queue length at intersections where queue carryover occurred (see the marks with circles in Figures 17 and 18). The M/M/1 model (the third model) significantly underestimates the queue length for all intersections. This consistent underestimation is because this model cannot properly represent the stop-and-go operation of signalized intersections. As a result, the queue formed during the red phase cannot be appropriately modeled. On the other hand, the TxDOT Roadway Design Manual method significantly overestimated the queue length at the intersections with high arrival rates. Figure 18 shows that it can overestimate the queue length up to 170% at some intersections. It is because the rule of thumb method does not consider the high service rate (the long green phase for left-turn movements) at these intersections.



*The intersection IDs are listed in Table 32.

Figure 17: Comparison of Proposed Model with Existing Models at Intersections in Austin



*The intersection IDs are listed in Table 32.

Figure 18: Comparison of Proposed Model with Existing Models at Intersections in Houston

These evaluation results suggest that both parts of the left-turn queue (the red-phase queue and the leftover queue) must be considered in estimating left-turn queue lengths. The method proposed in this study considerably outperforms existing methods by appropriately modeling both parts of a left-turn queue.

5.2 Determination of Storage Length of Left-Turn Lanes at Unsignalized Intersections

The existing method for determination of storage length of left-turn lanes at unsignalized intersections were discussed in Chapter 2. Among all the reviewed methods, the rule of thumb estimation is recommended for the determination of storage length of left-turn lanes at unsignalized intersections due to its simplicity and easiness of the implementation. Although this method does not consider the factors that determine the departure rate such as the opposing volume, the estimated results will not be greatly influenced because of the low traffic volume at unsignalized intersections. The existing rule of thumb method in TxDOT Roadway Design Manual is as following:

$$Q_L = (V/30)\ (2) \tag{24}$$

where:

Q_L: maximum left-turn queue length in number of vehicles

V: left-turn volume (vph)

After that, the actual left-turn storage length in feet can be estimated by using Equation (19). Finally, it is required to verify if the estimated storage length meet the minimum requirements. This is because the left-turn lane cannot be very short even if the left-turn queue is short. According to TxDOT Roadway Design Manual, a minimum storage length of 100 ft is set up for the intersections with very low left-turn volume.

5.4 Summary

In this chapter, a new method for estimating the storage lengths of left-turn lanes at signalized intersections was developed. The left-turn queue length was estimated by considering two factors: (1) the vehicles that arrive during the red phase (red-phase queue), and (2) the queue of vehicles carried over from previous cycles (leftover queue).

The influencing factors, including left-turn volume, opposing traffic volume, signal timing, vehicle headway, and vehicle mix were taken into account in the model. After that, a method was recommended to convert the estimated queue length in number of vehicles to the required storage lengths.

In the model evaluation, the results of the proposed model were compared with the queue length observed in the field and the estimates from other models. The evaluation results showed that the developed model considerably outperforms the existing methods by providing more accurate estimates of left-turning queue lengths.

For the unsignalized intersections, it is recommended that the existing rule of thumb method in TxDOT Roadway Design Manual be used for estimating the storage lengths of left-turn lanes.

CHAPTER 6

EXAMINATION OF PROCEDURES WITH OTHER TRAFFIC MODELS

The purpose of this chapter is to examine traffic model-based procedures to determine the required deceleration and storage length requirements. It focuses on the following two critical topics:

- Determination of queue storage length of left-turn lanes by using traffic models, and
- Determination of deceleration length of left-turn lanes by using traffic models.

For the first topic, this study is to exam the procedures for estimating left-turn queue storage length by using different traffic models and to compare the estimated left-turn queue storage length with the results from the developed analytical method (Chapter 5) and the field observations. The traffic models considered in this task include SYNCHRO (Version 6.0), SimTraffic (Version 6.0) and VISSIM (Version 4.20).

For the second topic, this chapter developed a traffic simulation-based method for left-turn deceleration length estimation. The microscopic simulation model, VISSIM, is used for this task. The estimated deceleration length is compared with the results from the TxDOT Roadway Design Manual.

Finally, based on the estimations of the queue storage length and the deceleration length, the total length of left-turn lane is estimated by considering the difference in traffic conditions during the peak-hours and off-peak hours.

6.1 Determination of Queue Storage Length by Using Traffic Models

This section discusses how to determine the queue storage length of left-turn lanes by using different traffic models. It begins with a description of the procedures for estimating left-turn lane storage length with the emphasis on the comparison between different models. Then, the results from different traffic models were validated by comparing with the field observations. Finally, recommendations on how to select the most cost-effective traffic model for left-turn lane storage length estimation were provided.

6.1.1 Procedures for Determining the Queue Storage Length by Using Traffic Models

Three traffic models were selected for determining the left-turn lane queue storage length. These models include the macroscopic simulation model SYNCHRO, and the microscopic simulation models, SimTraffic and VISSIM.

SYNCHRO (Version 6)

SYNCHRO is a macroscopic traffic model. It is a windows-based traffic signal timing program with modeling and optimization capabilities. The key features of this program include capacity analysis, coordination, actuated signal modeling, and time-space diagrams. SYNCHRO calculates average (50^{th}) and 95^{th} percentile queue lengths and indicates queue spillback. Note that, as a macroscopic traffic model, SYNCHRO uses analytical method to estimate the queue length. Therefore, it can acquire the results once the coding is completed. To compare with the results from the analytical method developed in Chapter 5, the 95^{th} percentile queue length from SYNCHRO is adopted for this study.

SimTraffic (Version 6)

SimTraffic is a microscopic model which is integrated with SYNCHRO. SimTraffic performs microscopic traffic simulation and can emulate the traffic operations at signalized, un-signalized intersections and freeway sections. SimTraffic reports the maximum queue, average queue and 95th percentile queue measures for each lane. The maximum queue is the maximum queue observed for the entire analysis time period. The average queue is the average of maximum queues observed in every 2 minutes. The 95th percentile queue is equal to the average queue plus 1.65 standard deviations. In this study, the 95th queue measure is adopted.

VISSIM (Version 4.20)

VISSIM is a microscopic, time-step and behavior-based simulation model developed to model urban traffic and public transit operations. It provides the user with simulation results consisting of on-screen animation of vehicular movements, traffic signal operation, and detector actuations. Besides its animation capabilities, VISSIM generates numerous user-customizable output files. This information includes queue length statistics, detailed signal timing information (green time, cycle length, etc.), graphical output such as time space diagrams, speed profiles, and environmental indicators, etc. The reasons that the research team selected this micro-simulation tool are: 1) few studies have been conducted on estimating queue lengths by using VISSIM, and 2) comparing with other microscopic simulation models, VISSIM provides more flexibility in specifying the model outputs.

Table 38 gives the step-by-step procedures for using these traffic models to estimate left-turn lane queue storage length. Some important issues in modeling and the differences between these three models are introduced by steps.

Table 38: Procedures to Determine Left-turn Lane Queue Storage Length by Using Three Traffic Models

Procedure/Software		SYNCHRO	SimTraffic	VISSIM
Coding and Inputs	Geometric Components	• Links • Number of lanes • Lane type • Lane width and length • Speed limit	Import network and inputs from SYNCHRO	• Scaling • Links • Connectors • Speed: Edit speed distribution and define speed reduction area
	Traffic	• Volume of each lane • Percentage of HV and PHV	Import networks and inputs from SYNCHRO	• Volume of each link • Volume of each route • Percentage of HV and speed of each type of vehicles
	Signal Timing	• Cycle length • Splits and phasing • Left-turn signal mode	Import networks and inputs from SYNCHRO	• In SSG: Edit the signal with NEMA type • Set the placement of each signal head
Model Calibration		N/A	Two calibration parameters: overflow and blockage percentages (static graphics function)	Two calibration parameters: overflow and blockage percentages (Observe the 3D on-screen animation)
Model Run and Outputs Analysis		The results can be obtained directly after coding.	• Setup "number of runs": 30 times • Get the 95 percentile maximum queue lengths among all the cycles directly from the report	• Setup "multiple run": 30 times • Get the 95 percentile maximum queue lengths after processing the simulation outputs

Step 1. Inputs and Coding

The first step in the process is to compile the input data needed for these three models, including transportation supply (e.g. geometric components), traffic demand (e.g. traffic volume), and traffic controls (e.g. traffic signal timing). The input data required for SYNCHRO and SimTraffic models are exactly the same since they are an integrated package and SimTraffic directly import the network coding from SYNCHRO.

The inputs for VISSIM include many different features. Unlike SYNCHRO/ SimTraffic, in which the network coding is in one file, the coding in VISSIM involves two input files: (1)

simulator ,and (2) signal state generator (SSG). The simulator generates traffic and provides graphical interface for coding network. Before “drawing” the network, right “scaling” should be set by importing at least one scaled background graphic. When drawing the network and applying the attributes (e.g., lane widths, speed zones, priority rules, etc.), special attentions needs to be paid to the following issues: 1) although links are used in the simulator, VISSIM does not have a traditional node structure like SYNCHRO/SimTraffic. The lack of nodes provides the user with more flexibility to control traffic operations and set vehicle paths within an intersection. However it would cost more efforts in coding the network, for example, “Connector” should be setup to bridge left-turn link and the upstream through link and it should be specified in “Attributes” that the left-turn lane could only be connected with the adjacent through lane; 2) different with SYNCHRO/SimTraffic, in which only a speed limit is required as the speed input data, VISSIM requires the input of temporary speed changes (e.g., for bends and turns) as well as the permanent speed changes which are defined in “Reduced Speed Areas” and “Desired Speed Decisions”, respectively; 3) for volume inputs, not only the volume on each entry point of the network needs to be inputted, but also the “route” should be defined and the volume for each route should be inputted; and 4) “Queue Counter” needs to be setup in the simulator file so that the queue length in the format of number of vehicles in each cycle can be obtained from the output files.

The SSG is where the signal control logic resides. Users have the ability to define the signal control logic and thus emulate most types of control logic found in the real world. After coding the SSG file (.nse), signal heads are placed on each lane, according to the “signal group” edited in the SSG file. Note that the signal heads could not be placed on connectors.

Step 2. Model Calibration

Once the network coding is completed, model calibration is needed for the microscopic model, i.e., SimTraffic and VISSIM, to ensure that the simulation can correctly represent the real-world traffic conditions in the field. Actually, model calibration is the most critical step in traffic simulation study and it provides the basis for further simulation results analysis. During model calibration, the on-screen animation must be carefully observed for the reasonableness of the simulation results. Occasionally, some unrealistic driver behavior will be observed in VISSIM (e.g., the blocked vehicles in the right lane can jump a long queue of through vehicles to make right turns in the red time). In these cases, changes to VISSIM input parameters (e.g., check the signal heads placement and reduced speed area) are needed.

In this study, the left-turn lane overflow rates and the left-turn lane blockage rates are used as the calibration measures. In SimTraffic, the percentages of overflow and blockage can be directly obtained by using the static graphic function. In VISSIM, the accurate overflow and blockage percentage can only be obtained by manually counting through observing the on-screen animation.

Step 3. Model Runs and Outputs

For SYNCHRO, the queue lengths results could be obtained directly once the coding was completed because it is a macroscopic model. For the microscopic models, i.e. SimTraffic and VISSIM, once the model calibration was completed, the simulation model need to be run multiple times to overcome the randomness in traffic simulation results.

In this study, the SimTraffic model was run for 30 times with 30 random seeds; the 95th left-turn queue length with each random seed was obtained directly from the reports generated by the simulation, and then the average queue length from these 30 runs were calculated for each scenario.

For VISSIM model, after 30 times' model running with 30 different random seeds, the outputs from VISSIM simulation is processed to obtained the 95th percentile queue length. Note that, the original output file only includes the maximum left-turn queue lengths for each cycle for each run.

6.1.2 Result Validation

Among twenty-four intersections collected in Houston and Austin, seven intersections (Table 39) which have serious overflow or blockage problems are selected for validating the results from the traffic models. From Table 39, it can be found that at least one of the two problems occurs more than or equal to 25 percent of cycles in these intersections. The reasons for selecting these intersections are:

1) Intersections with overflow and blockage problems have inadequate left-turn queue storage lengths, and providing accurate estimate of left-turn queue storage length is one important objective of this study, and
2) Overflow and blockage percentage have been selected as measures for model calibration.

Table 39: Selected Intersections for Result Validation

Left-Turn Signal	ID	Name of Intersection	Direction	Type of Left-Turn Lane	*v/c* Ratio	LT Queue Carryover (%)	LT Lane Overflow (%)	LT Lane Blockage (%)
Protected	119	Lamar and 5th	SB	TWLTL	0.67	13.86	25.00	0
	197	Manchaca & Slaughter	WB	Exclusive Single	0.77	41.38	62.00	23.05
	3213	Eldridge & West	WB	Exclusive Single	0.63	0	23.50	76.00
	3102	Mason & Kingsland	NB	Exclusive Single	0.58	4.50	30.30	60.60
	3106	Westgreen & Kingsland	SB	Exclusive Single	0.23	0	0	65.00
	3209	Barker Cypress & Little York	WB	Exclusive Single	0.62	3.10	12.30	98.00
Protected-Permitted	118	Lamar & 6th	NB	TWLTL	0.7	9.90	31.25	0

* Two-Way Left-Turn Lane

As introduced in section 1, after completing the network coding and inputs, model calibration was conducted. Table 40 lists the model calibration results of SimTraffic and VISSIM models for these seven intersections.

Table 40: Calibration Results of SimTraffic and VISSIM Models

Intersection	Overflow (%)			Blockage (%)		
	Observation	SimTraffic	VISSIM	Observation	SimTraffic	VISSIM
Westgreen & Kingsland	0.0	0~5	0.0	65.0	>50	57.0
Mason & Kingsland	30.3	20~30	26.1	60.6	>50	65.2
Eldridge & West	23.5	20~30	24.0	76.0	>50	68.0
Barker Cypress & Little York	12.3	10~20	7.5	98.0	>50	96.0
Manchaca & Slaughter	62.0	> 50	63.0	23.0	20~30	26.0
Lamar & 6th	31.3	30~50	34.8	0.0	0	0.0
Lamar & 5th	25.0	30~50	30.4	0.0	0	0.0

Table 40 shows that both of the microscopic simulation models, i.e. SimTraffic and VISSIM, produced quite close results to the observations in the field, which indicated that the base model had been well calibrated. Finally, after running simulation and analyzing the model outputs, the left-turn queue lengths of these seven intersections were estimated. The predicted results of left-turn queue lengths from these three models were listed in Table 41, which were compared with the results from the developed analytical model in Chapter 5 (TSU model) and the observations from the field.

Table 41: Left-turn Queue Length Predicted by Traffic Models

Intersection		Macroscopic	Microscopic		Analytical	95% QL* (Observation)
		95% QL* (SYNCHRO)	95% QL* (SimTraffic)	95% QL* (VISSIM)	95% QL* (TSU Model)	
Westgreen & Kingsland		3	4	4	4	**4**
Mason & Kingsland		10	11	16	9	**11**
Eldridge & West		16	16	18	14	**14**
Barker Cypress & Little York		13	16	19	14	**12**
Manchaca & Slaughter		18	19	15	17	**15**
Lamar & 6th		20	20	24	17	**16**
Lamar & 5th		20	19	19	16	**18**
Model Performance	**Score**	**1**	**3**	**3**	**3**	NA
	Accuracy (1-error%)	**83.9**	**85.0**	**57.3**	**90.6**	

* QL: Queue Length

In Table 41, the shaded cells indicate the best predictions which are closest to the field observation in that intersection among all the queue length predictions. According the number of best predictions from each model, a "score" is given as one criterion to evaluate the model performance. Another evaluation criterion is the "accuracy", which is the level of accuracy of the prediction and can be calculated by following equation:

$$\text{Accuracy} = 1 - \text{Avg}\left\{ \frac{|\hat{L} - L|}{L} \times 100\% \right\} \tag{25}$$

where:

$\hat{L}$: Predicted queue length from the traffic model

L : Left-turn queue length observed in the field

By using these two criteria, the performance of these traffic models were evaluated according to results presented in Table 41:

1) Among the three traffic simulation models, SimTraffic model has the best performance in both "Score" and "Accuracy" evaluations.
2) Although VISSIM obtains the high score, the average accuracy level of VISSIM predictions is lower because it sometimes significantly overestimates left-turn queue length. The reason for this is that, in VISSIM, the queue counter set in the left-turn lane tends to include the adjacent through vehicles under the left-turn lane overflow conditions. This is why the average accuracy level of the predictions from VISSIM is low.
3) Compared with the traffic simulation models, the analytical model developed in Chapter 5 (TSU model) has better performance in both "Score" and "Accuracy" evaluations.

6.1.3 Recommendations Based on Accuracy and Time-Cost of the Tested Traffic Models

Based on the estimation procedures and the validation results given above, the performance and the cost of time for modeling (is referred to as "time-cost" in this study) of these traffic models were compared in Table 42. The performance of each model is based on the "Score" and "Accuracy" evaluation results provided in Table 41. The time-cost of each model is calculated based on the average execution time of each step in the whole estimation procedure.

Table 42: Comparison of Model Performance and Time-Cost

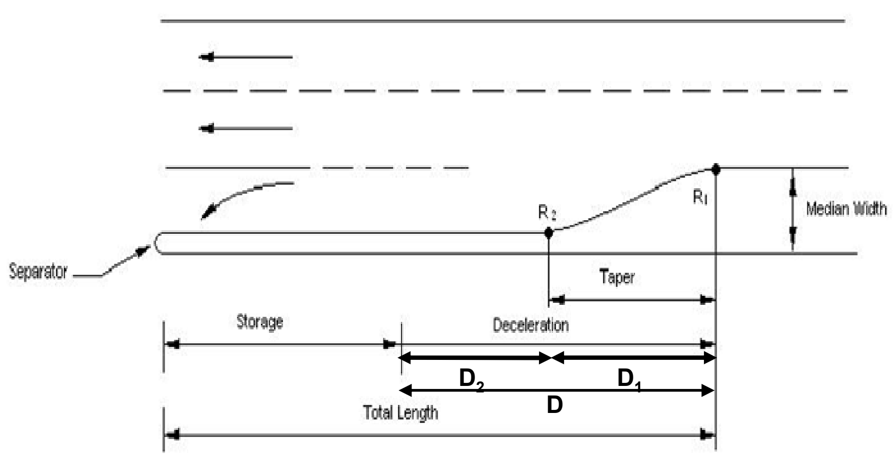
Criteria/Software		SYNCHRO	SimTraffic	VISSIM	TSU Model
Model Performance	Score	1	3	3	3
	Accuracy	83.9%	85.0%	57.3%	90.6%
Time-Cost for Each Simulation Scenario	Coding and Inputs	12-18min	12-18min	20-35min	N/A
	Model Calibration	N/A	10-20min	30-80min	N/A
	Model Runs and Outputs Analysis	N/A	10- 15min	20- 30min	N/A
	Total	12-20min	32 – 53min	70 -145min	5-10min

According to the comparisons in Table 42 and the experiences of the research team in this task, following recommendations are given:

1) Among the three traffic models, SimTraffic is considered to be the most cost-effective model in left-turn queue length estimation, due to its good prediction performance and acceptable time-cost.
2) VISSIM costs half an hour for modeling an intersection more than other models because the network coding and model calibration steps cannot be completed in a timely fashion. In addition, due to its low level of accuracy, VISSIM is not recommended for queue lengths estimation.
3) The developed analytical model (TSU model) over-performs the three traffic simulation models in terms of both model prediction performance and the time-cost.

6.2 Determine the Left-Turn Deceleration Length by Using Traffic Models

Left-turn lane deceleration length (D) is comprised of taper length (D_1) and length for fully deceleration (D_2), as shown in Figure 19. The deceleration length should allow the turning vehicle to come to a comfortable stop prior to reaching the end of the expected queue in the left-turn lane. The insufficient deceleration distance would lead to excessive deceleration rate in the left-turn lane which increases the crash risk, while the too long deceleration distance may entice through drivers unintentionally enter the left-turn lane. Therefore, the determination of the appropriate left-turn lane deceleration length is critical for both safety and efficiency of an intersection.



D_1 = Taper length: distance traveled while driver decelerates and maneuvers laterally;
D_2 = Length for fully deceleration: distance traveled during full deceleration and coming to a stop or to a speed at which the turn can be comfortably executed.
$D = D_1 + D_2$: Deceleration Length of Left-turn Lane

Figure 19: Deceleration Length of Left-turn Lane

Existing methods for deceleration length determination are based on engineering experiences or analytical methods. One drawback of the analytical methods is that it assumes that the vehicle merges to left-turn lane with a constant deceleration rate which can not reflect the real-world situation. TxDOT Roadway Design Manual provides the deceleration lengths under different speed limits, as shown in Table 43. However, according to the result of the survey conducted in this study (Chapter 3), it may yield longer deceleration length.

Table 43: Deceleration Lengths for Single Left-turn Lane

Source: TxDOT Roadway Design Manual

Speed (mph)	Taper Length (ft)	Deceleration Length (ft)
30	50	160
35	50	215
40	50	275
45	100	345
50	100	425
55	100	510

To investigate the appropriate deceleration length for a left-turn lane, a simulation-based method is proposed in this task. The simulation models can emulate the dynamic traffic conditions in the real world and the interactions between the vehicles. As a result, a more accurate relationship between deceleration length and the deceleration rate can be derived based on the simulation results. This relationship is critical for determining the left-turn deceleration length since the deceleration length should allow the turning vehicle to approach to a comfortable stop prior to reaching the end of the expected queue in the left-turn lane.

In this task, a simulation-based method for deceleration length estimation was developed by using the microscopic simulation model VISSIM. The research team chose this micro-simulation tool because of its capability in obtaining second-by-second individual vehicle information, such as location, speed, and deceleration rate, and its ability to customize the driving behavior parameters, such us gap acceptance, maximum lane change deceleration rate. This part of discussion includes two sub-sections: 1) A simulation-based method for deceleration length determination, and 2) Output analysis and results.

6.2.1 A Simulation-Based Method for Deceleration Length Determination

The key step of this method is to find out the relationship between deceleration length and deceleration rate under a given speed condition based on traffic simulation results. Once the relationship is developed, the corresponding deceleration length to the given comfortable deceleration rate can be identified. In this research, the comfortable deceleration rate was assumed to be $9 ft/s^2$ according to the literatures. The detailed procedure for this method is illustrated in the Figure 20.

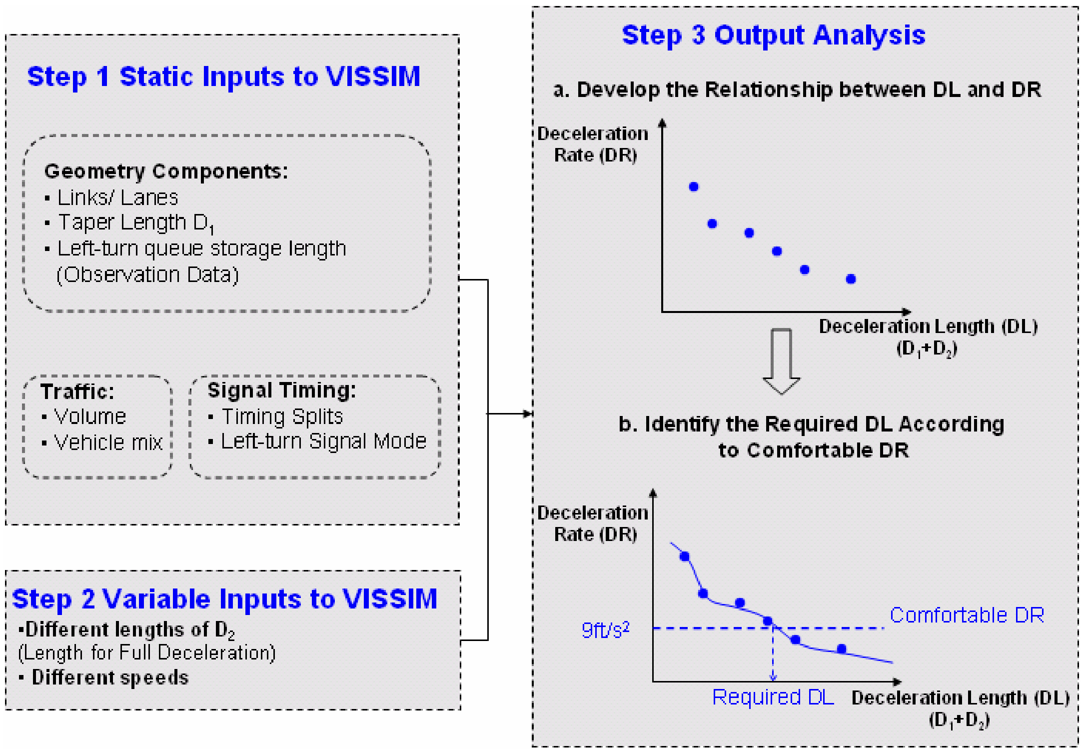


Figure 20: Procedures of Simulation-Based Method for Left-Turn Lane Deceleration Length Estimation

As shown in Figure 20, Step 1 is to finish the network coding by inputting all the static parameters to VISSIM, and to calibrate the baseline model. Note that the taper length D_1 is determined according to the taper lengths recommended by TxDOT Roadway Design Manual. In this manual, the taper length should be 50 feet when the speed limit is less than 45 mph and should be 100 feet when the speed limit is equal or above 45 mph. The estimation of D_1 will be discussed in details in Chapter 9. In this task, we will focus on the estimation of D_2 (length for fully deceleration).

In Step 2, simulation scenarios with different lengths of D_2 under different speed conditions were created and each scenario was run multiple times. In this study, a real-world intersection (Machaca & Slaughter, Austin) was used for developing baseline simulation model. The reason that we chose this intersection is that its geometry condition is typical (single exclusive left-turn lane with two through lanes and one right-turn lane) and the baseline model calibration result is among the best (see Table 40). Then, totally 61 simulation scenarios were created with different deceleration lengths under different speed conditions. Each scenario was run for 15 times with different random seeds, thus a total of 915 simulation runs were conducted.

In Step 3, the relationship between deceleration length and deceleration rate was developed based on the outputs from the simulations. The original output file includes following information: 1) speed and deceleration rate of each vehicle in each time-step (one second), and 2) the location of each vehicle in every second, i.e., the link ID and the X and Y coordination of the vehicle location. The analysis of the outputs is an important step, which will be introduced in details in the next section.

6.2.2 Output Analysis and Results

The first step of output analysis is to identify vehicles which once stopped in D_2 area, (the area for a full deceleration, see Figure 19 and Figure 21). If the vehicle did not stop on D_2 area, this means that either it did not need to stop (e.g. arrived in the green phase) or it used part of the queue storage area for full deceleration (the left-turn queue may be short in some times). Since these are not the most risk situations, vehicles without stops during their driving on D_2 area should be excluded for deriving the deceleration length requirements. Therefore, the second-by-second deceleration rates for the vehicles with full stops on D_2 area were extracted from the output file for analysis. The 85th percentile of the deceleration rates for all these vehicles was calculated. The reasons that we chose 85th percentile of deceleration rate are: 1) It is more conservative to use 85th percentile of deceleration rate instead of the average deceleration rate because it will ensure most (85%) of the vehicles' deceleration rates are below the comfortable deceleration rate, and 2) From the sample distribution of the deceleration rate, it was found that the deceleration rate data spreads widely above 85th percentile, which indicates there were some outliers in the areas above 85th percentile.

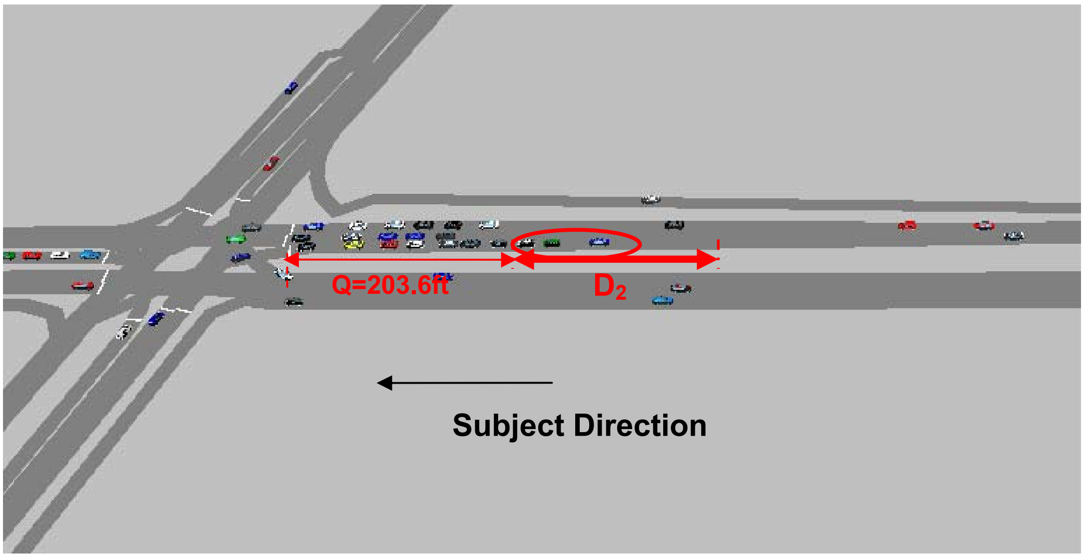


Figure 21: VISSIM Simulation for Intersection Manchaca & Slaughter, Austin

Based on the outputs from the traffic simulation scenarios with different deceleration lengths, the 85th percentile deceleration rate vs. deceleration length are plotted in Figure 22 for different traffic speed conditions.

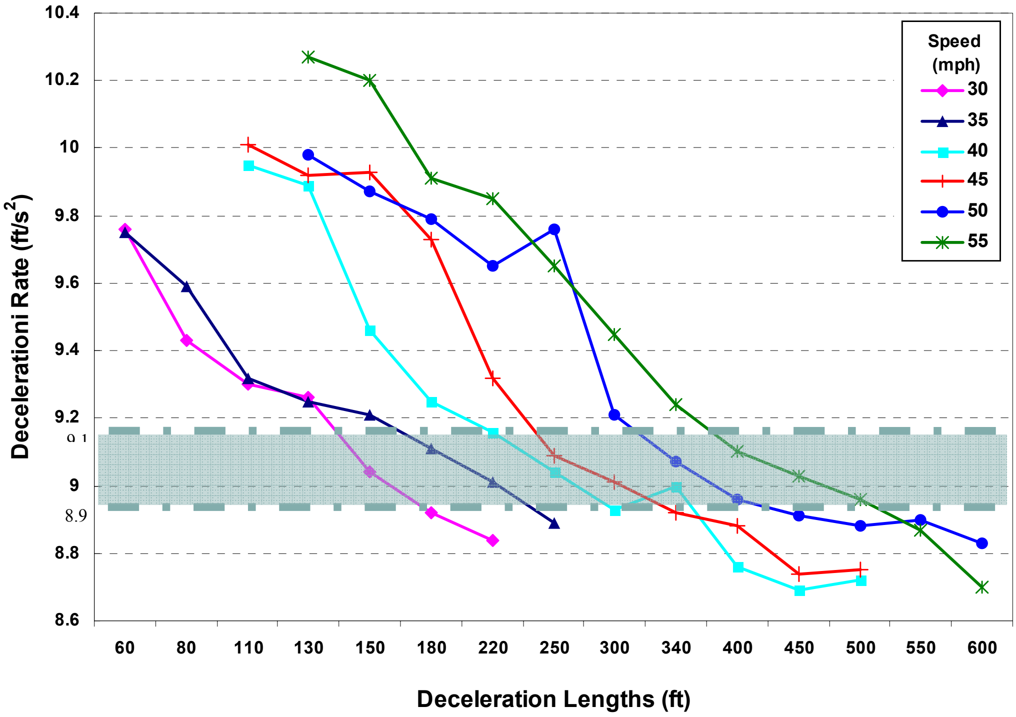


Figure 22: Deceleration Rates vs. Deceleration Lengths

It is found that, in the range of 8.9 ft/s^2 to 9.1 ft/s^2, the curve of the deceleration rate changes steadily and has some fluctuations. Therefore, the range from 8.9 ft/s^2 to 9.1 ft/s^2 (the shaded area in Figure 22), instead of one comfortable deceleration rate, i.e. 9 ft/s^2, was used to derive the required deceleration length. Therefore, all of the points on each curve falling into the shaded area in Figure 22 were used to find the corresponding deceleration lengths. After that, by averaging the deceleration lengths corresponding to these points on each curve, the required deceleration length for the speed of that curve could be derived. The derived results of the deceleration lengths under different speed conditions are listed in Table 44 and they are compared with the results from TxDOT Roadway Design Manual.

Table 44: Results of Deceleration Lengths under Different Speed Conditions

Speed (mph)	Deceleration Length (ft) by Simulation Method	Deceleration Length (ft) by TxDOT
30	165	160
35	200	215
40	287	275
45	323	345
50	397	425
55	450	510

It is found that most of the derived deceleration lengths by the simulation-based method are less than those recommended by the TxDOT Roadway Design Manual. This result agrees with the findings obtained from the survey (Chapter 3).

6.3 Total Length of Left-turn Lanes

Based on the estimation of queue storage length and deceleration length of left-turn lanes introduced in pervious sections, the total design length of left-turn lanes can be determined by adding the estimated storage length and deceleration length together. As discussed in the previous sections, the traffic volume is critical for determining the storage length and the intersection speed is an important factor for determining the deceleration length. Since the traffic volume and speed conditions during peak and the off-peak hours are very different, the total left-turn lane length should be estimated for the peak hour and off-peak hour individually at first. As shown in Figure 23, the heavy traffic volume in the peak hours leads to relatively low speed, so the deceleration length could be shorter during this time period while, at the same time, a longer queue storage length is required. On the other side, in the off-peak hours, the lighter traffic volume usually comes along with higher speed, which results in relatively lower requirements for queue storage lengths but higher requirements for deceleration lengths.

Therefore, the total length of the left-turn lanes can be determined as the maximum of the total lengths estimated for both peak hours and off-peak hours.

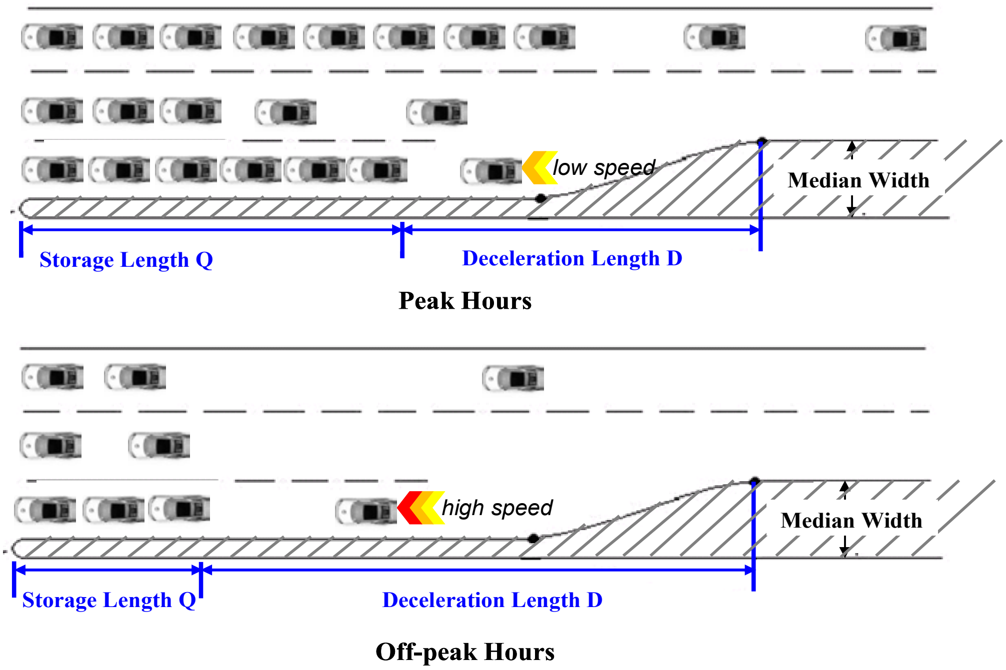


Figure 23: Impacts of Traffic Conditions in Peak Hours and Off-Peak Hours on Determinations of Left-Turn Lane Length

The whole procedure for determining the total length of left-turn lane is illustrated in Figure 24.

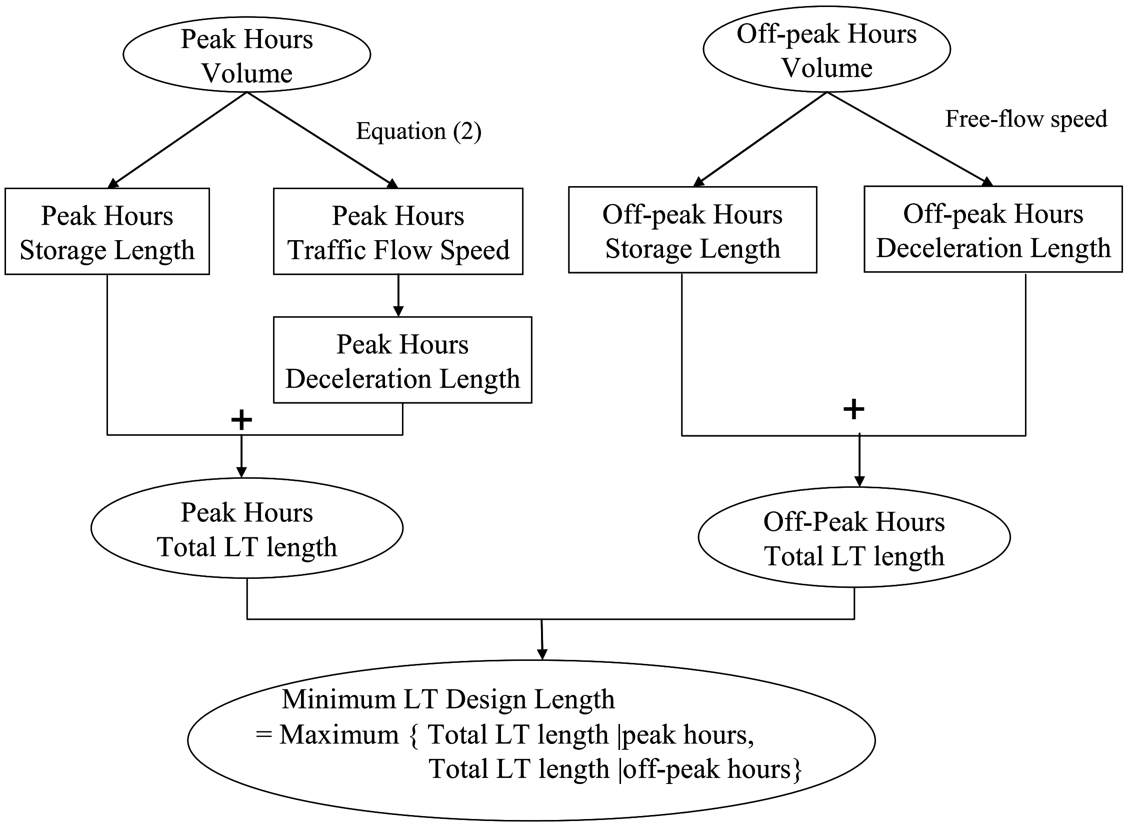


Figure 24: Procedures for Estimating Total Length of Left-turn Lanes

According to the traffic volume, left-turn storage queue length could be estimated by using the SimTraffic simulation model or the TSU model as recommended in Section 1. To estimate the deceleration length, the traffic flow speed needs to be estimated at first. For off-peak hours, the free flow speeds (the speed limits) are adopted for deceleration length estimation. For peak hours, the traffic flow speed is determined by traffic volume. In this study, the BPR (Bureau of Public Roads) equation was recommended for estimating the speed in the congested traffic conditions:

$$S = \frac{S_0}{\left[1 + 0.15(X)^4\right]} \tag{26}$$

where:

S = Average link speed (mph or km/hr)

S_0 = Free-flow link speed (mph or km/hr)

X = Volume to capacity ratio (*v/c*)

After the speed was derived, the deceleration length could be estimated according to Table 44 for both peak hours and off-peak hours. Then, by adding the estimated queue storage length and deceleration length together, the total lengths of left-turn lane for both time periods were estimated. Finally, the required total length of the left-turn lanes could be determined as the maximum of the total lengths under both conditions.

6.4 Summary

This chapter focuses on the estimation of left-turn lane queue storage length and deceleration length by using traffic models.

For storage length, it documents the procedures and methodologies which were used for the left-turn lane queue storage length estimation and examines the model performance and time-cost for the selected traffic models, including SYNCHRO (Version 6.0), SimTraffic (Version 6.0) and VISSIM (Version 4.20). The estimated queue storage lengths from these traffic models are compared with results from the analytical model developed in Chapter 5 (TSU model), and field observations. It is found that:

1) Among the three traffic simulation models, SimTraffic model illustrates the best performance.
2) VISSIM demonstrates relatively poor performance among the three traffic models, since it significantly overestimates the queue lengths during the studied times.
3) Comparing with the traffic simulation models, the analytical model developed in Chapter 5 (TSU model) has better performance.

For left-turn deceleration length estimation, a simulation-based method was developed by using VISSIM 4.20. Compared to the analytical method, the simulation model can better emulate dynamic traffic conditions in the real world and the interactions between vehicles. Therefore, this method should provide better deceleration length estimates than those recommended by analytical methods. Finally, the total left-turn lane design length was estimated by considering the difference in traffic conditions during the peak hours and off-peak hours.

CHAPTER 7
SAFETY BENEFITS OF INCREASED STORAGE LENGTH

In this chapter, the safety benefits of increased storage length will be analyzed. It begins with the introduction of the accident data collected at the study intersections.

7.1 Accident Data

The accident history of an intersection is the key indicator of its safety performance. Generally, different types of accidents may occur at intersections. Figure 25 shows a diagram illustrating possible taxonomy for accident type classification (Rodegerdts et al., 2004).

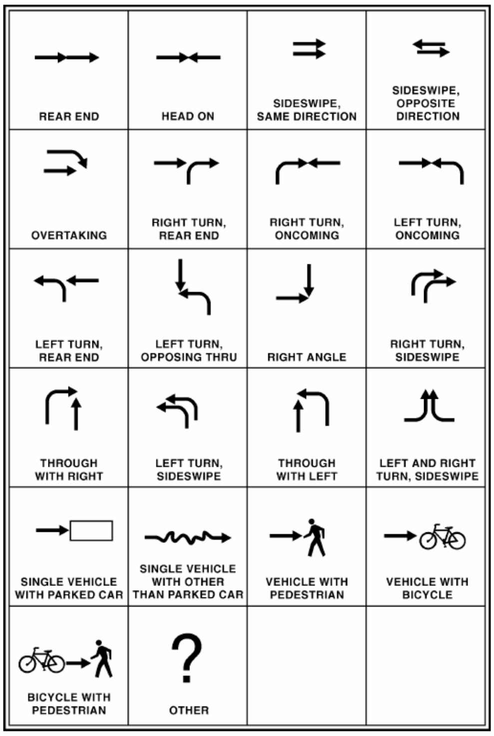


Figure 25: Different Types of Accidents
Source: Rodegerdts et al. (2004)

7.1.1 Rear-End Accidents

Among all these types of accidents, rear-end accidents are directly related to the length of left-turn lane. According to Rodegerdts et al. (2004), installation of a left-turn lane could be expected to decrease rear-end accidents. In addition, the designed left-turn lane should be long enough to accommodate left-turning vehicles. Insufficient lengths will result in left-turn lane overflow which may cause rear-end accident between through and left-turn vehicles. Therefore, rear-end accidents will be investigated in this research. Figure 26 displays this condition.

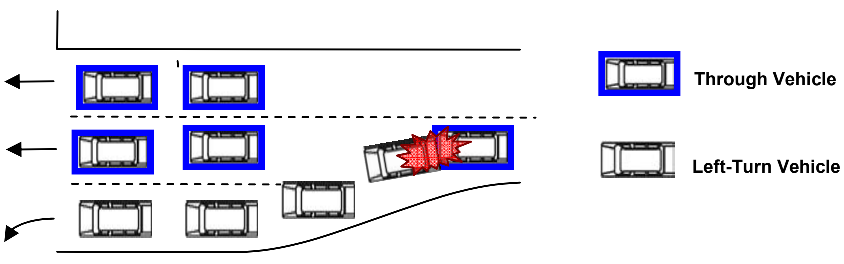


Figure 26: Rear-End Accident Caused by Left-Turn Lane Overflow

7.1.2 Accident Data Collection

To study the safety impacts of the length of left-turn lane, the historical accident data at the study intersections in Austin and Houston was obtained. This data was provided in different formats, varied from one jurisdiction to another. Among 28 intersections which are studied in this research, the accident reports were available for only 21 intersections. Thus, those 7 intersections were excluded from accident data analysis.

7.1.2.1 Austin Accident Data

Accident reports of Austin intersections were obtained from the Traffic Management Center at City of Austin. The reports had been provided in narrative format for a period of 18 to 30 consecutive months (from 2004 to 2006). To identify the rear-end accidents and the severity level of accidents, the reports were carefully examined. For each of the studied intersections, the number of rear-end accidents were counted. The findings from the narrative accident reports are summarized in Table 45 and Table 46. Table 45 is for total number of accidents in the Austin intersections by different severity level and Table 46 is only for rear-end accidents. A total of 115 accidents occurred in the Austin intersections during the recorded time period. Among them, 38 accidents were known as rear-end accidents. Table 46 presents the number and percentage of rear-end accidents in each intersection by severity level.

Table 45: Total Number of Accidents in Austin Intersections

Name	Period (month)	Total Number of Accidents	Total Accidents By Severity Level		
			Fatal	Injury	PDO
Anderson & Burnet	30	6	0	4	2
Braker & Metric	30	16	0	14	2
Braker & Burnet	18	5	0	5	0
Lamar & 5th	18	6	0	3	3
Brodie & Slaughter	30	7	0	5	2
Manchaca & Slaughter	30	10	0	6	4
Burnet & Justin	18	3	0	0	3
Lamar & 45th	30	7	0	4	3
Lamar & 38th	18	4	0	1	3
Lamar & 6th	23	9	0	6	3
Airport & M.L.K.	30	18	1	10	7
Pleasant Valley & 7th	18	13	0	8	5
Congress & Slaughter	18	11	0	10	1
Total	**-**	**115**	**1**	**76**	**38**

Table 46: Rear-End Accidents in Austin Intersections

Name	Period (month)	Number of Rear-End Accidents	Percentage of Rear-End Accidents	Rear-End Accidents by Severity Level		
				Fatal	Injury	PDO
Anderson & Burnet	30	2	33.3%	0	1	1
Braker & Metric	30	2	6.3%	0	1	1
Braker & Burnet	18	3	60%	0	3	0
Lamar & 5th	18	3	50%	0	1	2
Brodie & Slaughter	30	1	14.3%	0	1	0
Manchaca & Slaughter	30	6	60%	0	3	3
Burnet & Justin	18	0	0%	0	0	0
Lamar & 45th	30	5	71.4%	0	3	2
Lamar & 38th	18	1	25%	0	1	0
Lamar & 6th	23	3	33.3%	0	2	1
Airport & M.L.K.	30	6	27.8%	0	5	1
Pleasant Valley & 7th	18	5	38.5%	0	3	2
Congress & Slaughter	18	1	9.1%	0	1	0
Total	**-**	**38**	**33%**	**0**	**25**	**13**

7.1.2.2 Houston Accident Data

The information related to crashes in Houston intersections was obtained from Houston-Galveston Area Council (H-GAC) database for a period of 36 consecutive months (form 1999 to 2001). This information was in GIS format; and a systematic procedure was followed to extract useful information from the GIS-format database. A total of 120 accidents occurred in the Houston intersections. The data is summarized in Table 47 and Table 48. Table 47 presents the total number of accidents occurred at the studied intersections in Houston by severity levels.

Table 47: Accidents in Houston Intersections

Name	Period (month)	Total Number of Accidents	Total Accidents By Severity Level		
			Fatal	Injury	PDO
Eldridge & West	36	6	0	5	1
Atoscocita & Wilson	36	15	0	8	7
Atoscicita & Will Clayton	36	10	0	6	4
Westgreen & Kingsland	36	3	0	2	1
Louetta & Jones	36	19	0	10	9
Louetta & Kuykendahl	36	27	0	19	8
Clay & Barker Cypress	36	22	0	10	12
Barker Cypress & Little York	36	18	0	12	6
Total	**-**	**120**	**0**	**72**	**48**

Table 48 is only for rear-end accidents of Houston intersections. A total of 54 rear-end accidents were recorded in Houston intersections. The number and percentage of rear-end accidents are listed in Table 48.

Table 48: Rear-End Accidents in Houston Intersections

Name	Period (month)	Number of Rear-End Accidents	Percentage of Rear-End Accidents	Rear-End Accidents by Severity Level		
				Fatal	Injury	PDO
Eldridge & West	36	0	0%	0	0	0
Atoscocita & Wilson	36	3	20%	0	2	1
Atoscicita & Will Clayton	36	4	40%	0	3	1
Westgreen & Kingsland	36	0	0%	0	0	0
Louetta & Jones	36	8	42.1%	0	3	5
Louetta & Kuykendahl	36	13	48.1%	0	11	2
Clay & Barker Cypress	36	12	54.5%	0	8	4
Barker Cypress & Little York	36	14	77.8%	0	10	4
Total	**-**	**54**	**45%**	**0**	**37**	**17**

Overall, 235 accidents occurred in all the study intersections in both Austin and Houston during the recorded time period, and there were 92 rear-end accidents among them (about 39 percent of total accidents).

7.2 Safety Benefits of Increased Storage Length

To determine the safety benefits of increased storage lengths of left-turn lanes, two approaches are employed:

1) Accident data analysis: comparing the accident rates at the study intersections with and without left-turn lane overflow problem

2) Simulation-based safety analysis: comparing safety surrogate measures for the intersections with left-turn lane overflow problem before and after extending the lengths of left-turn lanes

7.2.1 Accident Data Analysis

The accident rates at the studied intersections with and without left-turn lane overflow problem were calculated and compared, to estimate safety benefits of installing the left-turn lanes with sufficient lengths.

7.2.1.1 Intersections with Left-Turn Lane Overflow Problem

Adequate length of a left-turn lane is crucial in the design of left-turn lanes. When a left-turn lane is too short to accommodate all the turning vehicles, the left-turn vehicle will overflow to the adjacent through lane. The left-turn lane overflow problem will cause rear-end accidents between through and left-turn vehicles. For the 28 intersections that were studied in this research, the recorded traffic videos were carefully examined to identify the cycles with left-turn lane overflow problems. Based on the manually observing and counting, 6 intersections among the total of 28 intersections were diagnosed with left-turn lane overflow problem. These intersections are listed in Table 49 with the calculated percentage of cycles with left-turn lane overflow problems.

Table 49: Intersections with Left-Turn Lane Overflow Problems

Intersection	Subjective Approach		Percentage of Cycles with LT Lane Overflow
	Direction	Approximate Length of LT Lane* (ft)	
Eldridge & West (Houston)	WB	168	23.5%
Barker Cypress & Little York (Houston)	WB	160	12.3%
Mason & Kingsland (Houston)	NB	144	30.30%
Manchaca & Slaughter (Austin)	WB	186	62%
Lamar & 5th (Austin)	SB	TWLTL**	25%
Lamar & 6th (Austin)	NB	TWLTL**	31.25%

* Excluding taper length

** Two-Way Left-Turn Lane

7.2.1.2 Accident Rate Calculation

Traditionally, accident frequency (number of accidents per time period) was used to evaluate the safety of a location. However, the safety study only based on accident frequency cannot make a correct judgment. High accident frequency may be related to the high traffic volumes. As a result, the accident risk for each passing vehicle will be relatively low. Thus, it is better to use accident rate (accident risk per vehicle) instead of accident frequency to evaluate the safety of a location. According to Preston et al. (1998), accident rates are defined as the number of accidents per time period (such as a month) divided by the average amount of traffic passing in that time period. In this study, accident rates in intersections were calculated by using following equation:

$$\text{Accident Rate} = \frac{(\text{NA/P}) \times 1000000}{30 \times 24 \times \text{V}} \tag{27}$$

where:

NA: Number of accidents

P: Study time period (in months)
V: Traffic volume (vph) in the subject direction

Therefore, the accident rate can be stated in terms of "*accident per million vehicles*".

Since this study focuses on rear-end accidents, the real-end accident rates were calculated by assuming the NA in Equation (27) is the number of rear-end accidents. Therefore, the rear-end accident rates of all 21 studied intersections were calculated and the results are presented in Table 50.

Table 50: Accident Rates of Study Intersections

Name	Type of Left-Turn Lane	LT Lane Overflow Rate (%)	Number of Rear-End Accidents	Accident Rate (Acc./MVeh)
Anderson & Burnet (Austin)	TWLTL*	0%	2	0.10
Braker & Metric (Austin)	Exclusive Double	0%	2	0.10
Braker & Burnet (Austin)	Exclusive Single	0%	3	0.15
Lamar & 5th (Austin)	TWLTL*	25%	3	0.19
Brodie & Slaughter (Austin)	Exclusive Double	0%	1	0.03
Manchaca & Slaughter (Austin)	Exclusive Single	62%	6	0.23
Burnet & Justin (Austin)	TWLTL*	0%	0	0
Lamar & 45th (Austin)	TWLTL*	0%	5	0.33
Lamar & 38th (Austin)	TWLTL*	0%	1	0.11
Lamar & 6th (Austin)	TWLTL*	31.25%	3	0.14
Airport & M.L.K. (Austin)	TWLTL*	0%	6	0.30
Pleasant Valley & 7th (Austin)	Exclusive Single	0%	5	0.34
Congress & Slaughter (Austin)	Exclusive Single	0%	1	0.10
Eldridge & West (Houston)	Exclusive Single	23.5%	0	0
Atoscocita & Wilson (Houston)	Exclusive Single	0%	3	0.07
Atoscicita & Will Clayton (Houston)	Exclusive Single	0%	4	0.11
Westgreen & Kingsland (Houston)	Exclusive Single	0%	0	0
Louetta & Jones (Houston)	Exclusive-Shared**	0%	8	0.20
Louetta & Kuykendahl (Houston)	Exclusive Single	0%	13	0.39
Clay & Barker Cypress (Houston)	Exclusive Single	0%	12	0.42
Barker Cypress & Little York (Houston)	Exclusive Single	0%	14	0.60
Average	-	**6.4%**	**4**	**0.19**

* Two-Way Left-Turn Lane
** One lane is exclusive and the other is shared with through traffic

7.2.1.3 Accident Rate Comparison

According to the results in Table 50, the average rear-end accident rate for the 6 intersections with left-turn lane overflow problem and for the 15 intersections without left-turn lane overflow problem were calculated and compared. The results are presented in Figure 27. This figure indicates that the average rear-end accidents rate at the intersections with left-turn lane overflow problems are 35 percent higher than that at the intersections without left-turn lane overflow problem.

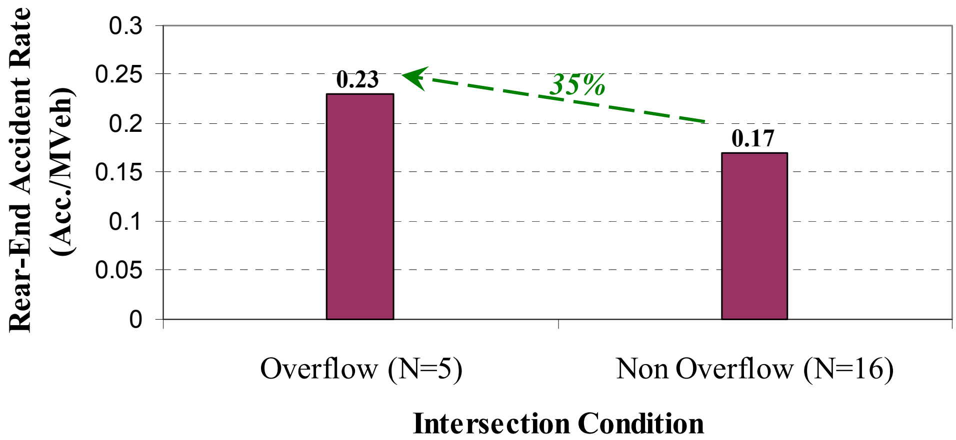


Figure 27: Average Accident Rates at Intersections with Left-Turn Lane Overflow and Non-Overflow Conditions

In Sum, the results of the accident rates comparison indicates that the left-turn lane overflow problem affects the safety of the intersection by increasing the rear-end accident risk.

7.2.2 Simulation-Based Safety Analysis

To further investigate the safety benefits of increasing the storage lengths of left-turn lanes at intersections with left-turn lane overflow problem, traffic simulation-based analysis was conducted by using VISSIM traffic model.

7.2.2.1 Simulation Scenarios

Two scenarios were constructed for each of the six intersections with left-turn lane overflow problem to evaluate the safety impacts of increasing the lengths of left-turn lanes:

- Scenario 1 - Before extending the left-turn lane (Existing Condition): The intersection with existing left-turn storage length, traffic volumes, signal timing, and other geometric conditions. This scenario was developed according to the data collected from the field. It reflected the real-world conditions in the field and served as the baseline scenario in this study.
- Scenario 2 - After extending the left-turn lane (Improved Condition): The intersection with existing traffic volumes and signal timing, but the storage length of left-turn lane for the subject direction was increased to eliminate the left-turn lane overflow problem. The required left-turn queue storage length was estimated according to the maximum left-turn queue length observed in the field.

These two scenarios are illustrated in Figure 28 and were simulated for each intersection which had left-turn lane overflow problem. To overcome the randomness in the simulation outputs, multiple simulation runs should be applied to each scenario. In this study 30 replications were applied to each scenario, resulting in a total of 360 simulation runs. The simulation period was set to 6000 seconds and the data was collected from 300 seconds to 6000 seconds after the simulation reach steady-state condition.

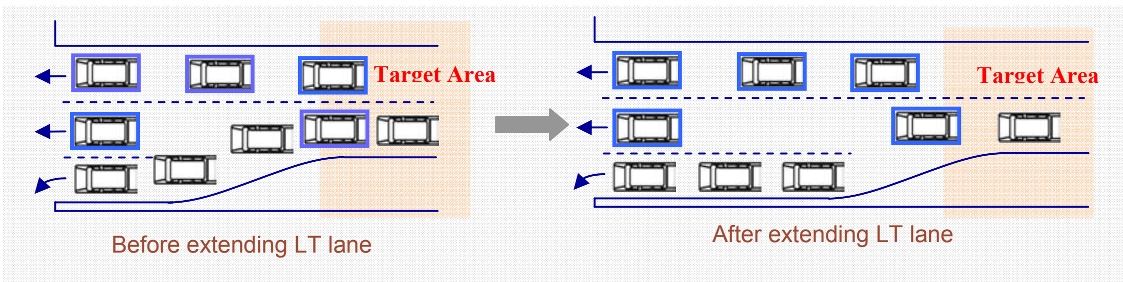


Figure 28: Simulation Scenarios

In addition, since the rear-end accidents due to insufficient lengths of left-turn lanes would occur in the upstream area of the adjacent through lane (see Figure 26), this portion of the intersection was selected as the target area for this study. Figure 28 illustrates the target areas in the intersections with and without left-turn lane extension. This target area is the focus of this study and only the simulation results on this area (in both scenarios) are used for safety analysis.

7.2.2.2 Model Calibration

To ensure the baseline scenario (scenario 1 - Existing Condition) can correctly represent the real-world traffic condition in the field, model calibration was conducted. In fact, model calibration is the most critical step in traffic simulation study and it provides the basis for further analysis based on simulation results. Same as in Chapter 6, the left-turn lane overflow rates and the left-turn lane blockage rates were used as the calibration measures. The left-turn lane overflow and blockage rates are defined as the percentage of total cycles in that the left-turn lane overflow problem and blockage problem were observed. The calibration results for the six simulated intersections are listed in Table 51. The table shows that the simulation results are quite close to the field observation, which indicates that the baseline model was well calibrated. In addition, the traffic videos collected at the studied intersections were reviewed and compared with the on-screen animations in VISSIM to further ensure the models' accuracy in the simulation operations.

Table 51: VISSIM Calibration Results for the Intersections with Left-Turn Lane Overflow Conditions

Intersection	LT Lane Overflow Rate (%)		LT Lane Blockage Rate (%)	
	Observed	VISSIM	Observed	VISSIM
Eldridge & West (Houston)	23.50	24.00	76.00	68.00
Barker Cypress & Little York (Houston)	12.30	7.50	98.00	96.00
Mason & Kingsland (Houston)	30.30	26.10	60.60	65.20
Manchaca & Slaughter (Austin)	62.00	63.04	23.05	26.10
Lamar & 5th (Austin)	25.00	30.43	0	0
Lamar & 6th (Austin)	31.25	34.78	0	0

7.2.2.3 Using Traffic Simulation for Safety Analysis

Simulation model itself cannot be directly applied for safety assessment because the occurrence of accidents cannot be simulated. However, according to a FHWA research (Gettman and Head, 2003) some safety surrogate measures can be derived from the results of microscopic traffic simulation to assess the safety of an intersection or a roadway segment. In this study, according to the identified accident risk and the feature of the VISSIM simulation model, following three safety surrogate measures were selected to measure the rear-end accident risks in the upstream of the adjacent through lane (see the target area illustrated in Figure 28).

- Maximum Deceleration (DC) of following vehicle
- Minimum Following Distance (FD) between two vehicles
- Minimum ratio of Following Distance to Speed of the following vehicle (FD/S)

These three safety surrogate measures are illustrated in Figure 29. This figure reveals that the bigger maximum deceleration, smaller minimum following distance and smaller minimum ratio of following distance to speed indicate higher rear-end accident risks. In general, they are all critical measures for assessing the risk of the rear-end accidents.

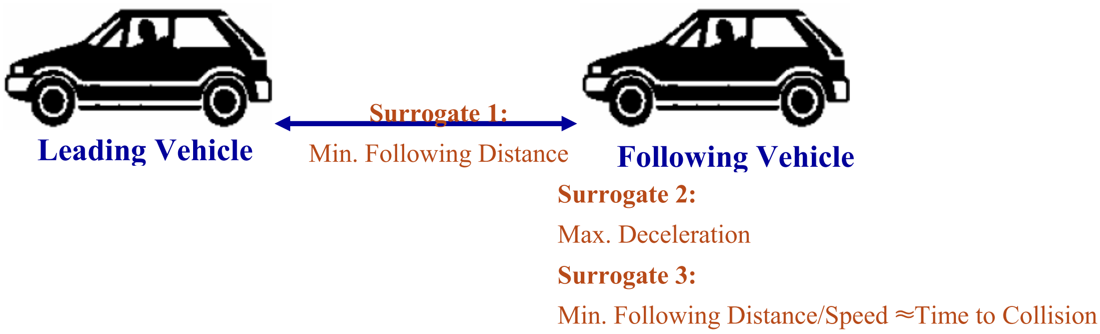


Figure 29: Illustrations of Safety Surrogate Measures

7.2.2.4 Simulation Results Analysis

Based on the simulation results, the three safety surrogate measures on the target area were derived for comparison as listed in Table 52, 53 and 54. Table 52 shows that, for all six intersections, the average maximum deceleration measured in the target area were reduced after eliminating the left-turn lane overflow problem by lengthening the left-turn lanes and the average reduction rate was 31 percent. This result indicates that extending left-turn lanes to eliminate the left-turn lane overflow problem will decrease the rear-end accident risk.

Table 52: Average of Maximum Deceleration (Dc) in the Upstream of Intersections with Left-Turn Lane Overflow Condition

Intersection	Average Maximum Deceleration, Dc (ft/s^2)		Percentage Reduction
	Scenario 1 (Existing Condition)	Scenario 2 (Improved Condition)	
Eldridge & West (Houston)	15.68	9.24	41%
Barker Cypress & Little York (Houston)	15.80	8.28	48%
Mason & Kingsland (Houston)	13.02	10.02	23%
Manchaca & Slaughter (Austin)	8.75	8.04	85%
Lamar & 5th (Austin)	15.79	7.36	53%
Lamar & 6th (Austin)	14.63	12.56	14%
Average	**13.95**	**9.25**	**31%**

Table 53 shows that the average minimum following distances (FD) in the target area was increased after increasing the length of left-turn lanes for all six intersections and the average increase rate was 64 percent. This result also indicates that increasing left-turn lanes at the intersections with left-turn lane overflow problems will significantly decrease the rear-end accident risk.

Table 53: **Average Minimum Following Distance (FD) in the Upstream of Intersections with Left-Turn Lane Overflow Condition**

Intersection	**Average Minimum Following Distance, FD (ft)**		**Percentage Increase**
	Scenario 1 (Existing Condition)	**Scenario 2 (Improved Condition)**	
Eldridge & West (Houston)	33.18	64.79	95%
Barker Cypress & Little York (Houston)	28.38	63.35	123%
Mason & Kingsland (Houston)	41.67	68.52	64%
Manchaca & Slaughter (Austin)	72.01	81.06	13%
Lamar & 5th (Austin)	36.32	57.54	58%
Lamar & 6th (Austin)	30.62	40.32	32%
Average	**40.36**	**62.60**	**64%**

Table 54 shows, in the target area of each intersection, the minimum ratio of following distance to the speed of following vehicle (FD/S) will increase after extending the left-turn lane lengths for all six intersections. The average percentage of increase was 75 percent. This result further confirmed that providing left-turn lane with sufficient length will significantly decrease the rear-end accident risk.

Table 54: Average Minimum Ratio of Following Distance to Speed (FD/S) in the Upstream of Intersections with Left-Turn Lane Overflow Condition

Intersection	Average Minimum Ratio of Following Distance to Speed, FD/S		Percentage Increase
	Scenario 1 (Existing Condition)	Scenario 2 (Improved Condition)	
Eldridge & West (Houston)	0.40	0.48	20%
Barker Cypress & Little York (Houston)	0.17	0.59	247%
Mason & Kingsland (Houston)	0.58	1.21	107%
Manchaca & Slaughter (Austin)	1.69	2.00	18%
Lamar & 5th (Austin)	2.34	2.75	18%
Lamar & 6th (Austin)	1.08	1.52	41%
Average	**1.04**	**1.43**	**75%**

Overall, by comparing the three surrogate measures from the results of simulation, it can be concluded that extending the lengths of left-turn lanes will significantly reduces the risk of rear-end accidents and improves the safety at the intersections with left-turn lane overflow problems.

7.2.3 Benefit and Cost Estimation

Figure 27 showed that the average rear-end accident rate at the intersections with left-turn lane overflow problem is 35 percent higher than that at the intersections without left-turn lane overflow problem. In other words, eliminating the overflow problem by extending the length of left-turn lane can averagely reduce rear-end accident rate by 26 percent. Therefore, the expected rear-end accident reduction can be estimated by multiplying the number of rear-end accidents (before extending) by the average percentage accident rate reduction (26%). Then, the annual safety benefits of extending the length of a left-turn lane can be estimated by multiplying the estimated rear-end accident reduction by the average applicable cost of rear-end accidents. The estimation of annual safety benefits of extending the length of a left-turn lane can be expressed by Equation (28).

$$\text{Annual Safety Benefit (\$)} = PR \times NA_{RE} \times C_{RE} \tag{28}$$

where:

PR = Percentage reduction in rear-end accidents (26%, based on Figure 27)

NA_{RE} = Average number of accidents per year, before extending the left-turn lane

C_{RE} = Average cost of rear-end accidents ($)

Campbell and Knapp (2005) estimated the average accident cost per vehicle in order to develop a new crash severity ranking procedures. According to their study, the average cost of rear-end accidents (C_{RE}) is $13,000. On the other hand, the cost of extending the lengths of left-turn lanes is estimated by using Equation (29).

$$\text{Cost (\$)} = (RL_{LT} - EL_{LT}) \times C_{LT} \tag{29}$$

where:

RL_{LT} = Required length of left-turn lane in order to eliminate overflow problem (ft) (see *Note*)

EL_{LT} = Existing length of left-turn lane (ft)

C_{LT} = Cost of extending left-turn lane per foot ($)

This cost also can also be converted to annual cost, considering interest rate and life cycle of the left-turn lane.

7.3 Summary

In this chapter, historical accident data related to the studied intersections was collected and analyzed. Then two methods were employed for intersection safety analysis: 1) accident data analysis, and 2) simulation-based safety analysis. The followings are the key findings from the safety analysis:

- The average rear-end accident at the intersections with left-turn lane overflow problem was 35 percent higher than that at the intersections without left-turn lane overflow problem.
- After extending the lengths of the left-turn lanes to eliminate the left-turn lane overflow problem at the study intersections, all of the safety surrogate measures derived from the traffic simulation results, changed significantly in a direction that indicated the reduction of rear-end accident risk at those intersections.

In summary, the results of this chapter concluded that extending left-turn lanes to eliminate the left-turn lane overflow problem significantly improves intersection safety by decreasing the rear-end accident risk.

CHAPTER 8

CRITERIA FOR MULTIPLE LEFT-TURN LANES INSTALLATION

This Chapter is to develop criteria for multiple left-turn lanes installation. For this purpose, literatures on warrants for multiple left-turn lanes and the operational characteristics of multiple left-turn lanes are reviewed and synthesized at first. Then, criteria for installing multiple left-turn lanes are developed by considering the warrants in following four categories: 1) capacity and volume based, 2) left-turn queue length based, 3) safety based, and 4) geometric condition based.

8.1 Literature Review

This literature review includes two parts: 1) summarization of the existing guidelines and current practices on dual and triple left-turn lanes installation, and 2) summarization of findings about the operational characteristics of multiple left- turn lanes.

8.1.1 Existing Guidelines and Current Practices

The existing warrants on multiple left-turn lanes installation were developed from different aspects, including capacity and volume analysis, safety analysis, and geometric condition analysis. These warrants are used by the transportation agencies in different states (see

Table 55). After a brief introduction of different types of warrants, the existing warrants on multiple left-turn lane installation are summarized in Table 55.

Capacity and Volume Based Warrants

Capacity and volume based warrants have been widely used for determining the installation of dual left-turn lanes or triple left-turn lanes. Some of the states like Louisiana, Maine, Maryland and North Dakota use capacity analysis for determining the upgrade of single left-turn lanes to dual left-turn lanes. In Texas, TxDOT Roadway Design Manual states “for major signalized intersections where high peak hour left-turn volumes are expected dual left-turn lanes should be considered”, but it did not give any specific volume criteria. Some of the states follow the rule of thumb method, which use a specific threshold left-turn volume to determine when to install multiple left-turn lanes. For example, California, Nevada and South Carolina States use the criterion of left-turn volumes over 300 vph; Arkansas and Kansas States use the criterion of left-turn volumes over 400 vph; and Wisconsin state upgrades the single left-turn lanes to dual when the left-turning volume exceeds 250-300 vph. There are relative fewer guidelines for installation of triple left-turn lanes compared to dual left-turn lanes. Only Nevada State uses a left-turn volume over 600 vph as the criteria for upgrading dual left-turn lane to triple left-turn lane.

Safety Based Guidelines

In Ackeret (1994), it is pointed out that it is inappropriate for installing multiple left-turn lanes when there is a potential for a higher number of pedestrian-vehicle conflicts.

Geometry Condition Based Guidelines

Some special geometric conditions are inappropriate for multiple left-turn lanes installation. Followings are three situations in which multiple left-turn lanes installation will not be installed:

a) When left-turn vehicles are not anticipated to queue uniformly within the provided left-turn lanes due to downstream conditions, it's not appropriate to install multiple left-turn lanes. As illustrated in Figure 30, there is a trip attraction (e.g., shopping mall) nearby the intersection and on the one side of the downstream through lanes. Since most of the left-turn vehicles will go to that attraction, these vehicles tend to use the outer left-turn lane. As a result, the queue in outer left-turn lane will block the entrance points of the inner left-turn lane and the capacity provided by the multiple left-turn lanes cannot be efficiently used.

b) When the geometric conditions that obscure, or result in, confusing pavement markings within the intersection, it is inappropriate to install multiple left-turn lanes.

c) If the number of receiving lanes is less than the number of left-turn lanes that will be provided, it is inappropriate to directly install multiple left-turn lanes (See Figure 31). Sufficient number of receive lane with enough length must be built before the installation of multiple left-turn lanes (See Figure 31).

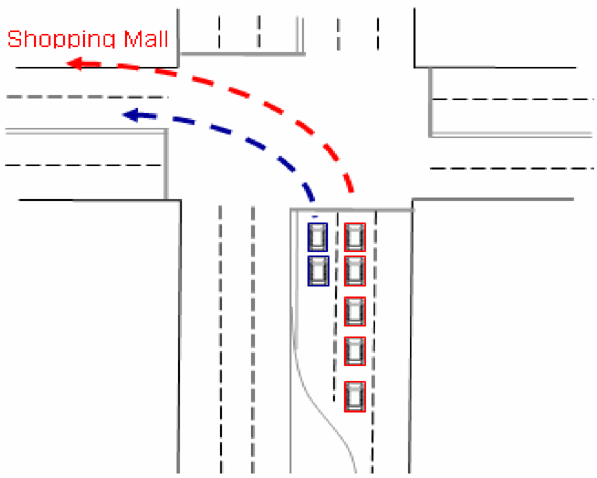
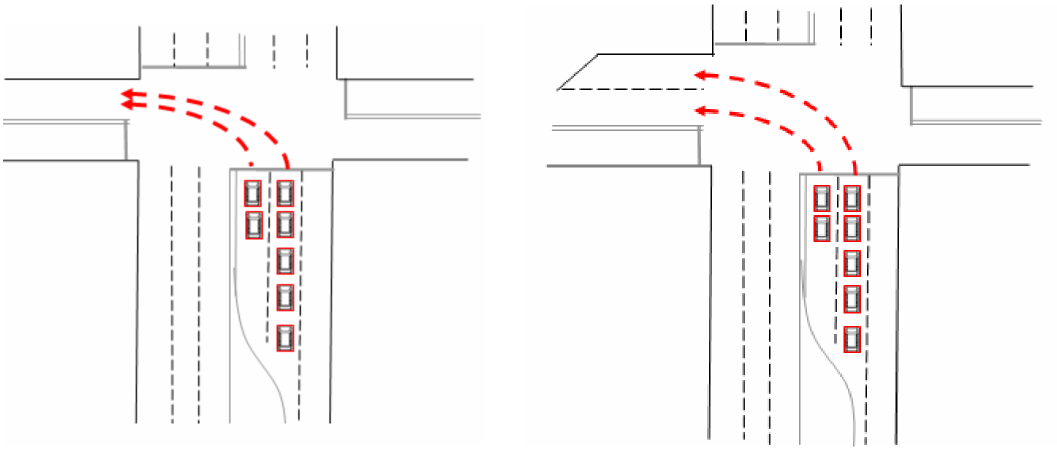


Figure 30: Unbalanced Left-Turn Lane Utilization



(a) (b)

Figure 31: Insufficient Receiving Lanes for Left-Turn Movements (a) and After Extra Lane Installation (b)

8.1.2 Operational Characteristics of Multiple Left- Turn Lanes

The literature review investigated the operational characteristics of multiple left-turn lanes from different aspects, including left-turn capacity analysis, safety, traffic control, lane utilization issues. Followings are the introduction of some major findings in literatures. The summarization of these findings was provided in Table 56.

Left- Turn Lane Capacity Analysis

To analyze the capacity of left-turn lanes, the left-turn lane saturation flow rate and left-turn adjustment factor needs to be estimated. Following are some findings about these two factors in literatures:

- Saturation Flow Rate: analysis conducted by ITE Technical Council Committee suggested that the saturation flow rate of 1950 pcphgpl should be used for double left-turn lanes, rather than 1656 pcphgpl as suggested by HCM. As for triple left-turn lanes, a research conducted by Leondard (1994) suggested that 1930 pcphgpl as the saturation flow rate rather than 1800 pcphgpl recommended by HCM. The saturation flow rate can also be estimated based on average vehicle headway. In Ray (1965), it is suggested that the average vehicle headway is between 2.6 to 2.9 seconds in the inner left-turn lane and is between 2.8 to 3.5 seconds in the outer lane.

- Left-turn Adjustment Factor: Left-turn factor (f_{LT}) is defined as the ratio of saturation flow rate per left-turn lane to saturation flow rate per through lane. To estimate the capacity of left-turn lanes, ITE Technical Council Committee (reference) suggested that a left-turn factor of 1.0 should be used for certain intersection geometric configurations rather than the factor of 0.92 suggested by 1985 HCM.

Traffic Control

Following guidelines has been used for the traffic controls for multiple left-turn lanes: 1) special markings to delineate the inner and outer turning vehicle paths; 2) turn lane designations indicated by overhead signs, sometimes supplemented by ground mounted signs (Robert, 1995), 3) Using protected only signal phasing for dual left-turn or triple left-turn lanes (Qureshi et al. 2004; Tarrall et al. 1998), and 4) Results of a interview conducted to 25 transportation agencies showed that the majority of the agencies use the leading protected green phasing with no conflicting pedestrians allowed for the signal control of multiple left-turn lanes. (Stokes, 1995)

Lane Utilization

Kikuchi et al. (2004) found that when the total left-turn volume is small, the drivers will choose the lane that allows him/her the best access to the desired lane downstream. When the total left-turn volume becomes large, the drivers become concerned about the possibility of not being able to clear the intersection in one cycle. Thus, each driver chooses the lane that has the shorter queue length. As a result, the queue lengths become nearly equal between the two lanes. Stokes (1995) found that each lane is used almost equally when both lanes are reserved exclusively for left-turn. The split of left-turn traffic between inner and outer lane was that 51.3 percent for the inner lane and 48.7 percent for the outer lane.

Table 55: Summary of Warrants for Multiple Left-Turn Lanes

Criterion	Warrant or Guideline in Different States			Reference
Capacity/ Volume	Capacity analysis should be used to determine the set of conditions.		Arkansas Kansas Louisiana Maine North Dakota Maryland	Qureshi et al. (2004)
	For major signalized intersections where high peak hour left-turn volumes are expected dual left-turn lanes should be considered.		Texas	• TXDOT Design Division's Highway Design Manual • Qureshi et al. (2004)
	LT Volume	> 300 vph (dual LT lanes)	South Carolina California etc	• Qureshi et al. (2004) • Stokes (1995) • Courage et al. (2002)
		> 400 vph (dual LT lanes)	Arkansas Kansas	Qureshi et al. (2004)
		> 250~300 vph (dual LT lanes)	Wisconsin	Qureshi et al. (2004)
		> 300 vph (dual LT lanes) > 600 vph (triple LT lanes)	Nevada etc.	Qureshi et al. (2004)
Safety	It is **inappropriat**e for installing multiple LT lane installations **w**hen there is a potential for a higher number of pedestrian-vehicle conflict.			Ackeret (1994)
Geometry	a) It is **inappropriate** for installing multiple LT lane installations when LT vehicles are not anticipated to queue uniformly within the provided left-turn lanes due to downstream conditions (See Figure 30 for example). b) It is **inappropriate** for installing multiple LT lane installations **w**hen conditions exist that obscure, or result in, confusing pavement markings within the intersection.			Ackeret (1994)
	c) The number of receiving lanes should not be less than the LT lanes (See Figure 31).			

Table 56: Summary of Operational Characteristics of Multiple Left Lanes

Criterion		Dual LT	Triple LT	Reference
Capacity analysis	Left-turn Adjustment Factor (f_{LT})	0.92		HCM (1985)
		1.00	1.00	ITE Technical Council Committee
	Saturation Flow Rate	1656 pcphpl	1800 pcphpl	HCM (1985) Leonard (1994)
		1950 pcphpl	1930 pcphpl	Stokes (1995) Leonard (1994)
	Headway	The average headway of vehicle is 2.6 to 2.9 seconds in the inside lane and 2.8 to 3.5 seconds in the outside lane. The inside lane headway compares favorably to a single lane headway.		Ray (1965)
Safety		Leading protected green phasing with on conflicting pedestrians allowed;		Stokes (1995)
Traffic Control		1) Special markings to delineate the common limit between the inner and outer turning vehicle paths 2) Turn lane designations indicated by overhead signs, sometimes supplemented by ground mounted signs 3) Use protected only signal phasing for multiple LT lanes		Stokes (1995) Qureshi et al. (2004). Tarrall et al. (1998)
Lane Utilization		1) When the total left-turn volume is small, the choosing the lane that allows the driver the best access to the desired lane downstream. 2) When the total left-turn volume becomes large, the queue lengths become nearly equal between the two lanes. 3) The split of LT traffic between inside and outside lane was 51.3 percent used the inside lane and 48.7 percent used the outside lane.		Ray (1965)

8.2 Development of Warrants for Multiple Left-turn Lanes

In this study, warrants for multiple left-turn lanes installation will be developed by analyzing both intersection delay and the impacts of left-turn queue length. As a result, two types of warrants were developed: 1) capacity and volume warrants and 2) left-turn queue length based warrants. By combining the developed warrants with the exiting warrants, a decision-making flowchart for installing multiple left-turn lanes was developed. Both intersection operational and safety impacts were considered in the developed warrants for multiple left-turn lanes.

8.2.1 Capacity and Volume Warrants

The capacity and volume based warrants has been widely used for multiple left-turn lanes installation. However, most of them just use a constant left-turn volume threshold as a warrant for multiple left-turn lanes installation and different states choose different left-turn volume thresholds (see Table 55). These warrants are developed based on engineering judgment instead of systematic intersection performance analysis. In this study, capacity and volume based warrants were developed based on intersection delay analysis. In addition, the multiple left-turn lanes installation was considered together with the traffic signal controls at the intersection since the installation of multiple left-turn lanes reduces the required green time for left-turn movements.

8.2.1.1 Development of Capacity and Volume Warrants Based on Intersection Delay Analysis

At intersections with single left-turn lane, when the volume of left-turn vehicles increases to a critical level, intersection delay will increase quickly. Under this condition, the single left-turn lane cannot meet the operation efficiency requirements and needs to be upgraded to multiple left-turn lanes. Hence, the development of the capacity and volume based criteria for multiple left-turn installation is to find a critical left-turn volume, beyond which the traffic delay for the single left-turn lane will increase quickly and will be significantly greater than the delay for the multiple left-turn lanes. Note that, after the installation of multiple left-turn lane, the intersection signal timing also needs to be adjusted according to the increased capacity of multiple left-turn lanes.

In this study, the capacity and volume warrants were developed based on intersection delay analysis. The relationship between left-turn volume and intersection delay was theoretically derived under single, double and triple left-turn lane conditions. Then, by comparing the intersection delay under different conditions, the critical left-turn volumes for upgrading single left-turn lane to double left-turn lane and for upgrading double left-turn lane to triple left-turn lane can be identified.

Modeling Intersection Delay with the Consideration of Signal Timing Updates

There are lots of exiting intersection delay models, such as the delay model proposed by Roess et al. (2003) (see Equation (D-1) in Appendix D), can be used to estimate the intersection delay under different traffic demand conditions. However, in these existing models, the green time allocated to the subject left-turn movement is fixed. Actually, when the left-turn volume increases, the original signal timing did not fit the new traffic demand and need to be updated at first. Therefore, to count the pure delay reduction caused by upgrading single left-turn lane to multiple left-turn lane, the relationship between the intersection delay and left-turn volume should be derived under the assumption that the signal timing are keep updating according to the increase of left-turn volume. In other words, the effective green time, i.e. $g_{LT,}$ in Equation (D-1) should be a function of left-turn traffic volume. According to Roess et al. (2003), the effective green time should be reallocated according to the ratios of traffic volume and saturation flow rate (V/S ratios) in competing directions. In this study, for simplification purpose, it is assumed that 1) the intersection cycle signal length is a constant, i.e. C, 2) left-turn signal control is protected only control, 3) the green time reallocation are just between the subject left turn movement (which is the critical left-turn movement) and its competing through movement (which is the critical through traffic volume), and 4) the total green timing for these two movements counts for λ percentage of cycle length . These four assumptions are illustrated in the scenario in Figure 32 and signal phase diagram in Figure 33.

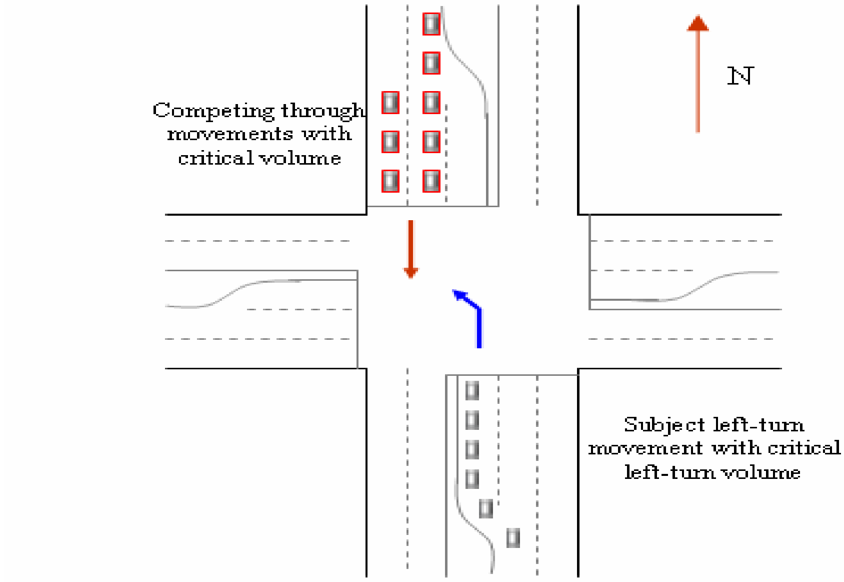


Figure 32: Assumed Intersection Scenario

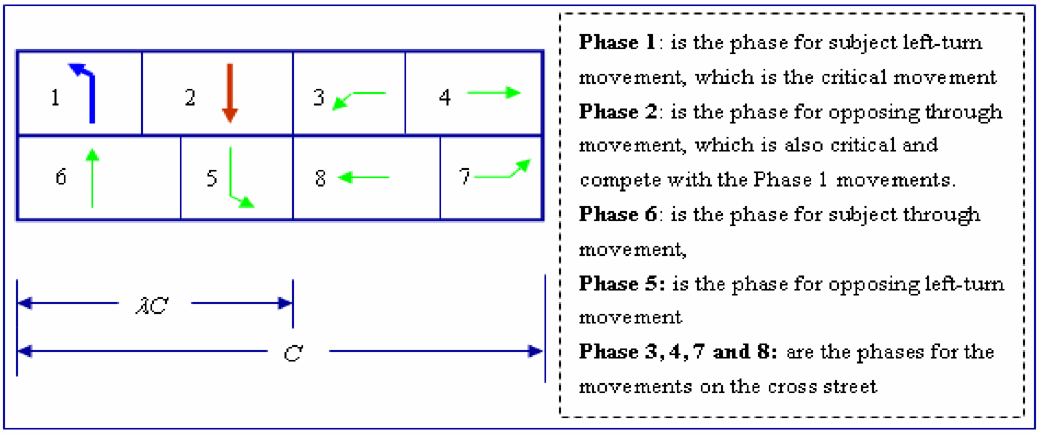


Figure 33: Signal Phase Diagram

According to assumption 3, only the delay for the subject left-turn movement and its competing through movement will change as the signal timing changes with the increase of subject left-turn volume. Therefore, to simply this problem, only the average delay of these two movements (subject left-turn and competing through) is estimated under different subject left-turn volume conditions. The detail derivation of the average intersection delay of these two movements as a function of subject left-turn volume was given in Appendix D. The derived functions for the average intersection delay of these two movements under the single left-turn lane, double left-turn lanes and triple left-turn lanes conditions were given in Equation (D-21), Equation (D-24) and Equation (D-25) in Appendix D, respectively. Then, the average delay vs. left-turn volume under different number of left-turn lanes conditions can be plotted (see Figure 34) by assuming the value of following variables in Equation (D-21), (D-24) and (D-25):

- The signal cycle length: $C = 120s$
- The left-turn saturation flow rate: $S = 1650 veh/h$
- The opposing through saturation flow rate: $S_T^o = 1800 veh/h$
- The total phase number: $N = 4$
- The total lost time for each phase: $t_L = 3s$
- The volume of the opposing through vehicles: $V_T^o = 400 veh/s$
- the total green timing for these two movements counts for half cycle length: λ=50%

According to Equation (D-15) in the Appendix D, when the *v/c* ratio is equal to 1 under signal left-turn condition, the left-turn volume can be estimated as follows:

$$V_{LT}^C = S_{LT}[\lambda(1 - \frac{Nt_L}{C}) - \frac{V_T^o}{S_T^o}] = 375 \text{ veh/h} \tag{30}$$

where:

S_{LT} : Saturation flow rate of the left-turn vehicles

λ : Percentage of the total green timing for the subjective left-tune and its competing through movements counts for the whole cycle

C : Signal cycle length

N : Total number of phases in a cycle

V_T^o : Volume of the opposing through traffic

S_T^o : Saturation flow rate of the opposing through traffic

From Figure 34, it can be seen that, when left-turn volume beyond this point (point 1), the average delay for the single left-turn lane will increase quickly and will be significantly greater than the delay for the double left-turn lanes. Therefore, this left turn volume, i.e. V_{LT}^{C}, is the critical volume for upgrading single left-turn lane to double left-turn lane.

Similarly, the critical left-turn volume for upgrading double left-turn lane to triple left-turn lane, i.e. V_{LT}^{2C}, is the left-turn volume when the v/c ratio for the double left-turn condition is equal to 1, which can be derived as follows

$$V_{LT}^{2C} = 2S_{LT}[\lambda(1-\frac{Nt_L}{C}) - \frac{V_T^o}{S_T^o}] = 750 \text{ veh/h} \tag{31}$$

In addition, when opposing through traffic, V_T^o, is very heavy, the critical left-turn volumes, V_{LT}^{C} and V_{LT}^{2C}, estimated by using equations (30) and (31) will become very small. It will cause that multiple left-turn lane be installed due to the heavy traffic in the opposing through direction, which is unreasonable. Therefore, to prevent this problem, the low boundary values need to be set for the critical left-turn volumes. According to the literature review results in Table 55, 300 veh/h and 600 veh/h was selected as the lower boundary for V_{LT}^{C} and V_{LT}^{2C}, respectively. Therefore, the finally critical left turn volumes for installing double left-turn lane and triple left-turn lane can be estimated by Equations (32) and (33), respectively.

$$V_{LT}^{C*} = \max\{S_{LT}[\lambda(1-\frac{Nt_L}{C}) - \frac{V_T^o}{S_T^o}], 300veh/h\} \tag{32}$$

$$V_{LT}^{2C*} = \max\{2S_{LT}[\lambda(1-\frac{Nt_L}{C}) - \frac{V_T^o}{S_T^o}], 600veh/h\} \tag{33}$$

Actually, using these new criteria of left-turn volumes given in Equation (32) and (33) has its practical meaning. It means that, when left-turn volume high than the older left-turn volumes criteria, i.e. 300 veh/h and 600 veh/h, multiple left-turn should not be provided immediately. Field engineers should check the capacity request for the left-turn movements at first and if the left-turn volume less than the critical value V_{LT}^{C} or V_{LT}^{2C} given in Equation (30) and (31), it is better to re-split the green time between the left-turn movement and its opposing through movement instead of installing multiple left-turn lanes to reduce the left-turn delay.

8.2.1.2 Developed Capacity and Volume Warrants

According to Figure 34, the final critical left turn volumes for installing double left-turn lane and triple left-turn lane, i.e. critical point 1 V_{LT}^{C} and critical point 2 V_{LT}^{2C}, can be estimated by Equations (32) and (33), respectively. Based on these critical left-turn volumes, the capacity and volume warrants for installing multiple left-turn lanes was developed, as shown in the decision-making flowchart in Figure 35. According to this flowchart, when the left-turn volume is greater than V_{LT}^{C*}, the single left-turn lane should be upgraded to double left-turn lanes. Then, the signal timing should be adjusted according to the capacity of double left-turn lanes. When the left-turn volume is greater than V_{LT}^{2C*}, the triple left-turn lanes should be provided.

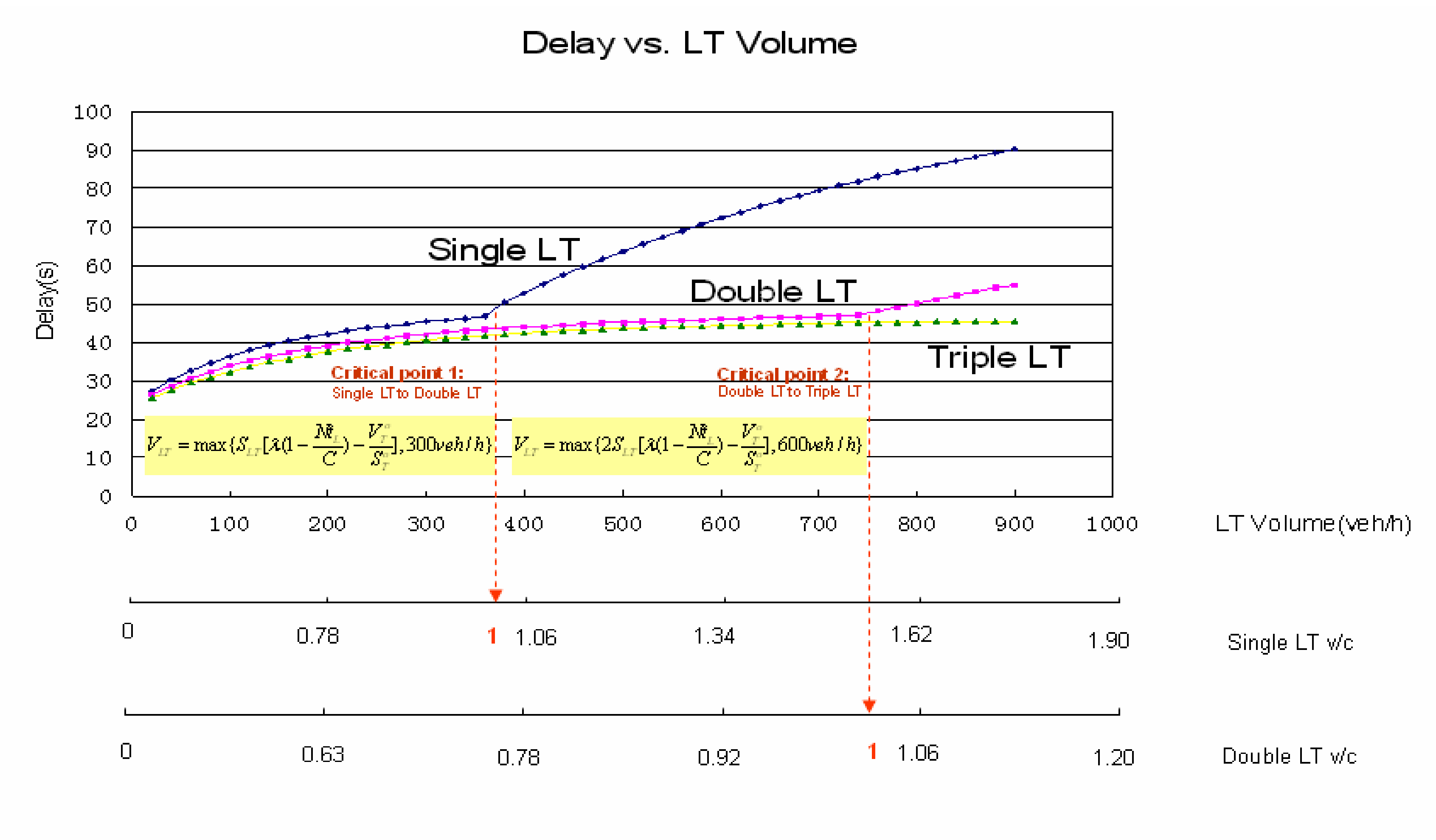


Figure 34: The Average Delay vs. Left-Turn Volume

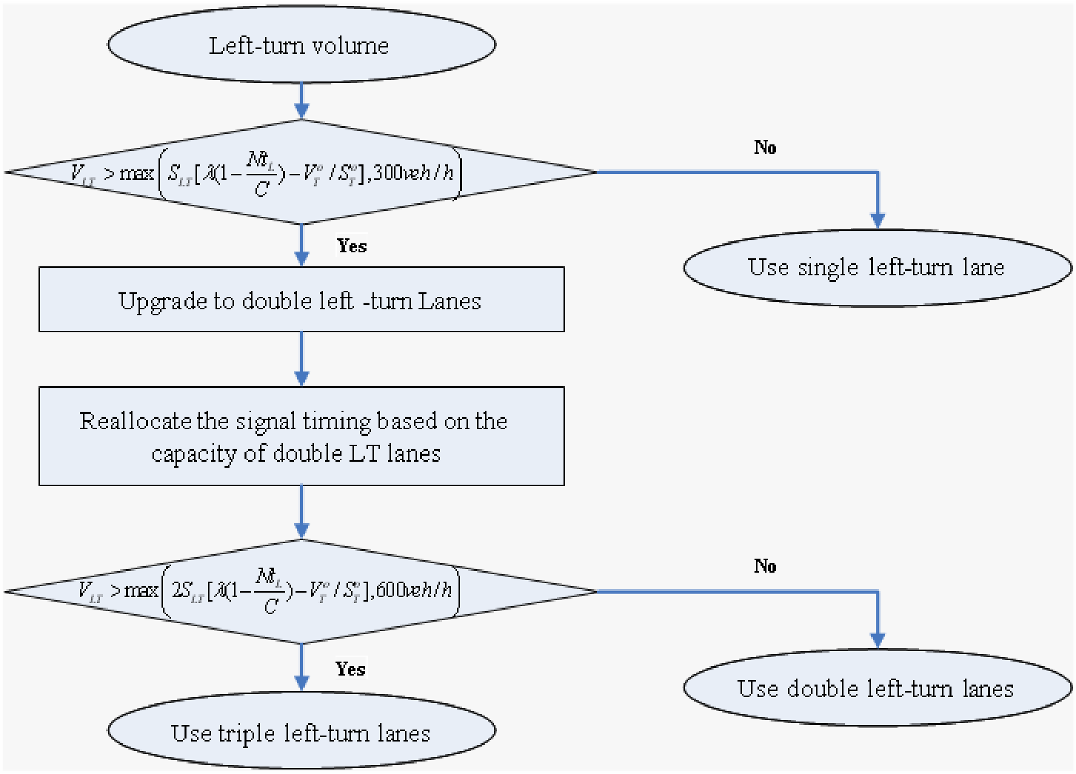


Figure 35: Flowchart for Volume and Capacity Based Warrants

8.2.2 Left-Turn Queue Length Based Warrants

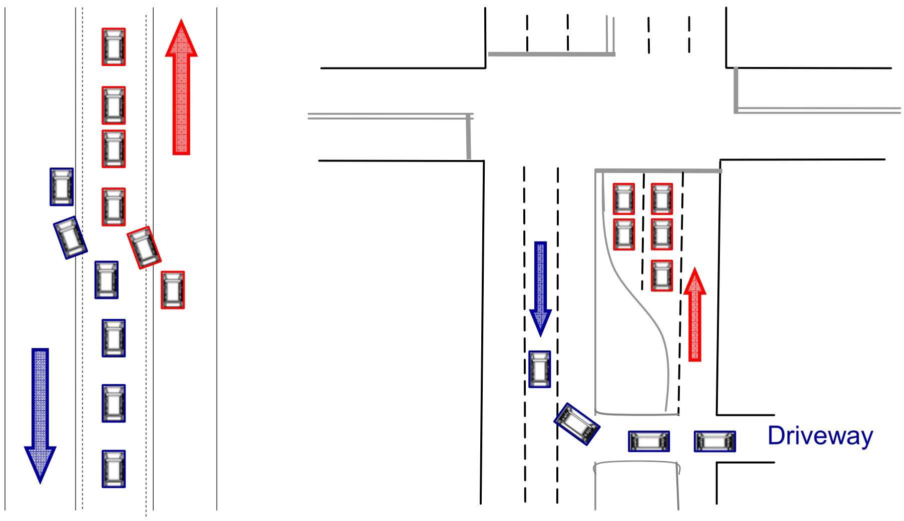
Besides the volume and capacity warrants, the research team found that the queue length of the left-turn lane is another import factor that needs to be considered for installing multiple left-turn lanes. It is because long left-turn queue will cause left-turn vehicle overflow to the through lane and multiple left-turn lane can be provided to prevent this problem.

8.2.2.1 Development of the Queue Length Based Warrants

The queue length based warrants was developed by considering two problems in left-turn operations: 1) left-turn lane overflow problem, and 2) unbalanced left-turn queue problem.

Left-Turn Lane Overflow Problem

If the left-turn queue length is greater than the storage length of the left-turn lane, the left-turn vehicles will overflow to the adjacent through lane and affect the movement of the through vehicles. In this case, not only the efficiency but also the safety of the intersection will be affected. There are two direct solutions to this problem. The first is to increase the length of the left-turn lane and the second is to upgrade the existing single left-turn to multiple left-turn lanes. However, under certain conditions, it is not feasible to increase the storage length of the left-turn lane. As examples, Figure 36 presents two specific situations. Figure 36-a shows a two-way left-turn lane (TWLT) condition, in which the central lane services as the left-turn lane for both directions and the total length of the TWLT are fixed (the link length). Figure 36-b shows another special scenario, in which there's a driveway nearby the left-turn approach. Increasing the length of the left-turn lane will block the entrance to the driveway for the opposing traffics. Therefore, under these specific conditions, we have to choose the second solution to the left-turn lane overflow problem: upgrading the existing single left-turn to multiple left-turn lanes.



(a) **(b)**

Figure 36: Two-Way Left-Turn Lane (a), and A Parking Lot Nearby (b)

To develop the queue length based warrants for preventing overflow problem, an important step is to determine if an intersection has overflow problem or not. Thus, the left-turn queues length need to be estimated at first. Many methods have been developed to estimate the left-turn queue length. In this study, we recommend to use a model developed in Chapter 5 for estimating left-turn queue length. Then, by comparing the estimated queue length with the left-turn lane storage length, it can be determined whether an intersection has the overflow problem or not.

Unbalanced Queue Problem

Although extending the length of single left-turn lane can solve the left-turn overflow problem in some case, the long left turn queue in a single Left-turn lane will cause another potential problem: unbalanced queue problem. This problem was illustrated in Figure 37. In the situation in Figure 37, the queue length of the left-turn vehicles is much longer than the queue in the adjacent through lane. In this case, some left-turn vehicles might take the adjacent through lane to approach the intersection and then try to squeeze into the left-turn queue from the adjacent through lane (it is referred to as left-turn-squeeze-in problem). These vehicles will block the following through vehicles and cause some potential safety problems. In this situation, multiple left-turn lane need to be provided to reduce the left-turn queue length in a single left-turn lane.

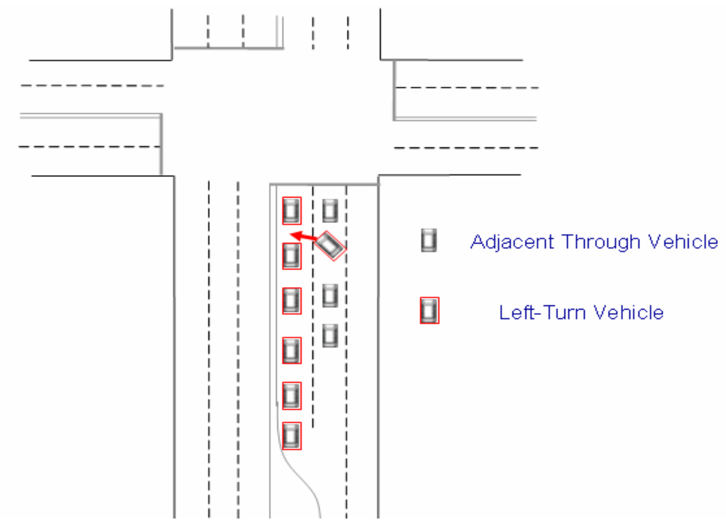


Figure 37: Unbalanced Queue Problem

8.2.2.2 Developed Queue Length Based Warrants

Based on the discussion in Part 8.2.2.1, queue length based warrants for multiple left-turn lanes were developed, which can be expressed by the flowchart in Figure 38. In this flowchart, the first step is to check whether there are left-turn lane overflow problems. It can be done by comparing the estimated left-turn queue lengths (Chapter 5) with the left-turn lane storage lengths. If the left-turn queue length is greater than the left-turn storage length, overflow problem exists. Then, we should further check if it is feasible to increase left-turn lane length. If there are some geometry limitations in increasing the left-turn lane length (such as the situations in Figure 36), multiple left-turn lanes should be provided to prevent the overflow problem. On the other hand, if such limitations do not exist, we should consider if unbalanced queue problem exists or not after increasing the length of signal left-turn lane. Based on the observation from the studied intersections, it is found that when the left-turn queue is 6 vehicles longer than the queue in the adjacent through lane, the left-turn-squeeze-in problem occurred. Therefore, the following criterion was set to determine whether the unbalanced queue problem exists or not

$$Q_{LT} - Q_{TH} > 150\,ft \tag{34}$$

where:

Q_{LT} : Left-turn queue length

Q_{TH} : Length of the queue in the adjacent through lane

150ft: the storage length for 6 vehicles by assuming 25 ft per vehicle

If this criterion is met, the multiple left-turn lane need to be installed. Otherwise, it is better to just increase the length of signal left-turn lane.

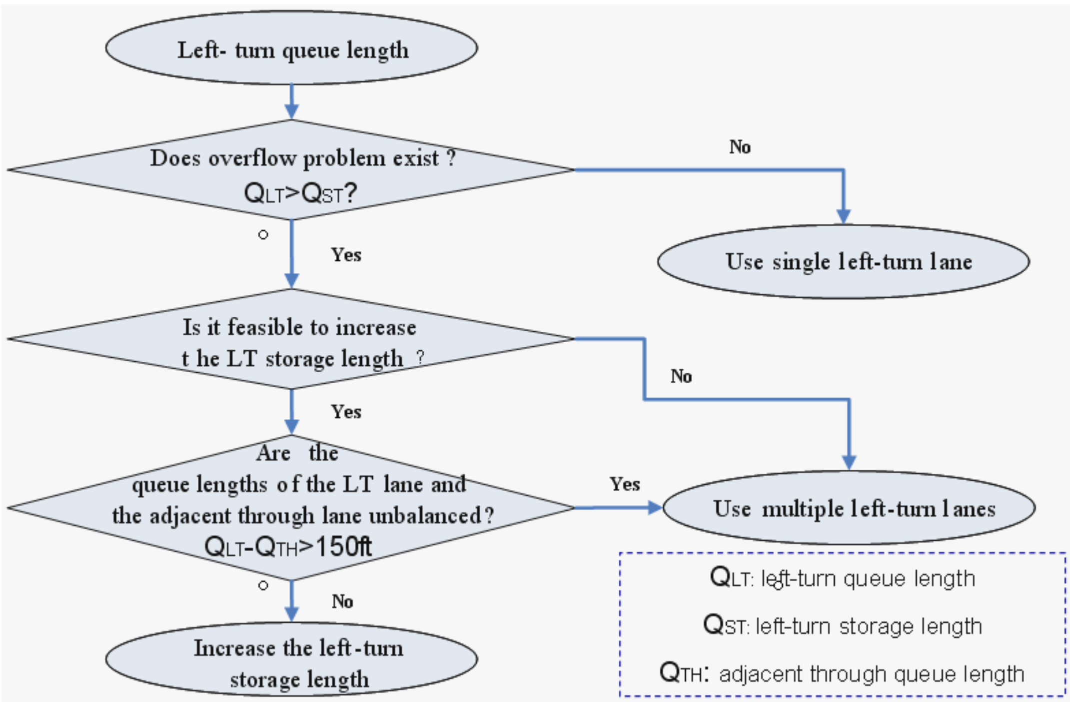


Figure 38: Flowchart for Queue Length Based Warrants

8.3 Decision-Making Flowchart for Installing Multiple Left-Turn Lanes

After development of the volume and capacity warrants and queue length based warrants for multiple left-turn lanes, the existing safety and geometric warrants from the literatures given in Table 55 were also need to be considered in the installation of multiple left-turn lanes. By combining all these warrants, a decision-making flowchart for installing multiple left-turn lanes was developed, as shown in Figure 39. In this flowchart, the first step is to check the volume and capacity warrants given in Figure 35, and the queue length based warrants given in Figure 38. If either of these two warrants is met, we should further check the existing safety warrants and geometric warrants given in Table 55. If the warrants in these two categories are also satisfied, multiple left-turn lanes should be installed at the study intersection.

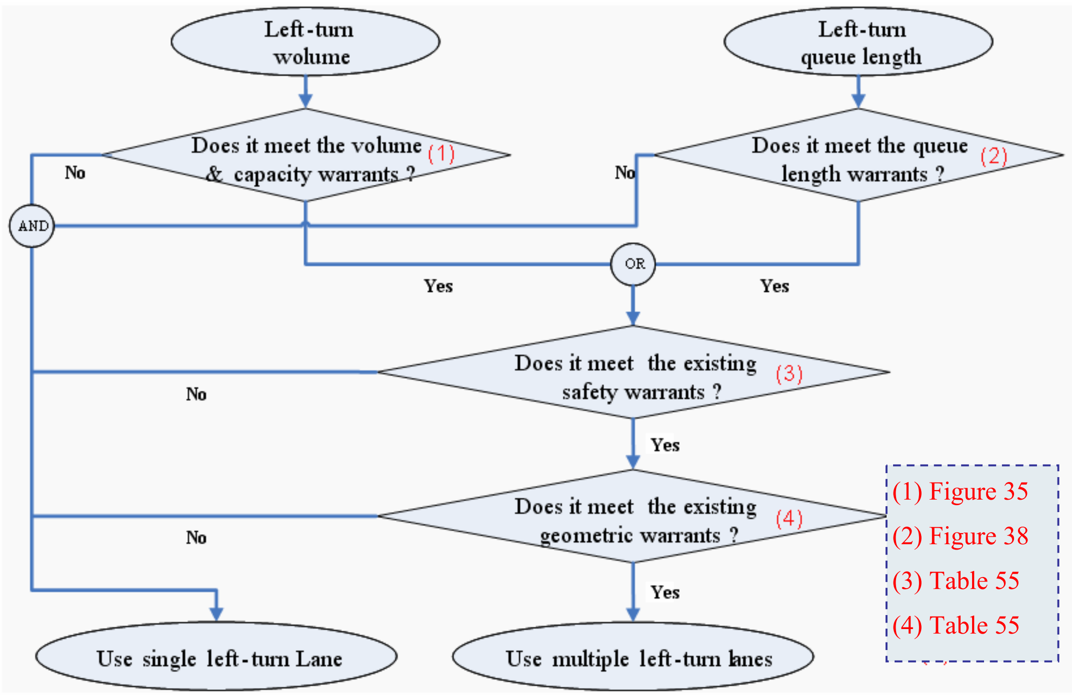


Figure 39: Decision-Making Flowchart for Installing Multiple Left-Turn Lanes

8.4 Summary

In this chapter, literatures on warrants for multiple left-turn lanes and the operational characteristics of multiple left-turn lanes were reviewed and synthesized. Criteria for installing multiple left-turn lanes were developed by considering the warrants in following four categories:

1) capacity and volume based, 2) left-turn queue length based, 3) safety based, and 4) geometric condition based. Among these warrants, we developed the capacity and volume based warrants, and the left-turn queue length based warrants based on intersection delay and safety analysis. Finally, a decision-making flowchart for installing multiple left-turn lanes was developed by combining the developed warrants with the existing warrants/guidelines.

CHAPTER 9

OTHER ELEMENTS RELATED TO LEFT-TURN LANES

In this chapter, two important issues related to left-turn lane design and operation are investigated: (1) Left-turn bay taper length estimation, and (2) the impacts of signal phasing sequence on left-turn operation.

9.1 Bay Taper

Bay taper is a part of deceleration length in left-turn lanes (Figure 40). Bay taper is a reversing curve along the left edge of the traveled way which directs vehicles to leave the through traffic lane and enter left-turn lane with minimum braking. Also it provides enough length for vehicles to decelerate and join the end of left-turn queue.

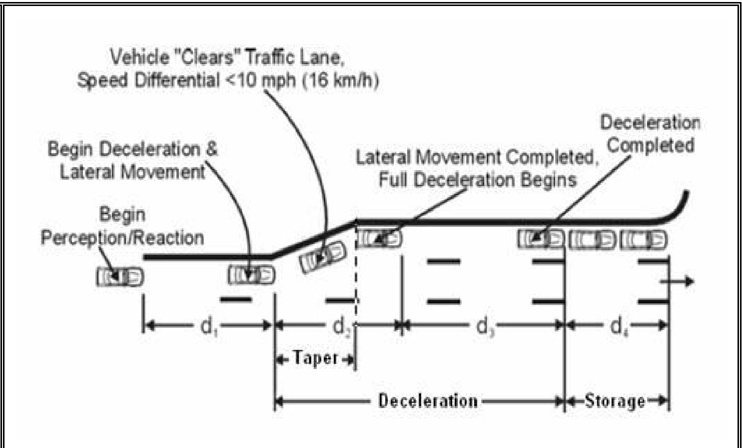
d_1: Distance traveled during perception-reaction time **d_2**: Distance traveled while driver decelerates and maneuvers laterally (4.5 fps^2) **d_3**: Distance traveled during full deceleration and coming to a stop or to a speed at which the turn can be comfortably executed (9.0 fps^2) **d_2+d_3**: Deceleration Length **d_4**: Storage length	

Figure 40: Left-Turn Lane Components
Source: Iowa Statewide Urban Design Standards Manual

9.1.1 Existing Methods for Estimating Bay Taper Length (Bay Taper Rate)

Short bay tapers may make vehicles be subjected to high decelerations and increase the potential of rear-end accidents. On the other hand, long bay tapers may result in through vehicles enter the left-turn lane unintentionally (especially when the bay taper is on a horizontal curve). The design of bay taper length is based on the speed before the vehicles entering the left-turn lane and the speed to which vehicles must reduce to complete lateral movement and begin full deceleration in the left-turn lane. Different methods have been recommended for bay taper length estimation. Following is a brief introduction of the existing methods.

AASHTO Method

AASHTO green book recommends short bay taper and a longer deceleration length for intersections with high traffic volume. The longer deceleration length can be used for storing vehicles due to the low speed during peak hours and also helps high-speed vehicles to decelerate during off-peak hours. According to AASHTO, a bay taper rate (Longitudinal:Transverse) between 8:1 and 15:1 is recommended for high-speed highways. The bay taper rate of 8:1 should be used for design speeds up to 30 mph, and 15:1 should be used for design speeds between 30 and 50 mph. AASHTO also suggested that bay taper length of 100 ft for a single left-turn lane, and 150 ft for double left-turn lane are used for urban streets. It can be found that AASHTO recommends the bay taper length for double left-turn lanes 1.5 times longer than the bay taper length for single left-turn lanes.

CDOT Method

Colorado Department of Transportation (CDOT) Design Guide Manual recommends different bay taper rates based on the different posted speed in the range of 25 mph to 70 mph. Table 57 presents the CDOT recommended taper rates.

Table 57: CDOT Recommended Bay Taper Rate for Left-Turn Lanes

Design Speed (mph)	25	30	35	40	45	50	55	60	65	70
Taper Ratio	7.5:1	8:1	10:1	12:1	13.5:1	15:1	18.5:1	25:1	25:1	25:1
50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1300 ft. Taper Length equals taper ratio times lane width.										

Koepke and Levinson Method

Koepke and Levinson (1992) recommended a 10:1 bay taper rate in single left-turn lane, and 7.5:1 in double left-turn lane for all posted speed limits. According to this recommendation, if the lane width is 12 ft, the bay taper should be 120 ft for single left-turn lane and 180 ft for double left-turn lanes, respectively. It is also found that the bay taper length recommended for double left-turn lane is 50 percent longer than the bay taper length for single left-turn lane.

FDOT Method

The Florida Department of Transportation (FDOT) Standards Index recommends the use of 4:1 ratio instead of 8:1 ratio for bay tapers on all multilane divided highways in the urban areas regardless of speed. Although this bay taper is relatively short, reduced bay taper length will increase the length for the deceleration area with full width. As a result, it causes less chance of a left-turning vehicle blocking through lanes (Figure 41). In addition, generally, vehicle speeds in the urban areas are not too high and it lessens the need for gradual tapers.

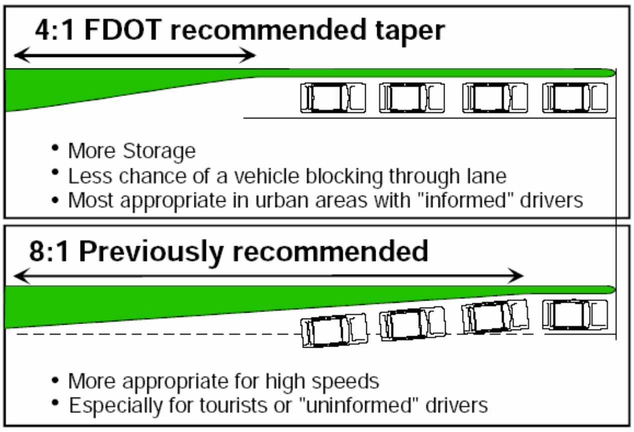


Figure 41: FDOT Recommended Bay Taper Rate
Source: FDOT Median Handbook

TxDOT Method

TxDOT Roadway Design Manual recommends bay taper length based on the speed and intersection geometric conditions in urban streets. The recommended bay taper length for single left-turn lanes is 50 ft for speed lower than 45 mph, and 100 ft for speed equal or higher than 45 mph. For double left-turn lanes, the bay taper length of 100 ft for speed lower than 45 mph, and 150 ft for speed equal or higher than 45 mph is recommended. Table 58 shows the bay taper length recommended by TxDOT Roadway Design Manual. Note that the results of the survey

(Chapter3) indicate that the bay taper lengths recommended by TxDOT Roadway Design Manual are too short.

Table 58: TxDOT Recommended Bay Taper Length for Left-Turn Lanes in Urban Streets

Speed (mph)	Taper Length (ft)	
	Single LT Lane	Double LT Lane
< 45	50	100
≥ 45	100	150

Neuman Method

Neuman (1985) estimated bay taper length by considering the speed and widths of lanes. Following equation was used for calculating bay taper length:

$$T_b = \frac{W_L \times S}{2.5} \tag{35}$$

where:

T_b: Length of bay taper (ft)

W_L: Width of lane (ft)

S: Speed (mph)

Table 59 shows typical values for T_b based on different speeds and different widths of lanes.

Table 59: Typical Values for T_b

S – Speed (mph)	W_L – Width of Lane (ft)		
	11	11.5	12
30	130	140	145
40	175	185	190
50	220	230	240
60	265	275	290

9.1.2 A Theoretical Method for Estimating Bay Taper Length

According to Figure 40, the distance traveled while driver decelerates and maneuvers laterally (d_2) can be calculated by using Velocity-Distance Equation, $D = \frac{V^2 - V_0^2}{2a}$, where *D* is the traveled distance, V_0 is the initial vehicle speed at the beginning of the taper, *V* is the final speed at the end of the taper, and *a* is acceleration. By assuming the speed differential is 10 mph ($V = V_0 - 10$), following equation can be derived:

$$d_2 = \frac{1.47^2 \times [V_0^2 - (V_0 - 10)^2]}{2 \times a} \tag{36}$$

where:

d_2: Distance traveled while driver decelerate and maneuvers laterally (ft)
V_0: Initial speed of the vehicle at the beginning of the taper (mph)
a: Deceleration (fps^2) (4.5 fps^2, according to Figure 40)

Finally, the length of bay taper is derived by subtracting the vehicle length from d_2(see Figure 40), which can be expressed by following equation:

$$T_b = d_2 - L \tag{37}$$

where:

T_b: Length of bay taper (ft)
d_2: Distance traveled while driver decelerate and maneuvers laterally (ft) (from Equation 36)
L: Average vehicle length (20 ft)

Table 60 presents the estimated lengths of bay tapers by using Equations (36) and (37) based on different speeds in the intersections.

Table 60: Bay Taper Length Based on the Proposed Method

Speed (mph)	Bay Taper Length (ft)
30	100
40	148
50	196
60	244

9.1.3 Recommended Bay Taper Lengths (Taper Rates)

The analysis of the existing methods shows that two factors are important in determining the length of bay taper: speed and geometric conditions (lane width, number of left-turn lanes). To recommend the most appropriate lengths for bay tapers, the taper lengths recommended by different methods were compared. Note that, some methods recommended bay taper rates (the ratio of bay taper length to lane(s) width) instead of taper lengths. Therefore, for comparison purpose, the bay taper rates were converted to the taper length by assuming 12 feet lane width. The comparison of the bay taper length recommended by different methods for single left-turn lanes were presented in Table 61.

Table 61: Comparison of Different Bay Taper Lengths* (Taper Rates) for Single Left-Turn Lanes (with 12-ft Lane Width)

Speed (mph)	FDOT	TxDOT	CDOT	Theoretical Method	AASHTO	Keopke and Levinson	Neuman
30	48 (4:1)	50 (4:1)	96 (8:1)	100 (8:1)	96 (8:1)	120 (10:1)	145 (12:1)
40	48 (4:1)	50 (4:1)	144 (12:1)	148 (12:1)	180 (15:1)	120 (10:1)	190 (16:1)
50	48 (4:1)	100 (8:1)	180 (15:1)	196 (16:1)	180 (15:1)	120 (10:1)	240 (20:1)
60	48 (4:1)	100 (8:1)	300 (25:1)	244 (20:1)	-	120 (10:1)	290 (24:1)

*The units of taper lengths are in feet.

Table 61 indicates that the Florida Department of Transportation (FDOT) Standards Index recommends the shortest taper lengths. Bay taper length recommended by TxDOT Roadway Design Manual is also shorter than the others, which agrees with the survey results. On the other hand, Neuman's taper length is the longest one. The taper lengths (taper rates) calculated using the proposed theoretical method are relatively long and they are consistent with those recommended by Colorado Department of Transportation (CDOT) Design Guide Manual.

Among all the studied methods, only three methods had studied taper lengths for double left-turn lanes: AASHTO, TxDOT, and Keopke and Levinson. Table 62 shows the comparison of the bay taper lengths recommended by these three methods for double left-turn lanes.

Table 62: Comparison of Different Bay Taper Lengths* for Double Left-Turn Lanes (assuming 12-ft Lane Width)

Speed (mph)	AASHTO	TxDOT	Keopke and Levinson
30	150	150	180
40	150	150	180
50	-	200	180
60	-	200	180

*The units of taper lengths are in feet.

Table 62 indicates that the lengths of the bay tapers recommended for double left-turn lanes by all the three methods are relatively close; and they are about 50 percent longer than the taper lengths recommended by those methods for single left-turn lanes.

By comparing the existing methods and their results and considering the survey results, the most appropriate bay taper lengths is recommended by considering three major factors: speed, geometric condition (number of left-turn lanes), and traffic conditions. In addition, the bay taper lengths are recommended based on 12-ft lane width assumption.

According to AASHTO green book and the Florida Department of Transportation (FDOT) Standards Index, in the urban areas, due to relatively high traffic volume and lower traffic speed, the bay taper length could be shorter and the deceleration length could be longer. And, the longer deceleration length can be used for storing more vehicles during peak hours to avoid any blockages in the adjacent through lane. Thus, the relatively short TxDOT taper lengths are recommended for the intersections in the urban areas. On the other hand, the relatively long taper lengths calculated by the proposed theoretical method are recommended for the intersections in the other areas. Finally, the recommended bay taper lengths for single left-turn lanes are given in Table 63.

Table 63: Recommended Bay Taper Lengths for Single Left-Turn Lanes

Speed (mph)	Bay Taper Length (ft)	
	Urban Areas	Other Areas
30	50	100
40	50	150
50	100	200
60	100	250

For double left-turn lanes, it is recommended to increase the lengths of bay tapers listed in Table 63 by 50 percent.

9.2 Signal Phasing Sequence

Signal phasing sequence is another import element related to the left-turn lane design and operations. Insufficient length of left-turn lane will result in the left-turn lane overflow and the blockage of left-turn lane entrance by through traffics. These two problems, which are referred to as left-turn overflow and blockage problems, will seriously increase the traffic delay and accident risk at intersections. Some methods, such as increasing the storage length of left-turn lane and using double left-turn lane, can be applied to directly solve these problems. However, in same cases, the length of the left-turn lane was limited by various local factors and cannot be increased as desired (such as no enough spare space for increasing or adding left-turn lanes). Another approach to mitigate the impacts of left turn overflow and blockage problems on left-turn operation is improving the signal phasing sequence.

9.2.1 Methodology

In this study, traffic simulation model (SYNCHRO/SimTraffic) was used to investigate the impacts of signal phasing sequence on left-turn operation under various traffic and roadway geometric conditions. Among the 28 intersections which were studied in the data collection part of this project (Chapter 4), four intersections with significant overflow and blockage problems were selected for investigation. The geometric layouts of these intersections and the problems for each intersection are presented in Figure 42**.**

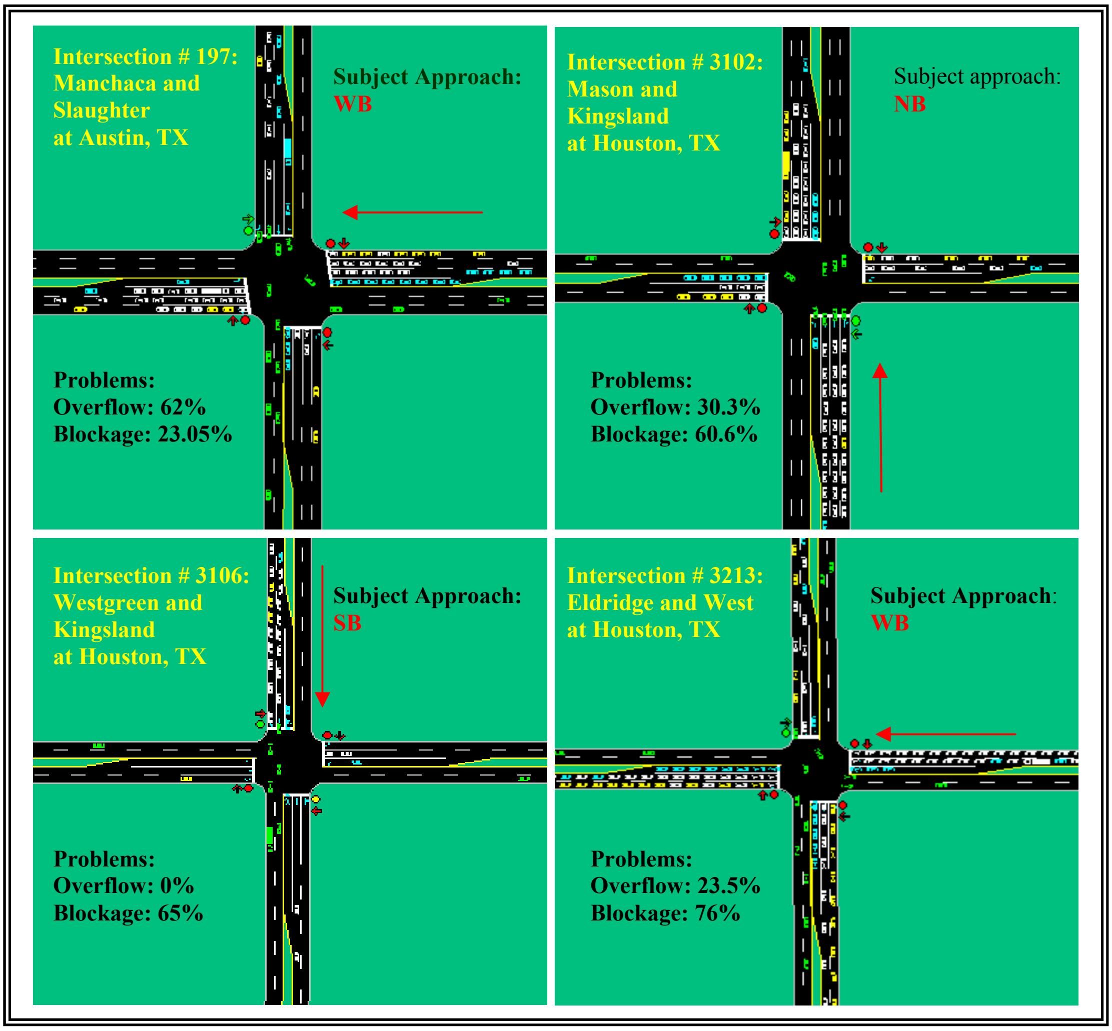


Figure 42: Geometric Layouts and Problems of Selected Intersections

For each intersection, the selected subject approach was investigated. The analysis procedure consists of following 4 steps:

<u>Step 1. Base Model Development:</u> input the information about intersection traffic, signal control and geometric conditions into SYNCHRO software based on the data collected from the field study. For the subject approach, the left-turn volume and the through traffic volume were collected from the field studies. For the other approaches, the traffic volumes were estimated base on the historical volumes and the volumes of the subject approach.

Step 2. Model Calibration: the developed base model is calibrated based on the three traffic measures collected from the field studies: (1) the maximum queue length in the subject left-turn lane, (2) the overflow rate of the subject left-turn lane and (3) the blockage rate, i.e. the rate that the left-turn lane entrance was blocked by the queue in the adjacent through lane. The calibration results are listed in Table 64. In Table 64, 95% QL indicates the 95% percentile observed queue length in number of vehicles (95% of the observed queue length is less than or equal to this number). The overflow and blockage rates are the percentages of the total cycles in that overflow and blockage problems were observed. The calibration results showed that the simulated results are quite close to what we observed in the field. Therefore, the base model is calibrated and it can be used for investigating the impacts of the signal phasing sequence on the left-turn operation in these intersections.

Table 64: SimTraffic Calibration Results for Study Intersections

Intersection ID	95% QL		LT Overflow Rate		LT Blockage Rate	
	SimTraffic	Field Data	SimTraffic	Field Data	SimTraffic	Field Data
197	12 veh	14 veh	>50%	62%	20%~30%	23.05%
3102	10 veh	10 veh	20~30%	30.3%	>50%	60.6%
3106	3 veh	4 veh	<1%	0%	>50%	65%
3213	11 veh	12 veh	20%~30%	23.50%	>50%	76%

Step 3. Alternative Analysis: model the intersections with alternative signal phasing sequences to derive the average traffic delays under the traffic signal controls with different phasing sequences.

Step 4. Simulation Results Analysis: the traffic simulation results under different signal phasing sequences were compared to find how the signal phasing sequence can impact the left-turn operations at the studied intersections. Based on the findings from the individual intersections, recommendations for selecting the best signal phasing sequence were made according to the left-turn operation conditions at the intersections. The analysis results were given in details in next section.

9.2.2 Results Analysis

In the result analysis, the average vehicle delays of each movement on each approach under different signal phasing sequences are presented in a table for comparison. Note that, the analysis will focus on the operation of the subject approach since the base simulation model was developed and calibrated based on the data collected from the subject approach. At the mean time, since the signal phasing sequence for the opposing approach will also change under the alternative signal phasing sequences, the operation efficiency of the opposing approach will also be considered in the analysis. On the other side, since the signal timing for the two approaches on the across street will not change, the traffic operations of these two approaches will not be analyzed and the vehicle delays for these two approaches are presented only for reference purposes.

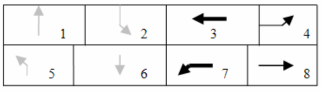
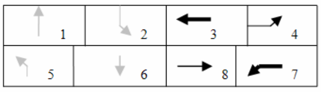
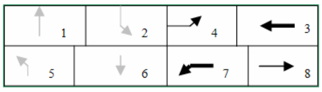
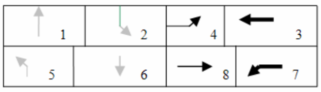
For each intersection, the analysis results are listed in one table. In each table, the results of the subject approach are in bold letters and shaded, and the results of the opposing approach are only shaded. In the phase diagram for each signal phasing sequence, the phases of the subject approach are in bold dark lines, the phases of the opposing approach are in dark lines and the phases of the other two approaches are in light color lines. In this study, the signal phasing sequence was evaluated or ranked based the average vehicle delay on the subject approach. The individual intersection results analyses are presented in the following.

9.2.2.1 Intersection # 197 (Manchaca & Slaughter, at Austin)

Subject approach: WB

Problems at the intersection: according to field observations, the WB overflow rate is 62% and WB blockage rate is 23.05%. From the simulation results, it was found that (1) the WB overflow rate was greater than 50% and blockage rate was between 20% and 30%, which matches the field observation; and (2) the EB overflow was less than 1% and EB blockage was around 50%.

Table 65: Results for Intersection 197, Manchaca & Slaughter, at Austin

Sequence		EB		WB		NB		SB		Total	Rank
Sequence 1 (Existing Sequence)	Approach	EB		WB		NB		SB		Total	
	Delay/Veh (s)	84.6		**58.1**		46.5		67.1		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	**WBL**	**WBT**	NBL	NBT	SBL	SBT	68.1	
	Delay/Veh(s)	93	82.5	**119.4**	**39.8**	82	48.1	107.2	75.7		**2**
Sequence 2 (Alternative Sequence1)	Approach	EB		WB		NB		SB		Total	
	Delay/Veh (s)	80.2		**62.7**		46.5		69.4		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	**WBL**	**WBT**	NBL	NBT	SBL	SBT	71.5	
	Delay/Veh(s)	74.8	82.6	**90.7**	**48.8**	82	48.1	107.4	80		**4**
Sequence 3 (Alternative Sequence2)	Approach	EB		WB		NB		SB		Total	
	Delay/Veh (s)	116.1		**49.7**		46.4		67.7		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	**WBL**	**WBT**	NBL	NBT	SBL	SBT	68.2	
	Delay/Veh(s)	160.7	82.4	**126.6**	**37.7**	82	48	107.4	75.1		**1**
Sequence 4 (Alternative Sequence3)	Approach	EB		WB		NB		SB		Total	
	Delay/Veh (s)	90.4		**61.2**		46.5		71.9		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	**WBL**	**WBT**	NBL	NBT	SBL	SBT	72.7	
	Delay/Veh(s)	108.3	82.9	**104.9**	**44.7**	82	48	107.5	83.4		**3**

<u>Results analysis:</u>

a. For the WB approach, the overflow problem is much more serious than the blockage problem. The results showed that sequence 3 is the best choice for WB movements because the WB through traffic delay (WBT) in sequence 3 was significantly less than that in the existing signal sequence 1. It is because, among the four sequences, the left-turn movement in sequence 3 starts earliest relative to the through movement in that direction. As a result, the through traffic delay caused by the blockage of through traffics by the overflow left-turn vehicles was significantly reduced. For this intersection, the best signal phasing sequence is sequence 3 in term of the traffic operations on the subject approach.

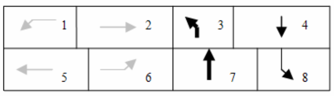
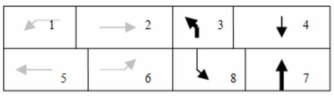
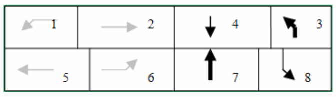
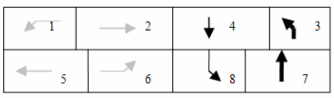
b. For the EB approach, according to the simulation results, the blockage problem is the only problem. The results showed that sequence 2 is the best choice for the EB movements; and it is found that the through movements in sequence 2 starts earliest relative to the left-turn movement in that direction. When blockage problem exist, it is better to let the through movement starts earlier than the left-turn movement to reduce the left-turn vehicle delay caused by the blockage of left-turn lane entrance.

9.2.2.2 Intersection # 3102 (Mason & Kingsland, at Houston)

<u>Subject approach:</u> NB

<u>Problems at the intersection:</u> According to field observations, the NB overflow rate is 30.3%, and NB blockage rate is 60.6%. From the simulation results, it was found that (1) the NB overflow rate was between 20% and 30%, and NB blockage rate was great than 50%, which matches the field observation; and (2) the SB overflow was less than 1% and SB blockage was between 20% and 30%.

Table 66: Results for Intersection 3102, Mason & Kingsland, at Houston

Sequence 1 (Existing Sequence)	Approach	EB		WB		**NB**		SB		Total	
	Delay/Veh (s)	59.7		111.0		**60.3**		40.8		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	WBL	WBT	**NBL**	**NBT**	SBL	SBT	84.8	
	Delay/Veh(s)	64.5	63.3	112.2	109.3	**98.1**	**57.1**	48.7	42.2		**3**
Sequence 2 (Alternative Sequence1)	Approach	EB		WB		**NB**		SB		Total	
	Delay/Veh (s)	59.7		107.5		**63.8**		43.5		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	WBL	WBT	**NBL**	**NBT**	SBL	SBT	86	
	Delay/Veh(s)	64.5	63.3	113.5	108.5	**114.7**	**56.8**	58.4	42.2		**4**
Sequence 3 (Alternative Sequence2)	Approach	EB		WB		**NB**		SB		Total	
	Delay/Veh (s)	59.3		110		57.6		39.6		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	WBL	WBT	**NBL**	**NBT**	SBL	SBT	83.6	
	Delay/Veh(s)	63.9	63	113.7	110.8	**63.6**	**59.9**	46.9	43.5		**1**
Sequence 4 (Alternative Sequence3)	Approach	EB		WB		**NB**		SB		Total	
	Delay/Veh (s)	59.3		110.4		**59.9**		42.6		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	WBL	WBT	**NBL**	**NBT**	SBL	SBT	83.8	
	Delay/Veh(s)	63.9	63	111	110.8	**84.4**	**59.3**	58.1	42.5		**2**

<u>Results analysis:</u>

a. For the NB approach, the blockage problem is much more serious than the overflow problem. The results show that sequence 3 is the best choice for NB movements because the NB left-turn delay (NBL) in sequence 3 was significantly less than those in other signal phasing sequences. It is because that the through movement in sequence 3 starts earliest relative to the left-turn movement in that direction. As a result, the left-turn traffic delay caused by the blockage of left-turn lane entrance by through traffics will be significantly reduced. For this intersection, the best signal phasing sequence is sequence 3 in term of the traffic operations on the subject approach.

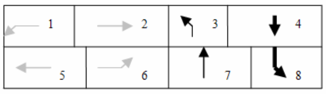
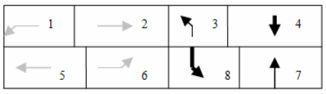
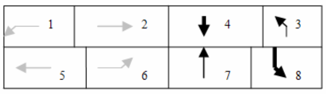
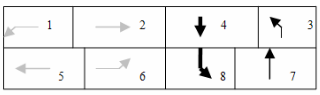
b. For the SB approach, the blockage problem is the only problem. So sequence 3 is still the best choice for SB movements due to the same reason.

9.2.2.3 Intersection # 3106 (Westgreen & Kingsland, at Houston)

Subject approach: SB

Problems at the intersection: According to field observations, the SB overflow rate is 0%, and SB blockage rate is 65%. From the simulation results, it was found that (1) the SB overflow rate was less than 1%, and NB blockage rate was great than 50%, which matches the field observation; and (2) the NB overflow and blockage were both less than 1%.

Table 67: Results for Intersection 3106, Westgreen & Kingsland, at Houston

Sequence											
Sequence 1 (Existing Sequence)	Approach	EB		WB		**NB**		**SB**		Total	
	Delay/Veh (s)	25.4		22		**23.7**		**45.9**		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	WBL	WBT	**NBL**	**NBT**	**SBL**	**SBT**	43.5	
	Delay/Veh(s)	21.1	31.8	35.7	23.5	**28.4**	**31.3**	**46.5**	**45.3**		**2**
Sequence 2 (Alternative Sequence1)	Approach	EB		WB		**NB**		**SB**		Total	
	Delay/Veh (s)	26		22		**21**		**50.6**		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	WBL	WBT	**NBL**	**NBT**	**SBL**	**SBT**	46.1	
	Delay/Veh(s)	21.1	31.8	35.6	23.5	**29**	**27.9**	**58.8**	**45.8**		**4**
Sequence 3 (Alternative Sequence2)	Approach	EB		WB		**NB**		**SB**		Total	
	Delay/Veh (s)	24.9		21.9		**23.2**		**44**		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	WBL	WBT	**NBL**	**NBT**	**SBL**	**SBT**	42.4	
	Delay/Veh(s)	21.1	31.7	35.6	23.3	**30**	**28.3**	**41.1**	**45.2**		**1**
Sequence 4 (Alternative Sequence3)	Approach	EB		WB		**NB**		**SB**		Total	
	Delay/Veh (s)	24.9		21.9		**22.2**		**46.2**		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	WBL	WBT	**NBL**	**NBT**	**SBL**	**SBT**	45.3	
	Delay/Veh(s)	21.1	31.7	35.6	23.3	**30.1**	**27.9**	**49**	**44.6**		**3**

Results analysis

a. For the SB movement, the blockage problem is the only problem. The results show that sequence 3 is the best choice for SB movements because the SB left-turn delay (SBL) in sequence 3 was significantly less than those in other signal sequences. It is because that the through movement in sequence 3 starts earliest relative to the left-turn movement in that direction. As a result, the left-turn traffic delay caused by the blockage of left-turn lane entrance by through traffics will be significantly reduced. For this intersection, the best signal phasing sequence is sequence 3 in term of the traffic operations on the subject approach.

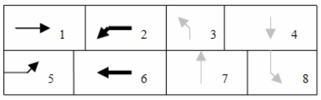
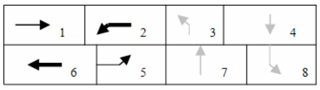
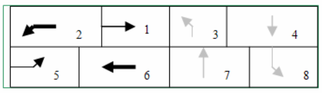
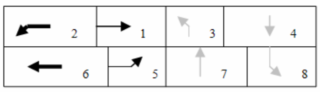
b. For the NB movement, there is no overflow and blockage problem. So the delay does not change a lot.

9.2.2.4 Intersection # 3213 (Eldridge & West, at Houston)

Subject approach: WB

Problems at the intersection: According to field observations, the WB overflow rate is 23.5% and WB blockage rate is 76%. From the simulation results, it was found that (1) the WB overflow rate was between 20% and 30%, and blockage rate was greater than 50%, which matches the field observation; and (2) the EB overflow was less than 1% and EB blockage was around 50%.

Table 68: Results for Intersection 3213, Eldridge & West, at Houston

Sequence 1 (Existing Sequence)	Approach	EB		**WB**		NB		SB		Total	
	Delay/Veh (s)	107.9		**103.2**		75.4		73.5		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	**WBL**	**WBT**	NBL	NBT	SBL	SBT	94.9	
	Delay/Veh(s)	103.1	110.2	**107.2**	**102.7**	110	76.2	90.1	72.1		**2**
Sequence 2 (Alternative Sequence1)	Approach	EB		**WB**		NB		SB		Total	
	Delay/Veh (s)	99.3		**96.6**		76.5		72.6		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	**WBL**	**WBT**	NBL	NBT	SBL	SBT	92.2	
	Delay/Veh(s)	60.6	110	**79.5**	**106.1**	110.5	77.7	90.8	69.3		**1**
Sequence 3 (Alternative Sequence2)	Approach	EB		**WB**		NB		SB		Total	
	Delay/Veh (s)	108		**119.8**		78.1		72.7		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	**WBL**	**WBT**	NBL	NBT	SBL	SBT	107.1	
	Delay/Veh(s)	103.1	110.8	**164.9**	**99.7**	111.4	77.3	91.3	71.8		**4**
Sequence 4 (Alternative Sequence3)	Approach	EB		**WB**		NB		SB		Total	
	Delay/Veh (s)	103.9		**109.6**		79		72.8		Delay/Veh(s)	**Rank**
	Movement	EBL	EBT	**WBL**	**WBT**	NBL	NBT	SBL	SBT	92.6	
	Delay/Veh(s)	73.4	110.9	**140.6**	**100.2**	114.8	77.5	92.9	68.5		**3**

Results analysis

a. For the WB approach, the blockage problem is much more serious than the overflow problem. The results show that sequence 2 is the best choice for WB movements because the WB left-turn delay (WBL) in sequence 2 was significantly less than those in other signal sequences. It is because that the through movement in sequence 2 starts earliest relative to the left-turn movement in that direction. As a result, the left-turn traffic delay caused by the blockage of left-turn lane entrance by through traffics will be significantly reduced. For this intersection, the best signal phasing sequence is sequence 2 in term of the traffic operations on the subject approach.

b. For the EB approach, the blockage problem is the only problem. Sequence 2 is still the best choice for EB movements due to the same reason.

9.2.3 Overall Findings

Based on the results analysis in the individual intersections, the following key findings were derived:

- Choosing appropriate signal phasing sequence can significantly reduce the vehicle delay caused by the left-turn overflow and blockage problems.
- When the overflow problem is much more serious than the blockage problem, the left-turn movement should start earlier than the through movement to reduce the traffic delay caused by the blockage of through traffics by the overflow left-turn vehicles.
- When blockage problem is much more serious than the overflow problem, the through movement should start earlier than the left-turn movement to reduce the left-turn vehicle delay caused by the blockage of left-turn lane entrance by through traffics.

9.3 Summary

In this chapter, two different issues related to the left-turn lane design and operation were investigated. In the first part of this chapter, existing recommendations on bay taper length as one of the important elements of left-turn lane design were reviewed. Then, a theoretical method for calculating the length of bay taper was introduced. Finally, by comparing all different methods and recommendations, the lengths of bay tapers were recommended based on different speed and different traffic conditions for both single and double left-turn lanes.

In the second part of this chapter, signal phasing sequence and its impact on left-turn operation was studied at four intersections with overflow and blockage problems at Austin and Houston districts. The simulation results showed that the vehicle delay caused by the overflow and blockage problem could be significantly reduced by choosing appropriate signal phasing sequence. It was recommended that, for the intersections with significant overflow problem, the left-turn movement should start earlier than the through movement and, for the intersections with significant blockage problem, the through movement should start earlier than the left-turn movement.

CHAPTER 10

CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

This research examined important issues related to the design and operation of left-turn lane. The results of this research provide answers to the following critical questions in left-turn deign and operation:

1. How long should the left-turn lane be?
2. When and where should multiple left-turn lanes be provided?
3. What are the safety benefits of extending the length of existing left-turn lanes?

For the first question, a new analytical model (TSU model) for determining the queue storage lengths of left-turn lanes at signalized intersections was developed by considering both parts of left-turn queue: (1) the vehicles that arrive during the red phase (red-phase queue), and (2) the queue of vehicles carried over from previous cycles (leftover queue). The evaluation results indicated that the developed model considerably outperforms the existing methods by providing more accurate estimates of left-turning queue lengths.

For the second question, two types of warrants for multiple left-turn lanes were developed: (1) the capacity and volume based warrants, and (2) the left-turn queue length based warrants. By combining the developed warrants with the existing warrants/guidelines, a decision-making flowchart for installing multiple left-turn lanes was developed. It provides comprehensive guidelines on multiple left-turn lane installation because both operational and safety impacts of multiple left-turn lanes were considered in the development of the guidelines.

For the third question, this research analyzed the safety benefits of increasing the storage lengths of left-turn lanes at intersection by two methods: (1) accident data analysis, and (2) simulation-based safety analysis. It was found that (1) the average rear-end accident at the intersections with left-turn overflow problem was 35 percent higher than that at the intersections without left-turn overflow problem; and (2) after extending the lengths of the left-turn lanes to eliminate the overflow problem at the study intersections, all of the safety surrogate measures derived from the traffic simulation results, changed significantly in a direction that indicated the reduction of rear-end accident risk at those intersections. These results concluded that extending left-turn lanes to eliminate the left-turn lane overflow problem significantly improved intersection safety by decreasing the rear-end accident risk.

In addition, this research investigated the estimation of left-turn storage length and deceleration length by using traffic simulation models. For left-turn storage length estimation, it was found that, among the three selected traffic simulation models, i.e. *SYNCHRO*, SimTraffic and VISSIM, SimTraffic model illustrated the best performance, VISSIM demonstrated relatively poor performance and the developed analytical model (TSU model) outperformed the selected traffic simulation models. For left-turn deceleration length estimation, a simulation-based method was developed by using VISSIM 4.20. It provided better deceleration length estimates than those recommended by analytical methods.

Finally, this research investigated two important issues related to left-turn lane design and operation: (1) left-turn bay taper length estimation, and (2) the impacts of signal phasing sequence on left-turn operation. By comparing all different methods and guidelines on left-turn bay taper length estimation, two different sets of bay tapers length were recommended for the intersections in urban areas and non-urban areas. By using traffic simulation based studies, it was found that the vehicle delay caused by the overflow and blockage problems could be significantly reduced by choosing appropriate signal phasing sequence.

10.2 Recommendations

Based on the results of the research conducted in this project, following recommendations on the left-turn lane design and operation are provided:

- Left-turn lane should be designed with adequate storage length. Both parts of left-turn queue need to be considered in the estimation of left-turn queue length. It is suggested that the developed analytical model (TSU model) be used for left-turn lane storage length estimation. The required storage length at different probability levels can be calculated based on the queue length estimates listed in a series of reference tables (Tables 33-37).
- Multiple left-turn lanes should be provided at the intersection where left-turn volume exceeds its capacity and extreme long left-turn queue exists. It is recommended that the developed decision-making flowchart in Figure 39 be used for determining the installation of multiple left-turn lanes.
- Extend the length of left-turn lane or update the single left-turn to multiple left-turn lanes for the intersections with left-turn lane overflow problem to reduce the rear-end crash risk.
- Longer bay taper lengths should be provided for intersections in the non-urban areas. The recommended bay taper lengths for single left-turn lanes are given in Table 63. For double left-turn lanes, the lengths of bay tapers need to be increased by 50 percent.
- Appropriate signal phasing sequence should be adopted to reduce the delay caused by left-turn lane overflow and blockage problems. For the intersections with significant overflow problem, the left-turn movement should start earlier than the through movement and, for the intersections with significant blockage problem, the through movement should start earlier than the left-turn movement.

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APPENDIX A

SURVEY FORM

Texas Department of Transportation (TxDOT)
Research Project 0-5290: Left-Turn Lane Design and Operation

Survey of Relevant Parameters

Dear traffic engineers and transportation professionals,

We need your help with completing a very important project on "Left-Turn Lane Design and Operation." Texas Southern University (TSU) is conducting a research project for TxDOT, which is to examine important issues related to the design and operation of left-turn lanes and to recommend best practices that could improve both safety and efficiency of intersections. The primary objectives of this research are to 1) develop procedures and methods for determining the required deceleration and storage lengths, and 2) identify criteria for multiple left-turn lane installation. The typical single left turn lane design is presented in Figure 1 for your reference.

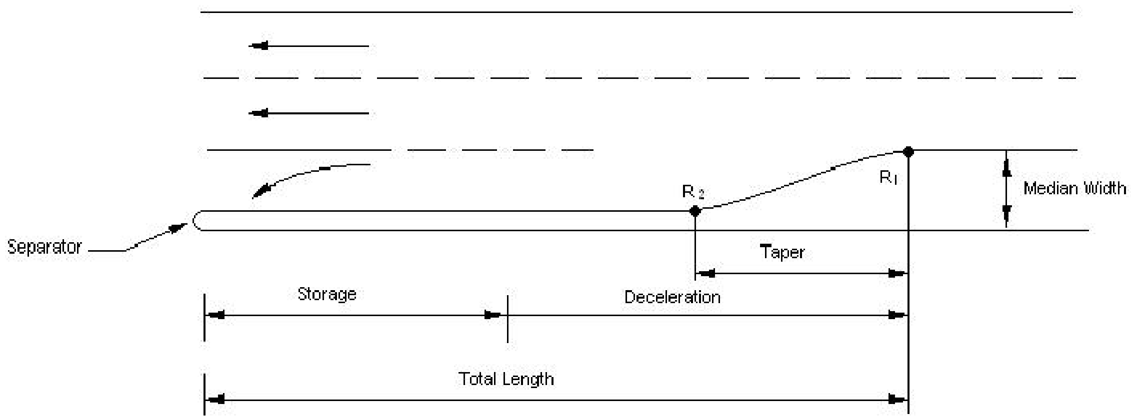


Figure A-1: Typical Single Left-Turn Lane Design

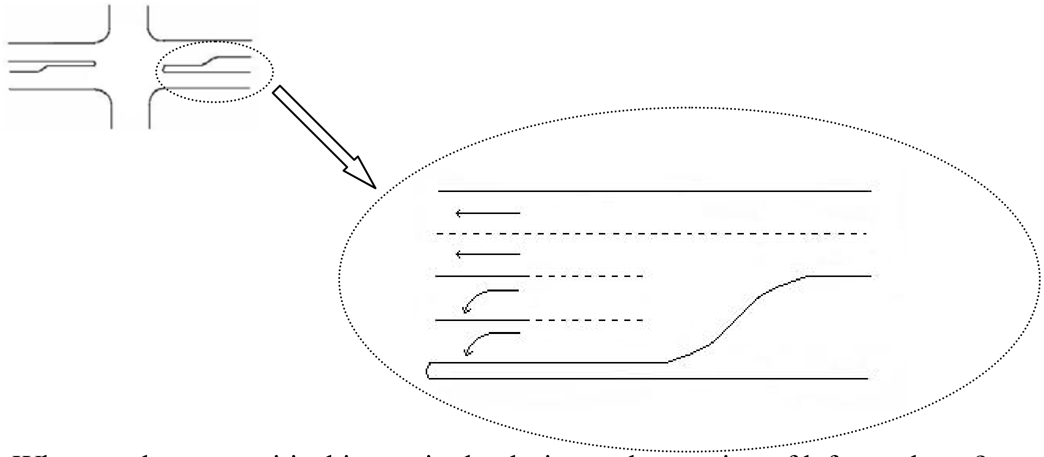
To achieve the research objectives, this survey is designed to seek your help in identifying and prioritizing all possible parameters. The parameters would potentially be included in the models for left-turn deceleration and storage lengths' determination and the warrants for multiple left-turn lane installation. Each parameter listed in the following table is given numbers from '1" to '5' with '1' indicating the lowest priority and '5' indicating the highest. Please grade each parameter by checking one box that represents the level of importance of the parameters (rows) in different aspects of left-turn lane design (columns). Please e-mail your response to yu_lx@tsu.edu or fax to (713) 313-1856 *before January 17 2006*. We appreciate your assistance with this survey.

Part I: Identification of Parameters Priority

Parameters		Different Aspects of Left-Turn Lane Design: Left-Turn Lane Deceleration and Storage Lengths (→ Priority Level)	Different Aspects of Left-Turn Lane Design: Warrants for Multiple Left-Turn Lanes (→ Priority Level)
Traffic Condition	Left-Turn Traffic Volume	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Opposing Traffic Volume	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Through Traffic Volume	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Vehicle Types/Fleet Compositions	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Intersection Congestion Level (*v/c*)	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
Geometric Condition	Grade	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Number of Left-Turn Lanes	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Number of Shared Lanes for Left-Turn Vehicles	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Number of Through Lanes	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
Driving Behavior	Average Speed at the Moment of Entering Left-Turn Lane	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Average Speed on Through Lane	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Deceleration and Acceleration Rate on Left Turn Lane	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
Traffic Control	Signalized and Unsignalized	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Signal Type: Pre-timed and Actuated	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Left-Turn Signal Type: Permitted or Protected	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Signal Cycle Length	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Phase Structure and Phase Length	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
Traffic Safety	Historical Accident Rate at Intersection	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	Historical Rate of Left-Turn Related Accident	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
Others*	_____	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	_____	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	_____	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	_____	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐
	_____	1☐ 2☐ 3☐ 4☐ 5☐	1☐ 2☐ 3☐ 4☐ 5☐

* If the space provided is not enough, please attach an additional sheet.

Part II: General Questions on Left-Turn Lane Design



1. What are the most critical issues in the design and operation of left turn lanes?

____

____

2. What are the most important criteria for evaluating the design of a left turn lane?

____

____

3. What is the existing practice on the determination of deceleration and storage length requirements in your agency?

____

____

4. What are the existing warrants for multiple left turn lanes in your agency?

____

____

5. Are there any good methods/experiences on the determination of deceleration and storage length requirements that can be shared with us?

____

____

6. Are there any good methods/experiences on developing the warrants for multiple left turn lanes that can be shared with us?

____

____

7. Additional Comments:

____

____

Part III: Acknowledgement

We appreciate your participation in this survey. Please provide the following contact information:
Name of the person who filled this survey:__________

Title: __________

Name of the Organization: _______________

Address: ____

Telephone: (_____). __________ Fax: (_____)

E-mail:_____

Website:_____

Please mail/fax/e-mail your response to the following address *before January 17 2006*:

Dr. Lei Yu, P.E.
Department of Transportation Studies, Texas Southern University
3100 Cleburne Avenue, Houston, Texas 77004
Telephone: (713) 313-7182;
Fax: (713) 313-1856
E-mail: yu_lx@tsu.edu
Website: http://transportation.tsu.edu/

APPENDIX B

A SAMPLE INTERSECTION SURVEY FORM

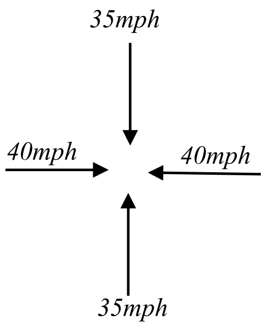
Intersection Survey Form - Houston

Name of Intersection: *Kingsland Blvd and S Mason Rd*
Camera ID: *3102*
Date: *04-13-06*
Time: *4:37 P.M.*
Layout of Intersection, Approach Speed Limits, and Location of Camera (☸):

Kingsland Blvd

S Mason Road

Approach Posted Speeds



Traffic Volume: ☐ Low ☒ Medium ☐ High
Type of Intersection: ☒ Signalized ☐ Unsignalized

Left-Turn Lanes Information:

Approach	Number of Lanes			Signal Control		
	LT	TH	RT	Protected	Permitted	Protected-Permitted
Northbound	*1*	*3*	*1-Share*	☒	☐	☐
Southbound	*1*	*3*	*1-Share*	☒	☐	☐
Eastbound	*1*	*2*	*1-Share*	☒	☐	☐
Westbound	*2*	*2*	*1-Share*	☒	☐	☐

Other Information Observed:
- *Both E-W and N-S streets are divided by median.*
- *Northbound and southbound traffic volume is higher.*
- *Since before the intersection in the westbound, there is a gap in median to let the vehicles turn left, the length of LT lane in the intersection had been short. Then double left-turn lane has been installed.*

APPENDIX C

QUEUE ESTIMATAION

D.1 Estimation of Queue Formed During the Red Phase

The probability that k left-turn vehicles arrive during the red phase is:

$$\Pr\text{ob}(\text{Arrivals in a red phase} < \text{k}) = \Pr\text{ob}(\text{k}) = \frac{(\lambda_t \text{R})^{\text{k}} e^{-\lambda_t \text{R}}}{\text{k}!} \qquad \text{(C-1)}$$

where:

λ_t : average arrival rate of left-turning vehicles in vehicles per second

R : duration of a red phase in seconds

So, λ_tR is the average number of arrivals during the red phase. Given the required probability level α, the maximum number of vehicles (Q_1) estimated to arrive during a red phase can be derived by the following equation:

$$\Pr\text{ob}(\text{Arrivals in a red phase} \le Q_1) = \sum_{0}^{Q_1} \Pr\text{ob}(\text{k}) = \sum_{0}^{Q_1} \frac{(\lambda_t \text{R})^{\text{k}} e^{-\lambda_t \text{R}}}{\text{k}!} = \alpha_1 \qquad \text{(C-2)}$$

From the Equation C-2, a reference table (Table 33) was developed to estimate the maximum queue length, Q_1, formed during the red phase based on a commonly observed range of λ_tR (the average number of arrivals during a red phase) at different probability levels (95%, 97.5%, 99%, and 99.5%).

D.2 Estimation of Leftover Queue at the End of Green Phase

Leftover queue at the end of the green phase is estimated using Discrete-Time Markov Chain (DTMC), which is a discrete random process. In DTMC, the state of the next point depends only on the state of the current point. In this study, the DTMC state is the number of vehicles in the queue at the end of the green phases (time points A, A+C, A+2C….A+nC in Figure 14 in Chapter 5). It is reasonable to assume that the queue length at the end of the next green phase depends solely on the queue length at the end of current green phase.

One-Step Transition Matrix P of DTMC

For the DTMC system, it is important to derive the one-step transition matrix P (see Figure C-1). In this P matrix, p_{ij} is the probability that the leftover queue length (in number of vehicles) is i at the current time point and becomes j at the next time point.

	0	1	2	……	j	……	ϕ
0	p_{00}	p_{01}	p_{02}	……	p_{0j}	……	
1	p_{10} I	p_{11}	p_{12}	……	p_{1j}	……	
2	p_{20}	p_{21}	p_{22}	……	p_{2j}	……	
⋮							
i	p_{i0}	p_{i1}	p_{i2}	……	p_{ij} II	……	
⋮							
m^*	p_{m0}	p_{m1}	p_{m2}	……	p_{mj}	……	
m+1	0	$p_{m+1,1}$	$p_{m+1,2}$	……	$p_{m+1,j}$	……	
m+2	0	0	$p_{m+2,2}$	……	$p_{m+2,j}$	……	
	0	0	0				
⋮							
ϕ							

* m is the maximum number of vehicles that could be discharged during the green phase.

Figure C-1: One-step Transition Matrix P

The P matrix is divided into three homogeneous parts: I, II and III. Before discussing the development of p_{ij} in each part, the following concepts are introduced:

1. P_k^C : the probability of k left-turning vehicles arriving during a signal cycle (from the current time point to the next time point). Because the vehicles that arrive at the left-turn lane follow a Poisson distribution, P_k^C can be derived by the following equation:

$$P_k^C = \Pr ob(\text{Arrivals in a cycle} = k) = \frac{(\lambda_t C)^k e^{-\lambda_t C}}{k!} \qquad \text{(C-3)}$$

Similar to Equation C-2, where:

λ_t average arrival rate of left-turning vehicles in vehicles per second

C : signal cycle length in seconds

So, $\lambda_t C$ is the average number of arrivals during a signal cycle.

2. m is the intersection service rate, i.e., the maximum number of vehicles that can be discharged during a green phase, including both protected and permitted left-turn phases.

   - The number of vehicles that can be discharged during the protected left-turn phase, i.e., m_1, can be estimated by the following equation:

$$m_1 = \text{Nearest Integer to}\left(\frac{g_{\text{effective}}}{T_L}\right) = \text{Nearest Integer to}\left(\frac{g_p - \ell_1 + e}{T_L}\right) \qquad \text{(C-4)}$$

where:

$g_{\text{effective}}$: duration of the effective protected green phase

g_p : duration of the protected green phase

ℓ_1 : start-up lost time

e : yellow encroachment time

T_L : headway of left-turning vehicles departing the intersection

The value of ℓ_1, e and T_L can be determined through field observations or can use the default values suggested in Roess, et al. (2004) as follows: ℓ_1 or e = 2 seconds and T_L = 2.1 seconds.

- The number of vehicles that can be discharged during a permitted left-turn phase , i.e., m_2, can be estimated by the following equation:

$$m_2 = \text{Nearest Integer to}\left(\frac{g_m}{T \times E_{LT}}\right) \qquad \text{(C-5)}$$

where:

g_m : duration of the permitted green phase

T: average headway of vehicles departing the intersection (the default value is 2 seconds)

E_{LT} : left-turn equivalence, whose value is determined by the opposing volume and the number of opposing lanes (see Table C-1)

Table C-1: Through Vehicle Equivalents for Left-Turning Vehicles, E_{LT}

Opposing Volume V_o (veh/h)	Number of Opposing Lanes, N_o		
	1	2	3
0	1.1	1.1	1.1
200	2.5	2.0	1.8
400	5.0	3.0	2.5
600	10.0*	5.0	4.0
800	13.0*	8.0	6.0
1,000	15.0*	13.0*	10.0*
≥ 1,200	15.0*	15.0*	15.0*
E_{LT} for all *protected* left-turns = 1.05			

* Indicates that the LT capacity is only available through "sneakers." (Roess et al, 2004)

Finally, the total number of vehicles that can make the left turn during both the protected green phase and permitted green phase can be estimated as follows:

$$m = m_1 + m_2 \tag{C-6}$$

where m_1 and m_2 are given in Equations C-4 and C-5.

A detailed discussion of the calculation of p_{ij} in each part of the one-step transition matrix P follows:

1. Part I: p_{ij} where $i = 1, 2, \ldots m$ and $j = 0$

This part indicates that all the vehicles in the queue in the current cycle will be discharged at the end of the green phase of the next cycle. Since the intersection service rate is m (vehicles per cycle) and the leftover queue length in the current cycle is i ($i < m$), the number of arrivals in this cycle should be equal to or less than $m - i$ to clear all the arrivals in a cycle,. Therefore, the individual element p_{ij} of the transition matrix in this part can be calculated as follows:

$$p_{ij} = \text{Prob (Arrivals in a cycle} \le \text{m-i)} = \sum_{0}^{m-i} P_k^C \tag{C-7}$$

where P_k^C is given in Equation C-3.

2. Part II: p_{ij} where $j > 0$ and $i - j \le m$

This part indicates that a left-turning queue carryover will occur in the next cycle. Because the intersection service rate is m (vehicles per cycle) and the leftover queue length is i in the current cycle and will become j in the next cycle, $m + j - i$ vehicles should arrive in the intersection in a cycle. Therefore, the individual element p_{ij} of the transition matrix in this part can be calculated by:

$$p_{ij} = \text{Prob (Arrivals in a cycle = m+j-i)} = P_{m+j-i}^C \tag{C-8}$$

3. Part III: p_{ij} where $i-j>m$

This part indicates that the leftover queue length is i in this cycle and will become j in the next cycle. Because m is the maximum number of vehicles that can be discharged during a green phase, it is impossible to discharge more than m vehicles ($i-j>m$) in one cycle. Therefore,

$$p_{ij} = 0 \tag{C-9}$$

Based on this discussion, a one-step transition matrix P of the DTMC was developed and is presented in Figure C-1. To calculate the stationary distribution of this Markov chain, an arbitrarily large number, ϕ, was selected as the upper bound of the leftover queue length (the probability that the leftover queue is greater than ϕ is close to 0). Thus, the matrix P becomes a $\phi \times \phi$ matrix with the elements $p_{i\phi} = 1 - \sum_{j=0}^{\phi-1} p_{ij}$. Because an intersection is assumed to be a stable system (*v/c* < 1), it can be proven that a stationary distribution of this DTMC exists.

Stationary Probability Row Vector of DTMC

Based on the developed P matrix, the stationary probability of a given number of leftover vehicles at the end of a green phase can be estimated by the following equation:

$$\pi P = \pi \tag{C-10}$$

where π is the DTMC stationary probability row vector, whose elements π_i ($i = 1......\phi$) represent the stationary probability of i vehicles leftover at the end of a green phase, which can be mathematically expressed as follows:

$$\pi_i = \text{Prob(Number of leftover vehicles = i)} \tag{C-11}$$

Therefore, given the required probability level α, the maximum leftover queue length, Q_2, can be estimated by the following equation:

$$\Pr ob(\text{Leftover number of vehicles} < Q_2) = \sum_{i=1}^{Q_2} \pi_i = \alpha_2 \qquad \text{(C-12)}$$

From the Equation C-12, a series of reference tables (Tables 34, 35, 36, and 37) were developed to estimate the maximum leftover queue length, Q_2, based on a commonly observed range of $\lambda_t C$ (the average number of arrivals during a cycle) and m (intersection service rate, vehicles per cycle) at different probability levels (95%, 97.5%, 99%, 99.5%).

APPENDIX D

MODELING INTERSECTION DELAY FOR SINGLE, DOUBLE, AND TRIPLE LEFT-TURN LANES

Notation:

S : the saturation flow rate

C : the signal cycle length

N : the total phase number

t_L : the total lost time for each phase

λ : the percentage of the effective green time for these two movements in the whole signal timing cycle

g : the effective green time

V : the volume of the vehicles

T : the time period in which the overflow delay is calculated

c : the capacity

V_{LT} : the total left-turn volume

V_T^o : the opposing through volume

S_{LT} : the saturation flow rate of the left-turn movement

S_T^o : the saturation flow rate of the opposing through movement

V_{LT}^{1c} : the critical left-turn lane volume for the single left-turn lane (left-turn volume when the volume to capacity ratio equals to one)

V_{LT}^{2c}: the critical left-turn lane volume for the double left-turn lanes (left-turn volume when the volume to capacity ratio equals to one)

V_{LT}^{2} : the average left-turn volume per lane for the double left-turn lanes

V_{LT}^{3} : the volume of the triple left-turn lanes

c_{LT} : the capacity of the left-turn movement

c_{T}^{o}: the capacity of the opposing through movement

g_{LT} : the effective green time for the left-turn movement

g_{T}^{o}: the effective green time for the opposing through movement

D : the average delay of the left-turn vehicles and the opposing through movements

UD_{aLT} : the aggregate uniform delay of the left-turn movement

UD_{aT}^{o} : the aggregate uniform delay of the opposing through movement

$\overline{UD_{aLT}}$: the aggregate uniform delay of the left-turn movement when the volume to capacity ratio equals to one

$\overline{UD_{aT}^{o}}$: the aggregate uniform delay of the opposing through movement when the volume to capacity ratio equals to one

OD_{aLT} : the aggregate overflow delay of the left-turn movement

OD_{aT}^{o} : the aggregate overflow delay of the opposing through movement

A model will be developed for estimating the delay of single, double and triple left-turn lanes as a function of left-turn volume by keeping updating signal timing according to the change of left-turn volume. According to Roess et al. (2003), the effective green time should be reallocated according to the ratios of traffic volume and saturation flow rate (v/s ratios) in competing directions. In this study, for simplification purpose, it is assumed that 1) the intersection cycle signal length is a constant, i.e. C, 2) left-turn signal control is protected only control, 3) the green time reallocation are just between the subject left turn movement (which is the critical left-turn movement) and its competing through movement (which is the critical through traffic volume), and 4) the total green timing for these two movements counts for λ percentage of cycle length . These four assumptions are illustrated in Figure D-1 (signal phase diagram) and Figure D-2 (assumed intersection scenario).

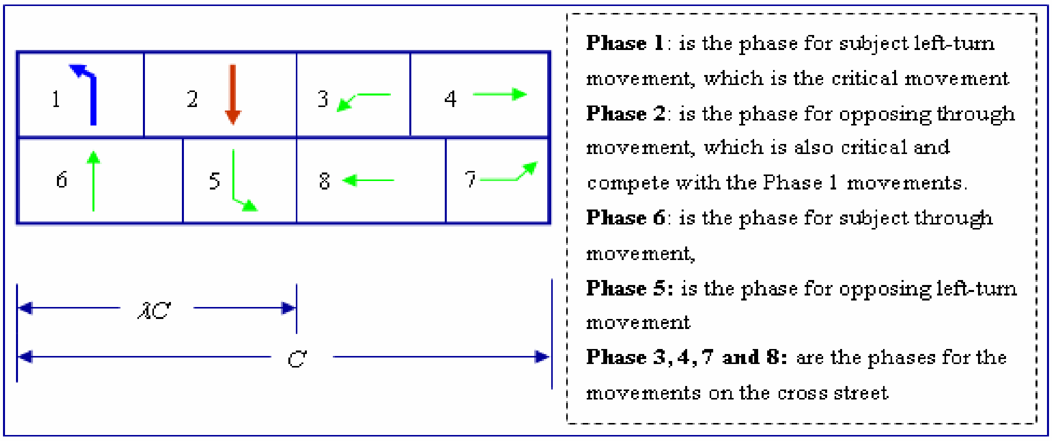


Figure D-1: Signal Phase Diagram

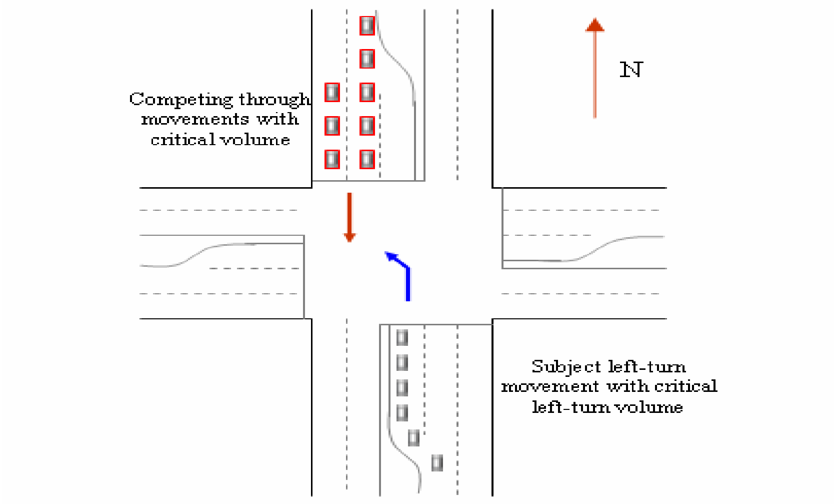


Figure D-2: Assumed Intersection Scenario

D.1 Delay Estimation Model

The delay estimation model was developed based on the queuing theory diagram by Roess et al. (2003). The delay includes not only the uniform delay but also the overflow delay which can be illustrated in Figure D-3.

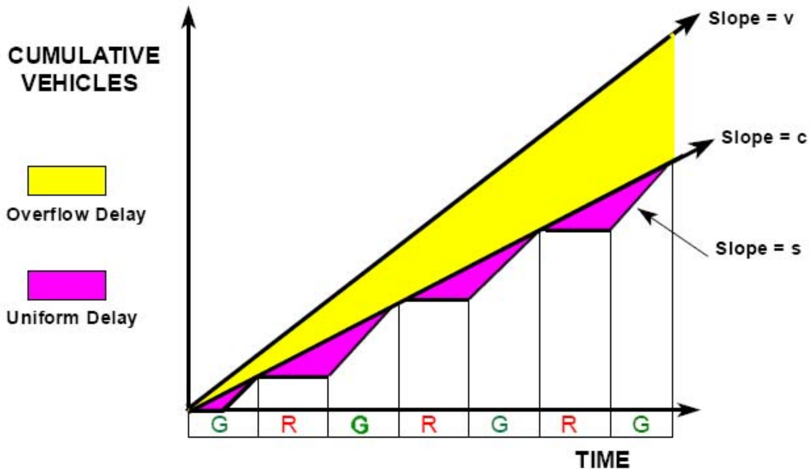


Figure D-3: The Illustration of Overflow Delay and Uniform Delay
Source: Roess et al. (2003)

When the volume to capacity ratio (*v/c*) is less than or equal to one, only the uniform delay needs to be considered and the expression of the aggregate uniform delay by Roess et al. (2003) is:

$$UD_a = \frac{1}{2}C^2[1-\frac{g}{C}]^2[\frac{V}{1-V/S}], \tag{D-1}$$

where:

g : the effective green time,

V : the volume of the vehicles,

S : the saturation flow rate,

C : the signal cycle length.

When the *v/c* is greater than one, the overflow delay also needs to be considered and the expression of the aggregate overflow delay by Roess et al. (2003) is

$$OD_a = \frac{T^2}{2}(V-c), \tag{D-2}$$

where:

T : the time period in which the overflow delay is calculated,

V : the volume of the vehicles,

c : the capacity.

Based on the this model, the delay of single, double and triple left-turn lanes as a function of left-turn volume can be derived under the assumption that the signal timing are keep updating according to the increase of left-turn volume.

D.2 Delay for Single Left-Turn Lane

The average delay of the subject left-turn and opposing through vehicles was calculated under different *v/c* conditions.

D.2.1 Average Delay When $v/c \leq 1$

The average delay of the subject left-turn and opposing through vehicles has the following form:

$$UD = \frac{1}{(V_T^o + V_{LT})C}(UD_{aLT} + UD_{aT}^o)\,, \tag{D-3}$$

where:

UD_{aLT} : the aggregate uniform delay of the left-turn movement,

UD_{aT}^o : the aggregate uniform delay of the opposing through movement,

C : the signal cycle length,

V_{LT} : the volume of the left-turn movement,

V_T^o : the volume of the opposing through movement.

Based on Equation D-1, UD_{aLT} and UD_{aT}^o can be expressed in the following forms:

$$UD_{aLT} = \frac{1}{2}C^2[1 - \frac{g_{LT}}{C}]^2[\frac{V_{LT}}{1 - V_{LT}/S_{LT}}] \tag{D-4}$$

and

$$UD_{aT}^o = \frac{1}{2}C^2[1 - \frac{g_T^o}{C}]^2[\frac{V_T^o}{1 - V_T^o/S_T^0}]\,, \tag{D-5}$$

where:

V_{LT} : the volume of the left-turn movement,

g_{LT} : the effective green time for the left-turn movement,

V_T^o : the volume of the opposing through movement,

g_T^o : the effective green time for the opposing through movement,

S_{LT} : the saturation flow rate of the left-turn movement,

S_T^0 : the saturation flow rate of the opposing through movement.

As we state before, the effective green time for the left-turn movement is the function of left-turn volume. The signal timing should be upgraded according to the left-turn volume. According to Roess et al. (2003), the effective green time was allocated cording to the volume to saturation flow rate (*v/s*) in the competing directions. In other words, the percentages of the effective green time should be equal to the percentages of v/s ratio for the competing movements. Therefore, the effective green time for the left-turn movement and opposing through movement can be determined by following equations

$$EffectiveGreentime\% = \frac{g_{LT}}{\lambda(C - Nt_L)} = v / sRatio\% = \frac{V_{LT} / S_{LT}}{V_T^o / S_T^o + V_{LT} / S_{LT}}$$

$$\Rightarrow g_{LT} = \lambda(C - Nt_L)\frac{V_{LT} / S_{LT}}{V_T^o / S_T^o + V_{LT} / S_{LT}} \qquad \text{(D-6)}$$

and $$EffectiveGreentime\% = \frac{g_T^o}{\lambda(C - Nt_L)} = v / sRatio\% = \frac{V_T^o / S_T^o}{V_T^o / S_T^o + V_{LT} / S_{LT}}$$

$$\Rightarrow g_T^o = \lambda(C - Nt_L)\frac{V_T^o / S_T^o}{V_T^o / S_T^o + V_{LT} / S_{LT}}. \qquad \text{(D-7)}$$

where:

λ : the percentage of the effective green time for these two movements in the whole signal timing cycle,

N : the total phase number,

t_L : the total lost time for each phase.

Thus, UD_{aLT} and UD_{aT}^o can be estimated by substituting Equations D-6 and D-7 into Equations D-4 and D-5 and has the following forms:

$$UD_{aLT} = \frac{1}{2}C^2\{[1 - (\lambda(1 - \frac{Nt_L}{C})\frac{V_{LT} / S_{LT}}{V_T^o / S_T^o + V_{LT} / S_{LT}})]^2 \cdot \frac{V_{LT}}{1 - V_{LT} / S_{LT}}\} \qquad \text{(D-8)}$$

and $$UD_{aT}^{o}=\frac{1}{2}C^{2}\{[1-(\lambda(1-\frac{Nt_{L}}{C})\frac{V_{T}^{o}/S_{T}^{o}}{V_{T}^{o}/S_{T}^{o}+V_{LT}/S_{LT}})]^{2}\cdot\frac{V_{T}^{o}}{1-V_{T}^{o}/S_{T}^{o}}\}. \qquad \text{(D-9)}$$

Then the average delay per vehicle of these two movements can be obtained by putting the estimated UD_{aLT} and UD_{aT}^{o} into Equation D-3 as follows:

$$UD=\frac{C}{2}(\frac{1}{V_{T}^{o}+V_{LT}})\{[1-(\lambda(1-\frac{Nt_{L}}{C})\frac{V_{LT}/S_{LT}}{V_{T}^{o}/S_{T}^{o}+V_{LT}/S_{LT}})]^{2}\cdot\frac{V_{LT}}{1-V_{LT}/S_{LT}}$$
$$+[1-(\lambda(1-\frac{Nt_{L}}{C})\frac{V_{T}^{o}/S_{T}^{o}}{V_{T}^{o}/S_{T}^{o}+V_{LT}/S_{LT}})]^{2}\cdot\frac{V_{T}^{o}}{1-V_{T}^{o}/S_{T}^{o}}\}. \qquad \text{(D-10)}$$

D.2.2 Average Delay When $v/c>1$

When the *v/c* is grater than one, the average delay per vehicle of these two movements includes not only the uniform delay but also the overflow delay and it can be expressed as follows:

$$D=UD+OD=\frac{1}{(V_{T}^{o}+V_{LT})C}(\overline{UD_{aLT}}+\overline{UD_{aT}^{o}})+\frac{1}{(V_{T}^{o}+V_{LT})T}(OD_{aLT}+OD_{aT}^{o}), \qquad \text{(D-11)}$$

where:

$\overline{UD_{aLT}}$: the aggregate uniform delay of the left-turn movement when the volume to capacity ratio equals to one,

$\overline{UD_{aT}^{o}}$: the aggregate uniform delay of the opposing through movement when the volume to capacity ratio equals to one,

OD_{aLT} : the aggregate overflow delay of the left-turn movement,

OD_{aT}^{o} : the aggregate overflow delay of the opposing through movement.

In order to find the expression of $\overline{UD_{aLT}}$ and $\overline{UD_{aT}^{o}}$, we need to obtain the left-turn volume when the *v/c* =1 (referred to as critical left-turn volume). Based on the g_{LT} given in Equation D-6, the capacity of the left-turn movement can be determined by

$$c_{LT}=S_{LT}\cdot\frac{g_{LT}}{C}=\lambda(1-\frac{Nt_{L}}{C})\frac{V_{LT}}{V_{T}^{o}/S_{T}^{o}+V_{LT}/S_{LT}}. \qquad \text{(D-12)}$$

Thus, the volume to capacity ratio of the left-turn movement is

$$\frac{V_{LT}}{c_{LT}}=\frac{V_T^o/S_T^o+V_{LT}/S_{LT}}{\lambda(1-\frac{Nt_L}{C})}. \qquad \text{(D-13)}$$

Similarly, the volume to capacity ratio of the opposing through movement can be derived as follows:

$$\frac{V_T^o}{c_T^o}=\frac{V_T^o/S_T^o+V_{LT}/S_{LT}}{\lambda(1-\frac{Nt_L}{C})}. \qquad \text{(D-14)}$$

From Equations D-12 and D-14, it can be seen that the volume to capacity ratio of the left-turn movement and opposing through movement are equal. This is reasonable, because the green time allocation is based on the volume to saturation flow rate ratio for each direction. From Equation D-12, the left-turn volume for single left-turn when *v/c* =1 can be derived as follows:

$$V_{LT}^{1c}=S_{LT}[\lambda(1-\frac{Nt_L}{C})-V_T^o/S_T^o]. \qquad \text{(D-15)}$$

Then, $\overline{UD_{aLT}}$ can be derived by substituting Equation D-15 into Equation D-8 and has the form:

$$\overline{UD_{aLT}}=\frac{1}{2}C^2S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}][1-\lambda(1-\frac{Nt_L}{C})+\frac{V_T^o}{S_T^o}]. \qquad \text{(D-16)}$$

Similarly, $\overline{UD_{aT}^o}$ can be determined by substituting Equation D-15 into Equation D-9 and has the form:

$$\overline{UD_{aT}^o}=\frac{1}{2}C^2(1-\frac{V_T^o}{S_T^o})V_T^o. \qquad \text{(D-17)}$$

By substituting the estimated left-turn lane capacity given in Equation D-12 into Equation D-2, OD_{aLT} can be derived as:

$$OD_{aLT}=\frac{T^2}{2}(V_{LT}-c_{LT})=\frac{T^2}{2}V_{LT}[1-\frac{\lambda(1-\frac{Nt_L}{C})}{V_T^o/S_T^o+V_{LT}/S_{LT}}]. \qquad \text{(D-18)}$$

Similarly, OD_{aT}^o can be derived as:

$$OD_{aT}^o=\frac{T^2}{2}(V_T^o-c_T^o)=\frac{T^2}{2}V_T^o[1-\frac{\lambda(1-\frac{Nt_L}{C})}{V_T^o/S_T^o+V_{LT}/S_{LT}}]. \qquad \text{(D-19)}$$

Finally, by substituting these four estimated delays, i.e. $\overline{UD_{aLT}}$, $\overline{UD^o_{aT}}$, OD_{aLT} and OD^o_{aT}, into Equation D-11, the average delay per vehicle of these two movements for single left-turn lane can be obtained as follows:

$$D=\frac{C}{2}\frac{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]}{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]+V_T^o}[1-\lambda(1-\frac{Nt_L}{C})+\frac{V_T^o}{S_T^o}]$$

$$+\frac{C}{2}(1-\frac{V_T^o}{S_T^o})\frac{V_T^o}{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]+V_T^o}+\frac{T}{2}[1-\frac{\lambda(1-\frac{Nt_L}{C})}{V_T^o/S_T^o+V_{LT}/S_{LT}}]. \tag{D-20}$$

D.2.3 General Formula for Average Delay under Single Left-Turn Lane Condition

Based on the discussions in sections D.2.1 and D.2.2, by combining the equations for delays under the $v/c\le 1$ and $v/c>1$ conditions, the general formula for the average delay per vehicle of the subject left-turn and its opposing through movements under single left-turn lane condition can be expressed as:

$$D=\begin{cases}\frac{C}{2}(\frac{1}{V_T^o+V_{LT}})\{[1-(\lambda(1-\frac{Nt_L}{C})\frac{V_{LT}/S_{LT}}{V_T^o/S_T^o+V_{LT}/S_{LT}})]^2\cdot\frac{V_{LT}}{1-V_{LT}/S_{LT}} \\ +[1-(\lambda(1-\frac{Nt_L}{C})\frac{V_T^o/S_T^o}{V_T^o/S_T^o+V_{LT}/S_{LT}})]^2\cdot\frac{V_T^o}{1-V_T^o/S_T^o}\} & (V_{LT}/c_{LT}\le 1) \\ \frac{C}{2}\frac{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]}{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]+V_T^o}[1-\lambda(1-\frac{Nt_L}{C})+\frac{V_T^o}{S_T^o}] & (V_{LT}/c_{LT}>1) \\ +\frac{C}{2}(1-\frac{V_T^o}{S_T^o})\frac{V_T^o}{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]+V_T^o}+\frac{T}{2}[1-\frac{\lambda(1-\frac{Nt_L}{C})}{V_T^o/S_T^o+V_{LT}/S_{LT}}] & \end{cases}. \tag{D-21}$$

D.3 Delay for Double and Triple Left-Turn Lanes

For double left-turn lanes, the average delay of the left-turn and the opposing through vehicles can be obtained by the same method as that for single left-turn lane. The only difference is that, for the double left-turn lanes, the average left-turn volume per lane is equal to the half of the total left-turn volume, which can be expressed as follows

$$V_{LT}^{2} = \frac{V_{LT}}{2}. \tag{D-22}$$

where V_{LT}^{2} is the average left-turn volume per lane for the double left-turn lanes.

Thus, according to Equation D-15, the critical left-turn volume for double left-turns (left-turn volume when v/c =1) can be derived as follows:

$$V_{LT}^{2c} = 2S_{LT}[\lambda(1-\frac{Nt_L}{C}) - V_T^o / S_T^o]. \tag{D-23}$$

By replacing the V_{LT} in Equation D-21 with $V_{LT}^{2} = \frac{V_{LT}}{2}$, the average delay per vehicle under double left-turn lanes conditions can be derived as:

$$D = \begin{cases} \frac{C}{2}(\frac{1}{V_T^o + V_{LT}})\{[1-(\lambda(1-\frac{Nt_L}{C})\frac{V_{LT}/2S_{LT}}{V_T^o/S_T^o + V_{LT}/2S_{LT}})]^2 \cdot \frac{V_{LT}}{1-V_{LT}/2S_{LT}} \\ +[1-(\lambda(1-\frac{Nt_L}{C})\frac{V_T^o/S_T^o}{V_T^o/S_T^o + V_{LT}/2S_{LT}})]^2 \cdot \frac{V_T^o}{1-V_T^o/S_T^o}\} & (V_{LT}^2/c_{LT} \le 1) \\ \\ \frac{C}{2}\frac{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]}{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]+V_T^o}[1-\lambda(1-\frac{Nt_L}{C})+\frac{V_T^o}{S_T^o}] \\ +\frac{C}{2}(1-\frac{V_T^o}{S_T^o})\frac{V_T^o}{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]+V_T^o}+\frac{T}{2}[1-\frac{\lambda(1-\frac{Nt_L}{C})}{V_T^o/S_T^o+V_{LT}/2S_{LT}}] & (V_{LT}^2/c_{LT} > 1) \end{cases}. \tag{D-24}$$

Similarly, by replacing the V_{LT} in D-21 with $V_{LT}^{3} = \frac{V_{LT}}{3}$, the average delay per vehicle under triple left-turn lanes conditions can be expressed as:

$$D=\begin{cases}\frac{C}{2}\left(\frac{1}{V_T^o+V_{LT}}\right)\{[1-(\lambda(1-\frac{Nt_L}{C})\frac{V_{LT}/3S_{LT}}{V_T^o/S_T^o+V_{LT}/3S_{LT}})]^2\cdot\frac{V_{LT}}{1-V_{LT}/3S_{LT}} \\ +[1-(\lambda(1-\frac{Nt_L}{C})\frac{V_T^o/S_T^o}{V_T^o/S_T^o+V_{LT}/3S_{LT}})]^2\cdot\frac{V_T^o}{1-V_T^o/S_T^o}\} & (V_{LT}^3/c_{LT}\le 1) \\ \\ \frac{C}{2}\frac{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]}{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]+V_T^o}[1-\lambda(1-\frac{Nt_L}{C})+\frac{V_T^o}{S_T^o}] & (V_{LT}^3/c_{LT}>1) \\ +\frac{C}{2}(1-\frac{V_T^o}{S_T^o})\frac{V_T^o}{S_{LT}[\lambda(1-\frac{Nt_L}{C})-\frac{V_T^o}{S_T^o}]+V_T^o}+\frac{T}{2}[1-\frac{\lambda(1-\frac{Nt_L}{C})}{V_T^o/S_T^o+V_{LT}/3S_{LT}}] \end{cases} \quad . \qquad \text{(D-25)}$$